Formwork for Concrete Structures

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Dr. Kumar Neeraj Jha
Department of Civil Engineering
IIT Delhi

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केन्द्रीय लोक निर्माण विभाग
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Central Public Works Department
Nirman Bhawan, New Delhi - 110011
Tel. : 23062556 / 1317 Fax : 23061884
E-mail : cpwd_dgw@nic.in

सत्यमेव जयते

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Foreword

Formwork is an important constituent of concrete structures, yet it has not received due attention in civil engineering curriculum. Very few books with limited coverage are available on this subject. The formwork is essential for all kinds of civil engineering structures right from roads, bridges, flyovers, building towers, tunnels, ports, jetties, drainage to sewerage system. It is a fact that cost, quality, schedule, performance, and safety of structures depend on the quality of formwork, its design, and management.

Dr. K.N. Jha has made an effort to write an exhaustive book covering almost all known types of formwork from simple to complex structures. The book has become unique because of its exhaustive coverage and use of locally available material and more importantly use of S.I. units throughout. It is quite clear that this book has both practical and theoretical inputs. A lot of practical examples throughout the book will ensure that the concepts are learnt properly and retained in one's memory. The examples used in this book very much relate to the field conditions and therefore the readers will be able to utilize it optimally. The book brings out so many untouched features and hence deserves a generous appreciation.

The book shall be useful for students, research scholars and practicing professionals in engineering and architecture. I recommend this valuable edition for educational institutions and professional departments/bodies like NTs, NITs, IBC, IRC, CIDC, CPWD, PWDs, NBCC, NTPC, Railways, MES, CWC, MCD, NDMC, BSNL, EPIL, EIL, NITHIE, CBRI, CRRI, NCCBM etc.

I am sure this book will fulfil the long cherished objectives of the engineering requirement.

C.S. Prasad

(C.S. Prasad)

Foreword

Concrete construction has found wide acceptability in almost all countries of the world. The preference of concrete construction over structural steel construction for roof structures depends on the economy, durability, sustainability, time of completion and other factors. Based on this the popularity of concrete structures vis-à-vis steel structures varies but it can be said without any controversy that concrete structures are still very popular. In the case of India concrete construction has grown by leaps and bounds in the last two decades as indicated by the national GDP growth of about 8-9% and cement production having exceeded 200 million tonnes out of a capacity of 300 million tonnes, thus taking India to the second position in the world. The expenditure on infrastructure in the 12th Five Year Plan is projected to be over US $ 1 trillion (approx. Rs.50,00,000 crore). This excludes the housing sector where the shortage is over 22 million housing units. All this would mean a continuous increase in efficient use of concrete so that the projects are completed with good quality, in short time, economically and safely and in an environment-friendly fashion in the years to come.

Concrete technology and its applications in India have improved considerably in the last decade, bringing high popularity to better methods of construction viz. use of ready-mix concrete in the urban areas, prefabricated steel reinforcement and good formwork. Cement, stone aggregates, sand and water make good concrete as well as not-so-good concrete, sometimes leading to undesirable problems like honeycombing, inferior surface quality and permeability to water, and durability. While the concrete from ready mix and reinforced bars is of assured quality, the same is not true about formwork. While formwork constitutes only around 20% of the cost of concrete the quality and time of completion depend upon the formwork methods adopted. In high-rise construction the cycle time for construction can be between 3 and 5 days as against 2-3 weeks with traditional formwork methods. A great majority of contractors continue to use the traditional timber-based systems which are improperly made causing problems such as inadequate safety during construction, permeability to water, durability, etc. Therefore, it can be said without hesitation that formwork technology assumes prime importance in terms of achieving high quality, fast construction, safety and overall economy in the long term to assure durability and sustainability and aesthetic look for the concrete surfaces.

Many contractors have now taken up the use of proprietary system formwork and modular formwork marketed by various international and Indian companies and there is a good sign of owners and government departments appreciating its advantages and specifying better quality system formwork now readily available. The only problem seems to be that the initial investment on such formwork is a deterrent and hiring industry has yet not developed. In the smaller cities and rural areas traditional formwork methods continue to be used and this too requires a gradual shift to system formwork which has standard components in steel, aluminium and timber in the form of ‘H’ beams and trusses, all of which offer more durability and overall economy due to more number of repeated uses in many projects. De-shuttering or removal and re-fixing is very slow due to some non-standard practices and waiting for 2-3 weeks before the forms are struck. This practice also needs to change on the basis of technical strength and deflection considerations to be taken to ensure more repetitive use of the same formwork components to bring in economy. The productivity of system formwork is 7 times to 10 times more when compared to traditional formwork in addition to the fact that the components used in a system formwork last 5-7 years bringing in economy besides speed and safety.

Historically system formwork was introduced by Larsen & Toubro Limited in 1985 by having a technical collaboration with Messrs. Doka of Austria to manufacture, use and sell formwork including climbing formwork and automatic climbing formwork for cooling towers and other special structures. Due to import restrictions some international companies could not sell their products in India. This was at a time when Acrow system was available to some extent which was more scaffolding than a regular system formwork. Ballies and bamboos are being used even today due to the initial expenditure being lower and the material being lost practically in one project.

While the formwork scenario is changing and some companies like Acrow, Doka, PERI, Meva, Paschal, FUVI Coppha, Noe, Mivan, Titan, L&T, Hünnebeck, Bridgebuilder are marketing their products, it is necessary to enhance the knowledge of practicing engineers as well as students and supervising technical personnel of government departments to see the subject is better understood and the use of formwork becomes systematic. Apart from the international books on the subject hardly any Indian textbook is available for students and practising engineers to learn more on the subject. It is only recently that system formwork is being exhibited in exhibitions and there is lot of interest among practising engineers to continuously learn as has been practised all over the developed world in exhibitions like BAUMA in Munich and INTERMAT in Paris. Hitherto concrete formwork has not been taught properly at the universities and polytechnics and practising engineers have not applied their mind and time to plan and engineer the formwork right in the beginning. This book will definitely remove the void we have in India for such quality literature, especially in the area of concrete formwork. I am certain the book will find wide acceptance and usage by engineers all over India and the neighbourhood as it deals comprehensively with practical examples. I compliment the author Dr. Kumar Neeraj Jha and the publishers Tata McGraw-Hill for bringing out this useful book and wish them success as this has the potential to ensure quality concrete construction and economic growth of India.

A. Ramakrishna
Former President & Deputy Managing Director
Larsen & Toubro Limited
Chennai
April 2012

Preface

It gives me great pleasure to present to you the first edition of this book. The book is the result of my twelve years of field experience working with Larsen and Toubro Limited and seven years of teaching under graduate and graduate students at IIT Kanpur and IIT Delhi, consulting, research, and organizing training programs for teachers and practitioners.

Formwork is an important constituent of RC construction. It is well known that quality, safety, and economy all are influenced tremendously by formwork, yet it has not got the treatment it deserves. Students are hardly aware of formwork and scaffolding and the lack of knowledge makes them feel that it is not an engineer's job. However, the readers would realize that formwork offers maximum opportunity of applying engineering and managerial skill. A proper formwork design coupled with proper management can provide a great opportunity in saving in formwork.

It is with this intention that the book has been organized to present and expose the readers with different types of formwork and scaffolding systems. In the beginning an introduction to different aspects of formwork has been provided. Different formwork materials and their properties are presented next. Subsequently the basics of formwork design have been presented.

Different types of formworks such as foundation, column, wall, and slab and beam formwork have been discussed. Both the conventional and proprietary formworks are discussed. Special formwork, Bridge formwork, flying formwork and slipform are presented next. Formwork systems presented by some leading manufacturers are discussed at length.

Formwork supports and scaffolds are discussed at length. Formwork for precast concrete elements requires different approach than that of cast-in-situ elements and accordingly they have been dealt with separately. Formwork management plays an important role in cost saving-accordingly pre-award and post-award formwork management issues are discussed separately in detail. Poor formwork leads to a number of failures and injuries to workmen and others. These issues have been dealt with separately under formwork failure. The issues in formwork faced in multi-storeyed construction specially the reshoring etc. have also been dealt appropriately.

The references at the end of the book, solved examples in different chapters, and review questions in different chapters will be found useful by the readers. I am eager to receive the comments from the readers of the book.

Dr. Kumar Neeraj Jha

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A book of such nature would not have been possible without the support and assistance of a number of people. First, I would like to thank Mr. S. Raghunath-my first mentor at L&T ECC for making me understand the importance of formwork. I wish to thank my trainers Mr. K.P.Raghwan, Mr. S.Natrajan, and Mr. B. Murugesan for the system formwork and slipform training I received from them at different points of time. I am also thankful to Mr. V.B. Gadgil for making me think differently in formwork application. I also wish to thank my other colleagues in the Larsen and Toubro Limited especially in formwork and construction method cell- namely Mr. A.L. Sekar, Mr. Navneet Kaul, Mr. Anil Kumar, Ms K. Bhawani, Mr. Harpal Singh, and Mr. C.S.Negi.

I would like to thank my colleagues, at the Department of Civil Engineering, IIT Delhi who contributed their time on a number of occasions discussing the contents of the book. I would like to thank my colleagues from other IIT's specially IIT Kanpur, IIT Madras, and IIT Guwahati. I would like to place on record the encouragement I received from Prof. S.N. Sinha, Prof. A.K. Jain, Prof. B. Bhattacharjee, Prof K.C. Iyer, Prof. A.K. Mittal, Prof. G.S. Benipal, Prof A K Singh, Prof. Koshy Varghese, and Prof. K.N. Satyanarayana on various occasions during the manuscript preparation. I am thankful to Prof. Vasant Matsagar for going through proofs of the book and for suggesting changes. I am thankful to Prof Suresh Bhalla for going through the design examples of the book and to Prof Shashank Bishnoi for reviewing certain portions of the book.

I would like to thank Dr. A Ramakrishna and Shri C.S. Prasad for going through the proof of the book and writing the foreword. I am thankful to Dr. A Ramakrishna for the complements I received from him. The country owes a lot to him for the work he has carried out for the construction industry in general and formwork and precast in particular.

I am thankful to Prof Sudhir Misra for the encouragements I receive from him time to time. The comments received from my colleagues at Larsen and Toubro Limited on the practical contents provided in the book is thankfully acknowledged. I am thankful to Mr. V.P. Sinha for going through some of the practical examples of the book. I am thankful to the management of Larsen and Toubro Limited in giving me suitable opportunities to work in different departments and in a number of projects during my association with them for about 12 years.

I am thankful to M/s PERI, M/s SGB, M/s Harsco Infrastructure, M/s Doka, M/s Larsen and Toubro Limited and many other leading manufacturers for granting me the permission and approval to use some of their products in the book.

I am thankful to Col Chitkara for the constant encouragement I receive from him and for introducing me to McGraw Hill Group of Companies. The support provided by staff members of McGraw Hill Group of Companies Mr. Sohan Gaur, Mr Simanta Borah, and Mr Sushil Gupta, and the typesetter M/s Shubham Composers is also thankfully acknowledged.

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I cannot forget my friend Prof Lalit Manral for inspiring me to take up the academic profession. I thank him for being the source of inspiration for me at all times.

I don't find any word to describe the efforts of my parents for all they have done for my upbringing. Without their blessings this book would not have been completed. I hope to continue to get their blessings. And of course, there are no words for the gratitude and love I feel for my family members, Arti (wife), Srijan (elder son) and Sajal (younger son) who, from time to time encouraged me to complete this book even at the cost of my time to be spent with them.

Dr. Kumar Neeraj Jha

Contents

Chapter

1 Introduction

Contents: Introduction; Formwork as a Temporary Structure; Requirements for Formwork; Selection of Formwork; Classification (Types) of Formwork; Organization of the Book

1.1 INTRODUCTION

There are different types of temporary structures which are used as enablers for constructing permanent structures. Some of the examples of temporary structures are: concrete formwork (shuttering) construction, scaffolding, falsework/shoring, cofferdams, underpinning, diaphragm/slurry walls, earth-retaining structures and construction dewatering. As can be noticed from these examples, the purpose of temporary structures vary. For example, a diaphragm wall may be used for the purpose of deep excavation, while a cofferdam may be constructed for making a bridge pier inside a water body.

Formwork is also a kind of temporary structure whose purpose is to support its own weight and that of freshly placed concrete as well as the construction live loads including materials, equipment and workmen. Formwork as a temporary structure is desired to safely support the concrete until it gains adequate strength to stand on its own.

According to IS: 6461 (Part 5)–1972, formwork is a complete system of temporary structure built to contain fresh concrete so as to form it into the required shape and dimensions and to support it until it hardens sufficiently to become self-supporting. It includes the surface in contact with the concrete and all the necessary supporting structure.

The terms sheeting (sheathing), form (shutter), falsework, centering, mould, and scaffold (scaffolding) are commonly used in the context of formwork, which must be clearly understood. The brief definition of these terms according to IS: 6461 (Part 5)–1972 is given below:

1. **Sheeting (Sheathing)** - That part of formwork, which is in contact with the concrete.
2. **Form (Shutter)** - (a) That part of formwork, which consists of the sheeting and its immediate supporting or stiffening members. (b) A temporary structure or mould for the support of concrete while it is setting and gaining sufficient strength to be self-supporting.
3. **Falsework** - (a) Falsework is the temporary structure erected to support the work in the process of construction. It is composed of shores, formwork for beams or slabs (or both) and lateral bracing. (b) That part of formwork, which supports the forms, usually for a long structure, such as a bridge.

4. **Centering (Centering)** - It is a temporary supporting structure to a soffit. It is the specialized formwork used in the construction of arches, shells space structure where the entire falsework is struck or decentered as a unit, to avoid introducing injurious stress in any part of the structure.
5. **Mould** - A frame for casting, precast concrete units.
6. **Scaffold (Scaffolding)** - A temporary structure for gaining access to higher levels of the permanent structure during construction.

1.2 FORMWORK AS A TEMPORARY STRUCTURE

Reinforced concrete (RC) construction primarily consists of three components: formwork, reinforcement, and concrete. Formwork represents a significant portion of any concrete construction project and accounts for 35-50% of the total concrete structure cost. In terms of time also, formwork operations consume maximum time, varying between 50-75% of the total time consumed in RC construction. Formwork is a different kind of a temporary structure in the following senses:

Unlike other temporary structures such as diaphragm wall and cofferdam, it can be erected and dismantled quickly.

Formwork components are highly loaded for a few hours during concrete placement. Hence we can design these components by allowing higher permissible stresses as compared to the stresses taken for the design of permanent structures. Although rigorous design procedure may not be required for a typical formwork, it is recommended for a complicated formwork, and in situations where the risk to human lives is involved.

Also, unlike other temporary structures, formwork can be disassembled within a few days for future use. The dismantling or deshuttering time may vary from structure to structure and is dependent on a number of characteristics. This issue has been discussed in detail at other places in the text.

The term "Temporary Structures" may sometimes give a wrong picture, since some of the formwork components such as steel forms, tie hardware, and accessories are used hundreds of times thus requiring high durability and maintainability characteristics and design that maximizes productivity.

1.3 REQUIREMENTS FOR FORMWORK

The formwork engineer is concerned with more than simply making forms of the right size; his objectives are threefold: Quality, Safety, and Economy. These aspects are discussed in detail in the following paragraphs.

1.3.1 Quality

Even a casual inspection of the concrete surface just after the formwork has been dismantled, would reveal number of defects in the concrete surface in most of the cases. We can notice plywood grains or timber stuck to the concrete surfaces at a number of locations, undulations in concrete surfaces, and patches of concrete showing honeycombing. Sometimes we may also find columns and beam bulging out from the desired line, and a heap of concrete slurry near the joint or dowel locations. All these defects can be attributed to poor formwork, either partly or fully. These defects are very costly to be repaired, and no amount of repair work can bring it back to the desired level of quality.

Some common defects observed after the stripping of formwork, primarily due to poor formwork, have been shown in Figs. 1.1(a) to (e).

Keeping these in mind, the formwork needs to be designed and built accurately so that the desired size, shape, position, correct location, quality, and finish of acceptable quality of the cast concrete are attained. Some of the measures to achieve the quality objectives of formwork are given below:

Formwork should suit the architectural and structural requirement. Sometimes one need not provide any treatment such as plastering or painting over the concrete surfaces, that is, they are supposed to be left in as-is position. These surfaces are also known as fare faced concrete surfaces. We need to provide special arrangements to cater to the needs of fare faced concreted surfaces.

Formwork should be able to be detailed well, to avoid damage to the concrete while deshuttering.

Joints in the formwork should be tight enough to prevent slurry leakage. Many ills of concrete construction can be avoided if we are able to have a tight joint, so that there is no leakage of slurry.

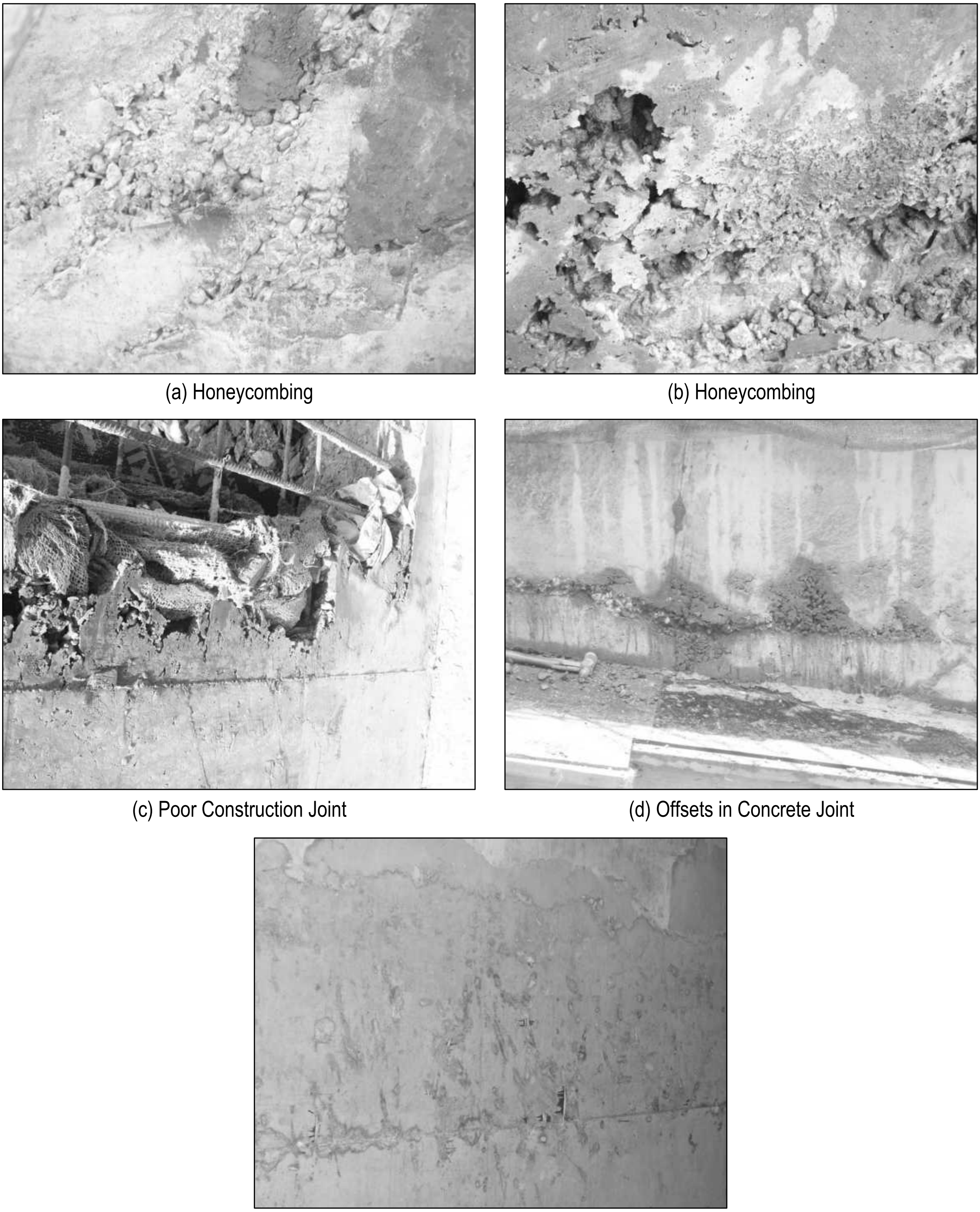
According to IS: 14687–1999, formwork should be made in such a manner that the finished concrete is in proper position in the space measured with respect to certain predefined reference points. Formwork should be of the proper dimensions and shape as per the drawings. Section 9.6 of IS: 14687–1999 provides guidelines for tolerances on the shape, lines, and dimensions.

Formwork should be able to be constructed within tolerance limits. This is extremely important, more so in the case of precast elements. If the precast elements are not cast within tolerable deviation limits, a number of problems are faced at the time of erection of these elements and joining of different elements.

1.3.2 Safety

The literature on formwork, Occupational Safety & Health Administration (OSHA) reports, etc. suggest that formwork construction is associated with a relatively high frequency of disabling injury and illness. Based on the 1997 OSHA accident statistics, Huang and Hinze (2003) report that 5.83% of falls were attributed to the construction of formwork or the construction of temporary structures, and 21.2% of all struck by accidents, involved wood framing or formwork construction. It is unfortunate that numerous lives are lost due to formwork failure every year even today. We come across news reports about formwork and scaffolding failure every now and then.

According to another study, in building construction, 60% failure is due to formwork collapse, shoring collapse, inadequate shoring and inadequate lateral bracing; 8% due to premature removal of shore; and 18% failure is observed due to faulty material. Har (2002) based on ergonomic studies suggests that the repetitive activities of lifting, sawing, and hammering commonly performed by formwork carpenters, lead to a high frequency of low-severity injuries such as discomfort and persistent pain.



(a) Honeycombing

(b) Honeycombing

(c) Poor Construction Joint

(d) Offsets in Concrete Joint

(e) Plywood Grains Stuck on the Concrete Surface

Figure 1.1 Illustration of Defects in Concrete Due to Poor Formwork Quality

This calls for greater safety in formwork operation. Formwork needs to be built adequately so that it is capable of supporting all the dead and live loads without danger to the workmen and to the concrete surface. Not only should the formwork be strong and safe, but it should also have arrangements for working platform where the workers can work without the danger of an injury. Formwork should have arrangements which can prevent the falling of objects from its working platform, so that injury to those working under the platform can be avoided.

1.3.3 Economy

Economy of formwork corresponds to an economical, careful, efficient, and prudent use of resources for making of the forms. Economy is a major concern since formwork is the largest single cost segment of the reinforced cement concrete construction cost in a project most of the time.

The total formwork cost is the sum of the following:

- cost of form materials,
- cost of labor for making, fixing, and removing,
- cost of equipment required to handle the form,
- cost of consumables such as form release agents or deshuttering agents such as diesel, oil, grease,
- other consumables such as nails, binding wire, cotton wastes, etc.

The cost may also include repair and maintenance cost of the form to keep it under working condition. Some storage cost component may also be there in the total formwork cost.

It may also be noted that the forms for a concrete structure may cost more than the concrete or the reinforced steel, and in some instances, they may cost more than the concrete and the reinforcing steel together.

There are two categories of costs associated with formwork costing: investment cost and cost per use. The distinction between the formwork investment cost and the cost per use must be clearly understood. The investment cost in form material (excluding plywood and timber) per m^2 may be in the range of Rs. 4,000–5,000, depending on the type of formwork system used. In tonnage terms, the formwork material with steel component, costs in the range of Rs. 45,000–50,000 per ton. In a large project the tonnage requirement may be in the range of 1,000 to 1,500 tons. So one can imagine the investment that is needed towards the formwork materials cost. Remember, this is without the plywood and timber cost. However, the formwork materials (steel component) can be used a number of times (sometimes more than 100 repetitions). Hence the cost per use for the material component may be in the range of Rs. 40-50. One may also notice that the formwork cost would also depend on the number of repetitions of the forms, the salvage value, and the cost of finishing the concrete surfaces, after the forms are removed. Sometimes contractors, instead of investing in the formwork material, borrow them on rent which is charged either daily or monthly.

From the discussion in the preceding paragraph, we can understand that any saving in the formwork cost can have large implications on the overall saving potential in a construction project. Hence it is extremely important to plan a structural system which is not only economical as far as the concrete quantity and reinforcement are concerned, but should also take formwork cost into consideration. This requires a systems approach as far as RCC is concerned.

To summarize, formwork needs to be designed and constructed so as to possess the specified strength and rigidity, and to ensure accuracy of locations, configuration and dimensions of permanent structures, so as to achieve the required quality of structure. This makes it imperative for the formwork to be made with the right materials, the right system, and the right detailing. In short, the entire operation should be kept simple. The standardization of form and its reuse is the key to economy. The summary of requirements for formwork system is given in Box 1.1.

BOX 1.1: The requirements from any formwork

- The formwork must be built and erected in such a way that the required shape, size, position, quantity, and finish of the concrete are obtained.
- The formwork must be strong enough to take the pressure or weight of the fresh concrete, so that the finish of the concrete is obtained. The material should not warp or get distorted.
- The formwork should be designed and constructed such that it can be easily and quickly erected and struck with minimum skilled workforce leading to savings in both time and money. It should be able to be set accurately to the desired line and levels.
- The formwork must be able to be struck without damage to the concrete or to the formwork itself.
- The formwork must be able to be handled using the available equipment or to be manually handled, if necessary.
- The formwork arrangement must provide safe access for the handling of concrete and its placing.
- The formwork must have all the necessary safety arrangement relating to working areas and platforms.
- The formwork arrangement must be flexible enough to get the desired finish. That is, it should be able to accommodate different types of sheathing material, e.g., plywood, steel etc.
- The formwork should also be able to accommodate any architectural features such as grooves, surface grains etc. It should be possible to attain tight joints so that there is no leakage of cement grout.

It is also very important to have a proper communication and coordination between the client, the consultant, the architect, and the contractor in order to affect maximum economy. The economy in a formwork can be affected right from the planning and design stage to the construction stage of the project.

1.4 SELECTION OF FORMWORK

The selection of a particular formwork system is governed by a number of factors. These factors can be divided into quantitative and qualitative factors. While quantitative factors include factors such as the cost and construction time, the qualitative factors include expected familiarity, flexibility, quality, and safety considerations. Elazouni *et al.* (2005) applied the Analytic Hierarchy Process (AHP) to compare the performance of the candidate formwork system vis-a-vis the base formwork system.

The design of a building, site constraints, available resources, the contractor's experience with different systems, and their availability, are some of the variables that affect the selection of a particular formwork. According to Horner and Thomson (1981), the important parameters affecting the choice of formwork can be classified under four headings: (a) the type of building element to be formed, (b) the type of sheathing material, (c) safety and serviceability of the structural frame, and (d) economics.

Typically, a senior member of the contractor's organization makes the decision of selection of the formwork system and the decision to select a particular system is heavily based on his individual experience. This experience may limit the selection of the system to the one that is not the optimum. Hanna and Willenbrock (1992) developed an expert system by systematically capturing the expertise of people involved in all the phases to assist the formwork selector/designer in making that decision. The different phases in the life of the formwork start right from the design phase through erection and concrete placement, to its removal. Guo and Tsai (1999) combined the Analytic Hierarchy Process (AHP), Fuzzy Multiple Attribute Decision Making (FMADM) and Technique Order Preference by its Similarity to an Ideal Solution (TOPSIS) method in order to evaluate and rank alternative formwork systems using 12 selected evaluation indices. Hanna (1999) elaborately discussed the factors affecting the selection of the formwork system. He characterized the factors into 4 classes and 18 evaluation factors. The four classes are: building design, job specification, local conditions, and supporting organisation. These are discussed briefly in the following sections.

1.4.1 Building Design

The building design proposed for a project can have a major influence on the selection of the formwork system. The type of slab — for example a flat slab system, a beam and slab system, or a waffle slab system — can influence the choice of the formwork system. The floor to floor height of the building can also have an impact on the decision to select a formwork system. As an example, a building with a floor to floor height less than about 4.5 m may be conveniently and economically executed by a prop-based formwork system while for a greater floor to floor height, the prop system may not work. One may like to adopt a shoring tower based formwork system for greater floor to floor height (say, more than 4.5 m). The shape of the building is an important parameter in the selection. For example, a building with an irregular shape may require a different formwork system compared to a building which is of uniform shape.

1.4.2 Job Specification

The requirement of concrete finish, for example, as cast concrete surface finish, exposed concrete finish, architectural concrete finish and so on, imposed by the specification of the job is also a major factor in the selection of formwork. Some formwork systems are capable of providing very good finish requiring no or minimal touch-up work after the concrete is exposed. Thus, under conditions requiring good concrete surface finish, such formwork systems would be favored. Further, the speed of construction, the rate of placement of concrete, and the sequence of construction also play an important role in the decision to select a formwork system. Some contractors have reported the completion of one to two floors per week using flying forms and tunnel formwork. Thus, the speed of construction required in a project can decide in favor or against a given formwork system.

1.4.3 Local Conditions

The selection of the formwork system is dependent on numerous local conditions such as: the formwork practice prevalent in the area, the local weather conditions, and other site characteristics.

The formwork system selection is dependent on the site condition. For example, an open site would call for a different formwork system than a site which is located in a congested area where the space for site operation is restricted.

1.4.4 Supporting Organization

The selection of formwork is also dependent on the supports that the organization can provide. The supports could be in the form of providing sufficient capital to invest in a given formwork system, making available the required hoisting equipment sufficient to carry a given formwork system and so on. Absence of the requisite crane in a project can straightaway rule out going in for a flying formwork system. The availability of supporting yard facility — either local or centralized — and the availability of experienced personnel for troubleshooting are also important in arriving at decisions pertaining to formwork selection.

1.5 CLASSIFICATION (TYPES) OF FORMWORK

In formwork literature we do not find any mention of a formwork classification system. Thus the present section draws heavily from the nomenclature used in the industry and that used by some of the texts available on formwork.

When we talk of traditional classification, the three types of formworks that come to mind are: *conventional* system, *proprietary* or *patented* system, and the *modular* system. This has been referred as the traditional system of classifying formwork systems in this text.

Hanna, in his book on formwork, has referred to the horizontal and vertical formwork systems. He has further subdivided the horizontal formwork system into hand set, crane set, and special horizontal formwork system. The vertical formwork system has been further classified into the crane dependent system and the crane independent system. We term this classifications as the classification based on Hanna (1999).

In India, often the formwork items are presented in a specific form in the bill of quantities. These items are mentioned in the Delhi Schedule of Rates (DSR 2007).We have taken a clue from that, and accordingly, the third system of classification is referred to as "classification based on Formwork items as used in DSR 2007". The three schemes of classification of formwork are shown schematically in Fig. 1.2 and discussed in the following sections.

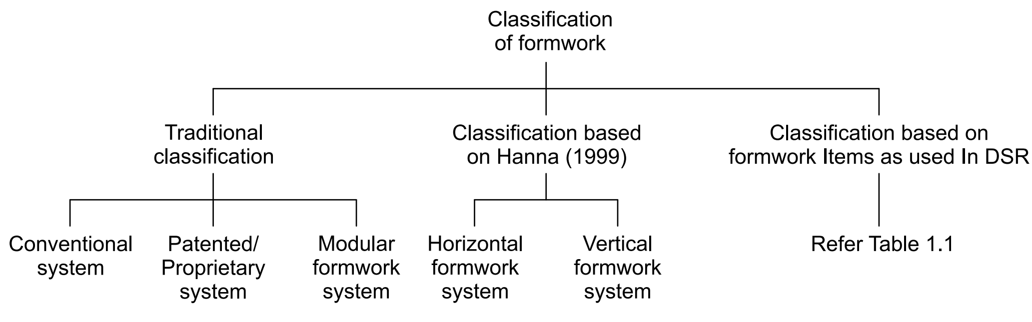


Figure 1.2 Different Systems of Formwork Classification

1.5.1 Traditional Classification

Broadly, formwork systems can be divided into three groups:

- Conventional system of formwork
- Proprietary/Patented system of formwork
- Modular system of formwork

1.5.1.1 Conventional system of formwork

In the conventional system, timber and plywood are used predominantly. The various formwork components are connected with the help of nails. This makes the conventional system of formwork quite flexible. However, some of the disadvantages are: their limited reuse value and their relatively higher cost. The conventional system of formwork can be fabricated, installed, and removed only by skilled workmen such as professional carpenters. Since the skill of carpenters and the quality of formwork material vary from site to site, the quality of concrete structures obtained is not consistent. Further, there is considerable risk involved regarding the quality of materials and the consequent unpredictable behavior. In large projects, it is difficult to ensure an efficient utilisation of the timber available on site. All these factors adversely affect the cost of formwork.

Some of the conventional systems of formwork for beam and slab, and column are shown in Figs. 1.3 and 1.4 respectively. It can be seen that timber and plywood have been used in the column, slab, and beam forming. Eucalyptus shores have been used for supporting the slab and beam formwork while they have also been used for aligning the column formwork.

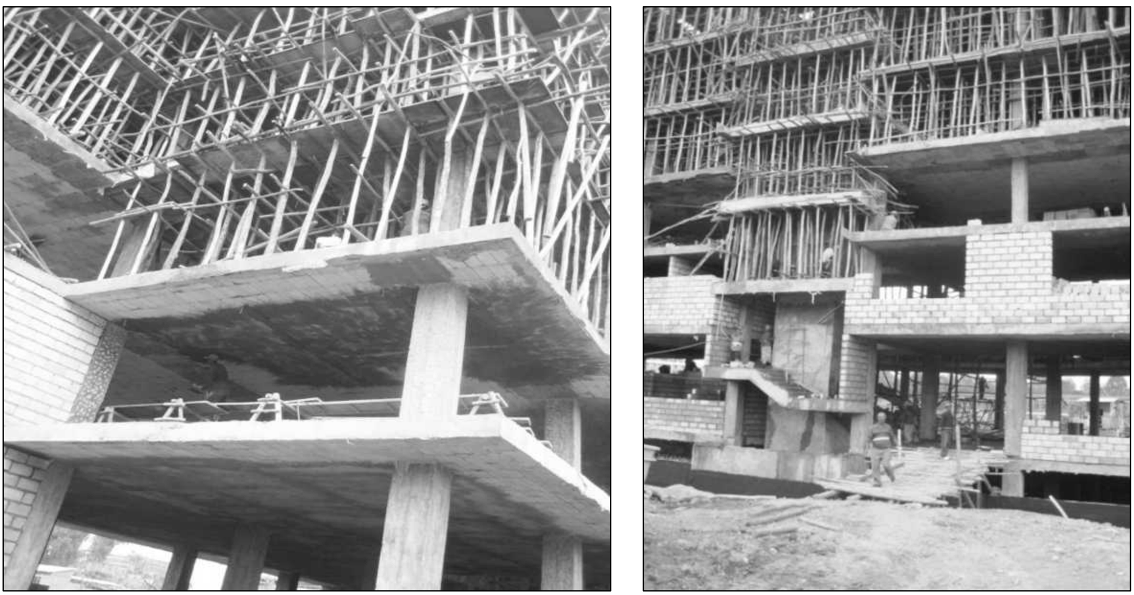


Figure 1.3 Conventional Slab and Beam Formwork

Figure 1.4 Conventional Column Formwork

1.5.1.2 Proprietary/patented system of formwork

Most of the disadvantages observed in conventional systems can be overcome by adopting good proprietary systems, which make use of standard factory made components. There is very little making of formwork involved at the site. These systems have features and accessories developed over many years of experience resulting in the saving of labor and higher productivity. With these systems, there is no labor spent in their making. With uniform quality standard products, it is possible to achieve uniformly good workmanship, improved quality of concrete surface and high reuse value. These can be assembled and dismantled with unskilled/semi-skilled labor. They also make planning and control easy and effective which also contributes to the efficiency of work at the site.

It may be noted that the proprietary/patented systems may consist of modular or non modular formwork components depending on the manufacturer. However, since modular formwork systems have some distinct features, they are discussed in a separate section.

Figure 1.5 shows the application of the proprietary formwork system for various RCC elements such as wall, column, beam and slab. The formworks shown here use plywood as the sheathing material. H-16 beams (described later in detail) have been used as the supporting material. These members are made out of timber and plywood in a factory. In case of slab formwork, they have been used both as the primary and the secondary members. The entire formwork assembly for beam and slab rests on collapsible tubular props. The details of the formwork system have been explained in the subsequent chapters.

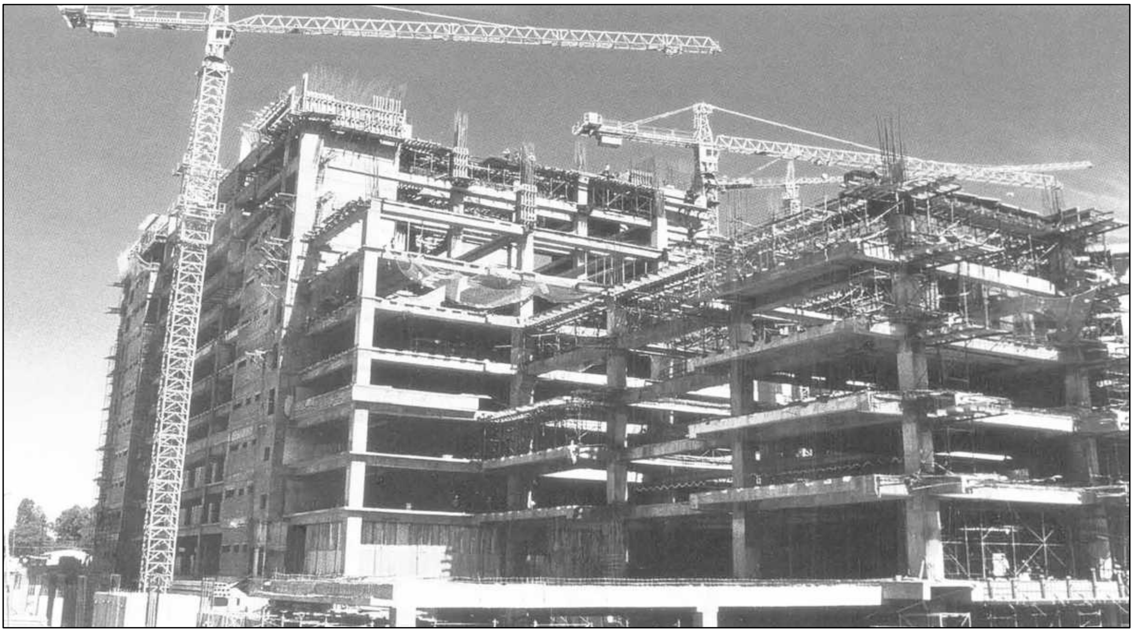


Figure 1.5 Proprietary Formwork for Casting Varied Concrete Elements Such As Wall, Column, and Slab and Beam Formwork (Courtesy L&T Formwork)

There is a growing trend to use pre-fabricated forms in concrete construction these days. This is obviously to save material and labor costs which result through greater efficiency attained during the mass production process employed for manufacturing the pre-fabricated unit. The technical details of the pre-fabricated forms are provided by the manufacturers to the users. In addition, the manufacturer may also provide the formwork layouts and the scheme if the user so desires. The pre-fabricated forms are available either on purchase basis or on rental basis. Pre-fabricated forms are generally of two types:

(a) Ready-made for general purpose; and
(b) Tailor-made for special purpose.

The ready-made forms can be adopted to build formworks for various sizes and shapes and thus can be considered as general purpose formworks. These forms may include modular panel systems and accessories. The tailor-made forms are fabricated to order and thus can be considered as special purpose formworks. These may include tunnel forms, dam forms, bridge girder formwork, etc. It may so happen that some ready-made form components may be used for the fabrication of tailor-made forms.

1.5.1.3 Modular formwork system

A lot of effort has been made over the past few years to bring automation into the formwork construction process. A large number of modular formworks have been developed which bring in standardization and make the entire formwork operation very easy. The formwork modules are manufactured in a factory set up, and delivered to the site in a pre-fabricated form. They can be assembled very quickly at the project locations. In contrast to the conventional system of formwork, the advantages offered by the modular formwork system are given below:

1. Since the modular formwork system uses standardized modules, and their installation process is also simple, the work can be accomplished even by less skilled workers.
2. There is a considerable reduction in erection time at the site.
3. A large number of repetitions (reuse) of modular formwork are possible which result in the cost of formwork being considerably on a lower side.
4. The safety of workers and materials is ensured, as the modular formworks have high strength of the form.
5. The quality of the concrete surface obtained is extremely good in a modular formwork which reduces the need for further finishing work such as plastering, after removal of the form.
6. The modular formwork systems automate the formwork operation and improve the productivity as well as the cost-effectiveness in a construction project.
7. In order to ensure the successful use of a modular formwork system, proper planning in the architectural design stage as well as in the construction phase of a project is necessary.
8. The factors such as the form reuse scheme, the allocation of modular form sets, cranes, workers, etc., and the construction sequence need to be carefully planned out to get the best out of the modular formwork.

The comparison of various features of the three systems (the conventional, the proprietary, and the modular formwork system) is given in Box 1.2. Horner and Thomson (1981) studied the relative costs of the site made and the proprietary formwork for a typical wall, column, and beam and slab soffit. The relative costs were arrived at by producing two sets of designs each for the site made and the proprietary formwork. For costing purposes however, they chose to take the labor and material cost only for the two alternatives. The results of the study showed that if one or more uses per week are anticipated, it is cheaper to hire than to make. The breakeven point is about one use every four weeks. They further found that it is uneconomic to buy proprietary forms unless some 20 uses are anticipated.

1.5.2 Classification of Formwork (Based on Hanna 1999)

Hanna (1999) classifies the formwork systems into essentially two systems: Horizontal formwork system and Vertical formwork system. The further subdivision of the two systems is given in Fig. 1.6, and a brief discussion on the systems follow:

1.5.2.1 Horizontal formwork system

Under horizontal formwork system, we have essentially three subsystems: Hand set, Crane set, and some special horizontal formwork system.

BOX 1.2: Comparison of various features for the conventional, proprietary, and modular formwork system

Features	Conventional system of formwork	Proprietary/patented system of formwork	Modular formwork system
Flexibility	It has high flexibility.	It has reasonably good flexibility.	The system brings in standardization and makes the entire formwork operation very easy.
Reuse values	Limited reuse value. The connection between various timber members by nailing, renders the material unusable after a few uses.	The system has high reuse value.	A large number of repetitions (reuse) of modular formwork are possible.
Making involved at site	There is large making of formwork involved at the site.	There is very little making of formwork involved at the site.	Very less making at site, only assembly of standard components required.
Expertise requirement for making, fixing, and removing	Considerable expert carpentry effort is required to be put in for making, fixing and removing the form.	These can be assembled and dismantled with unskilled/semiskilled labor.	The work can be accomplished even by the less skilled worker.
Effort required for assembly at site	Considerable effort is required for assembly at the site.	These systems have features and accessories developed over many years of experience that result in saving of labor and higher productivity.	They can be assembled very quickly at project locations. There is considerable reduction in erection time at the site. The installation process is also simple.
Quality of concrete surface	The quality of formwork and productivity varies from site to site depending on the skill available at the site.	Uniform quality can be expected.	The quality of concrete surface obtained is extremely good.
Productivity	Low	High	Higher
Cost	Cost of formwork is the highest for large projects.	Cost of formwork much lower compared to conventional formwork for large projects.	Cost of formwork is the lowest.
Planning requirement	It is difficult to plan. The indiscriminate cutting of timber is difficult to be controlled in large projects.	They also make preplanning easy and effective and this also contributes to the efficiency of work at the site.	The standardization offers ease in planning.

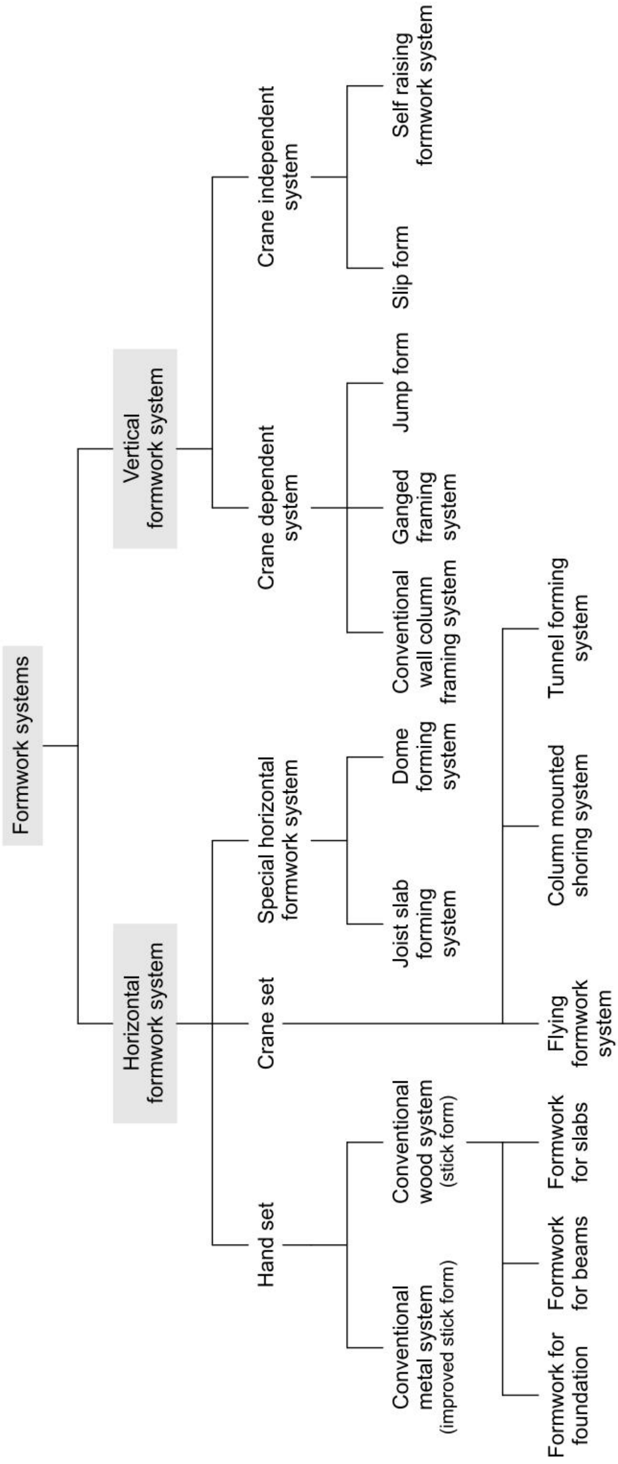


Figure 1.6 Formwork Classification Based on Hanna (1999)

In Hand set formwork, the commonly used formwork systems are: (a) conventional wood system (also known as *stick form*) and (b) conventional metal system (also known as *improved stick form*). The conventional wood system is available for laying the foundation, beams and slabs, and columns etc.

In Crane set formwork systems, we have the flying formwork system, column mounted shoring system, and tunnel forming system. These are discussed at greater length in Chapter 10 of this book.

In special horizontal formwork systems, we have the joist slab forming system, and dome forming system. These are discussed at greater length in Chapter 8 of this book.

1.5.2.2 Vertical formwork system

Under vertical formwork system, we essentially have two subsystems: crane dependent system and crane independent system.

In crane dependent system, we have the conventional wall column framing system, Ganged framing system, and Jump form.

In crane independent system, the commonly used formwork systems are: (a) slipform and (b) self raising formwork system.

1.5.3 Classification of Formwork Based on DSR Formwork Items

In this classification (see Table 1.1), the various formwork items are classified under the following categories.

Table 1.1 Formwork Classification Based on DSR

Sl. no.	Formwork item	DSR item reference
1.	Foundations, footings, bases of columns, etc.	5.9.1
2.	Walls (any thickness) including attached pilasters, buttresses, plinth of string course	5.9.2
3.	Suspended floors, roofs, landings, balconies, and access platforms	5.9.3
4.	Shelves	5.9.4
5.	Lintels, beams, plinth beams, girder, bressumers, and cantilevers	5.9.5
6.	Columns, pillars, piers, abutments, posts, and struts	5.9.6
7.	Stairs (excluding landings) except spiral staircases	5.9.7
8.	Spiral staircases (including landings)	5.9.8
9.	Arches, domes, vaults upto 6 m span and exceeding 6 m span	5.9.9
10.	Chimneys and Shafts	5.9.11
11.	Well Steining	5.9.12
12.	Vertical and horizontal fins individually or forming box louvers band, facias, and eaves boards	5.9.13
13.	Centering in circular work	5.9.14
14.	Small lintels	5.9.15
15.	Cornices and mouldings	5.9.17
16.	Weather shade, chajjas, corbels etc.	5.9.19

In addition to the above methods of classification, formworks can also be classified according to the size, materials of construction, nature of operation, and sometimes according to the brand name of the product. According to size, they are classified into small and large formwork. While the small formwork can be hand set, the large formwork requires crane assistance. In a small formwork, small form panels — light in weight and usually made up of timber and plywood or aluminum — are used, while in case of a large formwork, the form panels used are large in size and heavy. Large area wall formwork and ganged formwork are examples of large formwork. According to the materials used for form construction, formworks are referred to as timber formwork, steel formwork, aluminum formwork, composite formwork etc. According to the operation involved, the formworks can be classified into manually operated or mechanically operated formwork. Timber and aluminum formworks are usually manually operated, while the heavy steel formworks are normally mechanically operated. Some examples of mechanically operated formworks are: crane assisted climbing formwork or self raising formwork. It is also common to refer to the formwork with the names of the manufacturers. Thus, in this classification, we have L&T formwork, Doka formwork, PERI formwork, Mivan formwork, and so on. Some of the formwork solutions offered by these manufacturers have also been discussed in different chapters.

In this book we have not tried to follow any classification in totality. Instead, we have first covered the frequently required formwork solutions in the context of building works such as foundation formwork, wall formwork, column formwork, and slab and beam formwork. Subsequently, we have covered special applications followed by bridge formwork, and the special formworks such as flying formwork, slipform and so on. The organization of the book is briefly presented in the next section.

1.6 ORGANIZATION OF THE BOOK

The book is organized in 18 chapters. In the beginning of each chapter, the major contents are outlined. Thereafter in very well defined sections and subsections, the objectives of the chapter are dealt with one after another.

In Chapter 1, the introductory aspect of formwork has been covered. The classifications of different formwork systems are discussed. The decision variables affecting the selection of a particular formwork system are also briefly discussed.

In Chapter 2, the issues related to formwork materials have been covered. The chapter contains details on various formwork materials such as form materials, support materials, coating and release agents, tie systems, form anchor, and form liner. The properties of some of the commonly used formwork materials have also been briefly mentioned in this chapter.

In Chapter 3, the various issues pertaining to different loads on the formwork are discussed. They are dead load, superimposed load, and environmental loads. The provisions of ACI method, CIRIA method, Indian Standard method, and DIN standards have been illustrated to compute the lateral pressure on the formwork. Illustrative examples are given to reinforce the concept of computation of lateral pressure using different codes of practices. The permissible stresses for the commonly used materials are introduced. The formulas for bending moment, shear forces, and deflection for commonly encountered situations in formwork design are explained briefly.

In Chapter 4, the various aspects of formwork for foundations are discussed. The conventional foundation formwork and the proprietary formwork are introduced. Some design issues pertaining to foundation formwork are discussed.

In Chapter 5, we discuss the conventional wall formwork and different varieties of the proprietary wall formwork such as the climbing scaffold, the traveling climbing scaffold and the automatic climbing scaffold. Some real life illustrations of wall formwork in use have also been provided. We discuss the different design steps needed to perform the wall formwork design. Some design aids for getting assistance in the wall formwork design are also presented. Towards the end we illustrate a few wall formwork design examples.

In Chapter 6, we discuss the conventional column formwork and different varieties of the proprietary column formwork such as the Doka formwork and the PERI formwork. Some real life illustrations of the column formwork in use have also been provided. We discuss the different design steps needed to perform column formwork design. Some design aids for getting assistance in the column formwork design are also presented. Towards the end we illustrate a few column formwork design examples.

In Chapter 7, we discuss the conventional slab formwork and different varieties of the proprietary slab formwork such as the Doka formwork, the PERI formwork, and the Mivan formwork. Some real life illustrations of slab formwork in use have also been provided. We discuss the different design steps needed to perform slab formwork design. Some design aids for getting assistance in the slab formwork design are also presented. Towards the end we illustrate few slab formwork design examples.

In Chapter 8, formworks for some special structure such as domes, shells, folded plates, tunnels, cooling towers, and overhead water tanks are briefly discussed. The real life example of the formwork arrangement for such structures is illustrated.

In Chapter 9, formwork for different elements of bridges is discussed. Formworks for cutting edge, kerb, well steining, pier and pier caps are discussed. In addition, the deck slab formwork, girder formwork, and the formwork for crash barrier are also discussed.

In Chapter 10, some commonly used flying formwork systems are discussed. They are the table form and the tunnel form. The step by step working procedure with these formwork systems is explained. The advantages and limitations of these systems are also briefly discussed.

In Chapter 11, the slipform is introduced. The various types of slipforms such as straight, tapered, and inclined slipform are explained with illustration. The various components of slipforming operation and their functions are explained. The assembly and dismantling practices of slipform are briefly discussed. The design issues in slipform are briefly covered; besides, the productivity values obtained in a leading construction organization are illustrated for getting assistance in planning and costing of the slipform operation. Towards the end we discuss some safety issues in slipform and cases in slipform.

Chapter 12 starts with an introduction to shoring and its utility followed by the classification of the various shoring towers. Further, different families of shoring towers are compared. The design aspects and work inputs of shoring towers are discussed in brief. The reuse of shoring towers and work inputs for high shoring towers are also discussed. Towards the end of the chapter, the recommendations for practitioners have also been provided for the use of shoring towers.

Chapter 13 starts with an introduction to the scaffold system. The design guidelines and checklist for the scaffolds are discussed in this chapter. The possible causes for collapse of the scaffold system are also discussed. Finally the recommendations for the scaffold system and the checklist for safe implementation have also been provided for the practitioners.

In Chapter 14, the advantages of pre-casting over *cast-in-situ* concrete construction are discussed. The different types of moulds used for pre-casting concrete elements are discussed. The various stages and steps involved in pre-casting are briefly explained. The real life cases in pre-casting are explained with the help of photographs.

In Chapter 15, the pre-award formwork management issues are addressed. These include: understanding the requirement of the customer, study of drawings, layouts of the structure and estimation of cycle time of formwork activities. The issues involved in the selection of formwork system have been discussed and some guidelines for the selection of formwork system have been given. Formwork economy considerations in planning and design stage of a project, computations of formwork material requirement, cost estimation of formwork are also discussed in detail in the chapter.

In Chapter 16, the post-award formwork management issues are addressed. This is done in two stages: one, immediately on award of the contract, and two, during detailed planning just before execution. In the latter, emphasis has been placed on: preparation and finalization of the formwork scheme, preparation of mobilization schedule of formwork materials, effective usage of systems/materials and the proper implementation, upkeep/ maintenance of formwork materials, accountability for materials, training, monitoring of formwork cost, and preparation of demobilization schedule of formwork materials. Towards the end of the chapter, key positions and their responsibilities in formwork management are also taken up.

Chapter 17 deals with formwork failure. The causes of formwork failure such as triggering, enabling, and procedural causes are discussed. Some case studies in formwork failure are also discussed. Towards the end, some checklists relevant for formwork related activities are provided for safe working.

In Chapter 18, a simplified method is presented to compute the loads on slabs, shores, and reshores at the time of construction. The determination of the expected strength of concrete slab is also explained using a simplified approach. Knowing the loads on slabs and the strength of slab, a formwork engineer can ensure the safety of slab at the time of construction by adjusting various parameters such as construction technique, cement type, curing temperature etc.

Glossaries of the terms related to formwork are briefly defined towards the end of the book. In the construction industry, there are widely varying perceptions and understanding of different formwork related terms. It is thought prudent not to add to the confusion by suggesting definitions specific to the text and thus the definitions are taken from different Indian Standards and are produced in this chapter. The IS: 3696 (Part 1)–1987, IS: 4014 (Part 1)–1967, and IS: 6461 (Part 5)–1972 have specially been useful in compiling the various formwork related terms. Wherever it is felt appropriate, the definitions have been explained a bit. The reader may find the compilation quite useful.

REVIEW QUESTIONS

Q1. True or False

(a) Formwork is a temporary (T/F)/ permanent (T/F) structure desired to safely support the concrete until it reaches adequate strength to stand on its own.

(b) Examples of temporary formwork are: concrete formwork construction, scaffolding, falsework/staging, cofferdams, underpinning, diaphragm wall, slurry wall, earth retaining structures, construction dewatering.

(c) The three major objectives of formwork are quality, safety, and economy.

(d) Honeycombs and bug-holes constitute evidence of good quality formwork.

(e) Total formwork cost is the sum of costs of form materials, labor, equipment, consumables, and others.

(f) Bug-holes and honeycombs are the result of poor formwork.

(g) The economy in a formwork can be affected right from the planning and design stage to the construction stage of the project.

Q2. Match the following

1. Sheathing (Sheeting)	(a) Gaining access to higher levels of permanent structure during construction
2. Form (Shutter)	(b) Part of form which is in contact with concrete
3. Falsework	(c) Frame for casting precast concrete units
4. Centering	(d) Part of formwork consisting of sheeting to support concrete till it becomes self supporting
5. Mould	(e) To support work in process of construction and is composed of shores, formwork for beams etc.
6. Scaffolding	(f) Specialized formwork used in the construction of arches, shells space structures

Q3. Match the following. The safety statistics shown below are on an average reported in literature.

1. Falls	(a) 60%
2. Struck	(b) 5.83%
3. Formwork and shoring collapse	(c) 21.2%
4. Premature removal of shores	(d) 18%
5. Faulty material	(e) 8%

Q4. Match the following in the context of parameters affecting the choice of formwork.

1. Hanna (1999)	(a) Type of building element formed by the sheathing material, safety, and economics.
2. Horner and Thomson (1981)	(b) Supporting organization, local condition, job specification, building design.

Q5. Match the following.

1. Traditional classification of formwork	(a) Formwork item based classification
2. Hanna (1999) classification of formwork	(b) Conventional, patented, modular formwork system
3. DSR formwork classification	(c) Horizontal and vertical formwork

Q6. Answer in brief the following questions.

(a) Why is formwork different from other temporary structures such as diaphragm wall and cofferdam?

(b) Distinguish between (a) formwork, (b) scaffolding, and (c) centering.

(c) What safety features can you think of incorporating in a formwork system to avoid accidents?

(d) Discuss the consequences of poor formwork in reinforced concrete construction.

Q7. Identify the requirements of formwork. Take feedback from the experts and decide the relative importance of each of these parameters.

Chapter

2 Formwork Materials

Contents: Introduction; Timber; Plywood; Steel; Aluminum Form; Plastic Forms; Other Materials; Form Coatings and Mould Linings; Form Anchors; Tie System; Spreaders, Spacers; Form Lining Materials

2.1 INTRODUCTION

Timber and plywood have traditionally been used as form materials. A wide range of form materials such as steel, aluminum, fiberglass reinforced plastic etc., have been used in the recent past. The construction industry is witnessing newer forms of materials every now and then.

Formwork was formerly built in one place, used once, and wrecked. Because of high labor costs prevailing currently in India and abroad, the trend today is towards increasing prefabrication, assembly in large units, and erection by mechanical means such as *flying* the forms into the place by crane, and continuing reuse of the forms. In 1908, the use of wood versus steel formwork was debated at the ACI convention. Also, the advantages of modular panel forming with its own connecting hardware, and good for extensive reuse were realized. By 1910 steel forms for paving were being produced commercially and used in the field in the USA. Today modular panel forming is the norm.

The selection of material to be used for formwork and shoring shall take into account its strength, rigidity, durability, workability, finished quality of concreted surface, effect on the fresh concrete placed, and the economy.

In this chapter, we will discuss the various form materials and their properties, support materials, tie systems, form anchor, form liner, and coating and release agents.

2.2 TIMBER

Practically all formwork jobs require some timber. Frequently the choice of timber species depends on the local availability and cost. The timber used should be reasonably seasoned to avoid warping. They should hold nails well. It must be necessary to go for hard wood in cases of heavy structures imposing large loads on the formwork. The timber for formwork should be softwood of partially seasoned stock to avoid swelling or warping as per IS: 883–1994.

2.2.1 Characteristics of Good Quality Timber Formwork

The characteristics of good quality timber formwork are given in Box 2.1.

BOX 2.1: Characteristics of good quality timber formwork amenable to reuse

1. It should be easy to work with by hand or machine and should not split when nailed.
2. It should be hard enough to withstand damage on the contact surface under normal conditions of erecting and stripping of the formwork, fixing steel reinforcement bars and placing concrete.
3. It should be light enough for the workers to handle and carry to their respective work areas.
4. It should be stiff so as to avoid undue deflection when loaded, and strong enough to safely carry considerable loads and pressures that may be applied during concreting.
5. It should be reasonably stable and not unduly liable to cast or warp when exposed to sun and rain or when unevenly wetted during rainy days or the monsoon seasons.
6. It should have the correct amount of moisture so that it will not warp and swell after concrete is placed.
7. It should not have excessive formation of hemicelluloses (wood sugars) on the first use after exposure to sunlight, as it will retard the curing of concrete.

2.2.2 Commonly Used Timber Sections for Formwork and Their Properties

The commonly used timber sections for formwork purposes are given in Table 2.1 and the properties of some commonly used sizes of sawn timber are given in Table 2.2.

Table 2.1 Commonly Available Sizes of Timber for Formwork (Based on Recommendations of IRC: 87–1984)

Sl. No.	Description of formwork element	Size (mm)
1.	Sheathing	25 to 50 mm
2.	Beam and column sides	25 to 50 mm
3.	Beam bottoms	50 mm
4.	Joists	50 mm × 100 mm to 70 mm × 200 mm
5.	Ledgers	50 mm × 100 mm to 75 mm × 200 mm
6.	Posts	75 mm × 100 mm to 150 mm × 200 mm
7.	Column yokes	50 mm × 100 mm to 100 mm × 200 mm
8.	Struts and walings	50 mm × 100 mm to 150 mm × 200 mm

Table 2.2 Properties of Sawn Timber

$b \times d$ (inches)	b (mm)	d (mm)	Area (mm^2)	Z_{xx} (mm^3)	I_{xx} (mm^4)	r_{min} (mm)
2 × 2	50.8	50.8	2,580.64	21,849.42	5,54,975.23	14.66
2 × 2.5	50.8	63.5	3,225.80	34,139.72	10,83,936.00	14.66
2 × 3	50.8	76.2	3,870.96	49,161.19	18,73,041.42	14.66
2 × 4	50.8	101.6	5,161.28	87,397.67	44,39,801.87	14.66

$b \times d$ (inches)	b (mm)	d (mm)	Area (mm^2)	Z_{xx} (mm^3)	I_{xx} (mm^4)	r_{min} (mm)
2×4.5	50.8	114.3	5,806.44	1,10,612.68	63,21,514.78	14.66
2×5	50.8	127	6,451.60	1,36,558.87	86,71,488.03	14.66
2×6	50.8	152.4	7,741.92	1,96,644.77	1,49,84,331.32	14.66
3×3	76.2	76.2	5,806.44	73,741.79	28,09,562.12	22.00
3×4	76.2	101.6	7,741.92	1,31,096.51	66,59,702.81	22.00
3×5	76.2	127	9,677.40	2,04,838.30	1,30,07,232.05	22.00
3×6	76.2	152.4	11,612.88	2,94,967.15	2,24,76,496.98	22.00
4×4	101.6	101.6	10,322.56	1,74,795.35	88,79,603.75	29.33
4×6	101.6	152.4	15,483.84	3,93,289.54	2,99,68,662.64	29.33
4×8	101.6	203.2	20,645.12	6,99,181.40	7,10,36,829.97	29.33

2.2.3 Specifications of Timber for Formwork Applications

The specification for timber to be used in a formwork application must conform to IS: 3629–1986 which specifies the requirement for structural timber. The pertinent structural requirements of timber to be used for formwork are given in Table 2.3.

Table 2.3 Summary of Specifications of Timber for Formwork Application

1	Modulus of elasticity	The modulus of elasticity of clear specimens of timber tested in bending shall not be less than 5,500N/mm^2.
2	Moisture content	The timber shall be seasoned so as to have moisture content ranging from 12 to 20% depending on the climatic zone of the place of construction.
3	Defects	The timber shall not have prohibited defects.
4	Durability and treatability	The timber shall be suitable in respect of the durability and treatability.
5	Design	The timber members shall be designed in accordance with prescribed stipulations for arriving at permissible stresses.

2.2.4 Permissible Stresses for Timber

The codes provide allowable unit stresses for different species of timber. The permissible stresses corresponding to short time loadings can also be found in these codes. Moisture present in the timber tends to reduce the allowable unit stress. IS: 883–1994 classifies timber for constructional purposes in three groups based on their strength properties, namely, modulus of elasticity (E) and extreme fiber stress in bending and tension (f_b). Table 2.4 provides E and f_b values for the three groups of timber mentioned.

Table 2.4 Characteristics of Different Timber Groups

Group	E (modulus of elasticity in bending)	f_b (permissible bending stress on the extreme fiber)
Group A	above 12.6×10^3 N/mm^2	above 18.0 N/mm^2
Group B	above 9.8×10^3 N/mm^{2}and up to 12.6×10^3 N/mm^2	above 12.0 N/mm^2 and up to 18.0 N/mm^2
Group C	above 5.6×10^3 N/mm^2 and up to 9.8×10^3 N/mm^2	above 8.5 N/mm^2 and up to 12.0 N/mm^2

The allowable stresses in timber are given in Table 2.5.

Table 2.5 Allowable Stresses in Timber

Classification	Trading Name of Timber	Basic Stress in N/mm^2					
		Bending	Tension along grain	Compression		Shear parallel to grain	Modulus of elasticity N/mm^2
				Parallel to grain	Perpendicular to grain		
Group A	Kongo/Sal	15.2	15.2	10.6	4.6	1.2	1.26
Group B	Casuarina/ Gurjan/ Benteak/ Sal (MP)/ Teak/ Kindal/ Laurel/ Irul	10.2	10.2	6.3	1.8	0.9	1.12
Group C	Poon/ Deodar/ Mango/ Chir	7.0	7.0	5.6	1.7	0.6	1.77

The permissible stresses (N/mm^2) For Grade I timber is given in Table 2.6.

Table 2.6 The Permissible Stresses (N/mm^2) For Grade I Timber

Sl. No.	Types of stresses	Group A	Group B	Group C
1	Bending and tension along grain	18.0	12.0	8.5
2	Shear–horizontal	1.05	0.64	0.49
3	Shear—along grain	1.5	0.91	0.70
4	Compression parallel to grain	11.7	7.8	4.9
5	Compression perpendicular to grain	4.0	2.5	1.1
6	Modulus of elasticity	12.6×10^3	9.8×10^3	5.6×10^3

The allowable axial compressive stress in N/mm^2 parallel to the grain of timber for different l/r and different groups of timber are given in Table 2.7.

Table 2.7 Allowable Axial Compressive Stress in N/mm^2 Parallel to Grain of Timber

l/r	Group A	Group B	Group C
0	10.6	6.3	5.6
5	10.6	6.3	5.6
10	10.6	6.3	5.6
15	10.1	6.2	5.4
20	9.0	5.9	5.1
25	6.6	5.3	4.4
30	4.6	4.2	2.8
35	3.4	3.0	2.1
40	2.6	2.3	1.6
45	2.1	1.8	1.3
50	1.7	1.5	1.0

2.2.5 Modification Factors

The modification factors to allow for change in the slope of grain (represented by K_1) are provided in Table 2.8. The K_1 values are provided for both beams and ties and posts or columns.

Table 2.8 Modification Factors to Allow for Change in Slope of Grain

Slope	K_1 Beams and Ties	K_1 Posts or Columns
1 in 10	0.80	0.74
1 in 12	0.90	0.82
1 in 14	0.98	0.87
1 in 15	1.00	1.00

The modification factors for change in the duration of loading are provided in Table 2.9. This modification factor is represented by K_2. Table 2.9 provides K_2 values for various duration of loadings such as continuous loading, two months duration, seven days loading, wind and earthquake loading and instantaneous or impact loading. It may be noted that with the decrease in loading duration, the K_2 values increase. It is maximum for instantaneous or impact loading conditions.

Table 2.9 Modification Factor for Change in Duration

Sl. No.	Duration of loading	K_2
1	Continuous	1.0
2	Two months	1.15
3	Seven days	1.25
4	Wind and earthquake	1.33
5	Instantaneous or impact	2.00

2.2.6 Application of Timber in Formwork

Timber is used for formwork sheathing, joists, beam bottoms, batten supporting floor boards, battens supporting vertical sheeting of vertical formwork, and for timber posts (vertical shores). The requirements of timber for these applications are given in the following sections.

2.2.6.1 Requirement for timber board as sheathing member

Timber board as sheathing member is shown in Fig. 2.1.

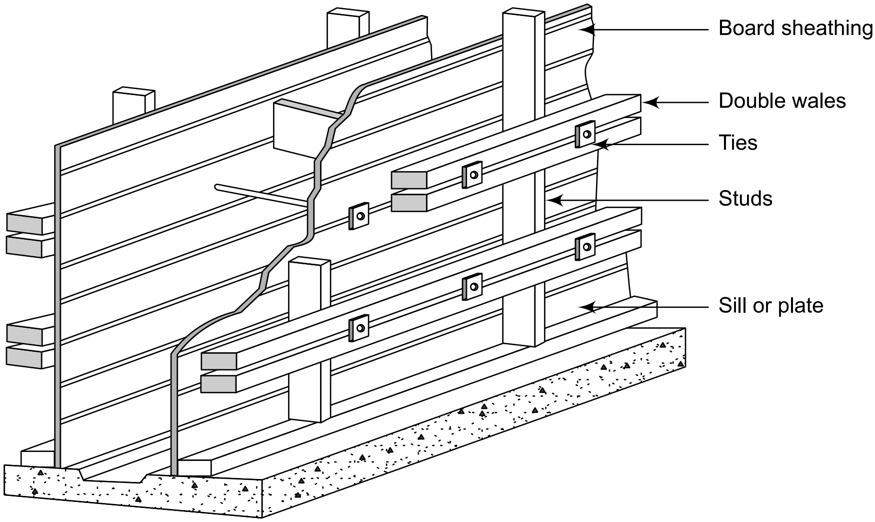


Figure 2.1 Typical Wall Form With Timber Sheathing.

The material for the sheathing should be selected in such a way that it does not deteriorate extensively on the absorption of moisture. The repeated cycles of wetting and drying may cause some timber boards to crack, spall, or become brittle. This may cause a considerable reduction in the strength. In order to avoid the deterioration and consequently the reduction in strength, the timber material should be selected and stored carefully.

The timber used for formwork sheathing is usually made plain on the side in contact with the concrete. The timber should be rigid to avoid local bending and denting. The recommended minimum thicknesses of timber board to act as a sheathing member are given below for different applications even though the design calculations might be showing a lower thickness. The block boards should not be used as sheathing member.

For wall, vertical sides of beams	:	25 mm
For floors of normal loads	:	30 mm
For floors where heavier constructional loads are possible	:	37 mm

Tongue-and-groove joint boards are preferred when sheathing boards are made into panels. This is done to ensure smooth and plain concrete surfaces. Use of mechanical saws under trained supervision can ensure that the boards closely fit with one another at the time of their installation. Failure to do so would lead to gaps at the installation which would require extra effort in the form of either filling with thin reapers or filling by oil-based putty. The gaps if not properly filled, will eventually lead to the leakage of slurry which would give rise to a number of problems.

2.2.6.2 Requirement for timber joists, beam bottoms, batten supporting floor boards, and battens supporting vertical sheeting of vertical formwork

Timber board as joists, batten, and stringers are shown in Figs. 2.2 and 2.3.

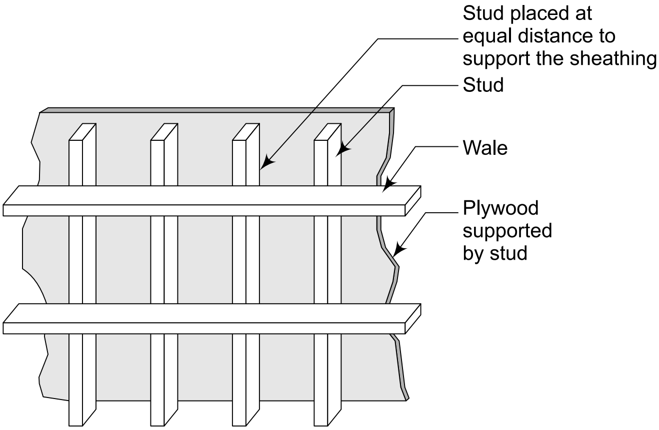


Figure 2.2 Parts of Typical Wall Formwork.

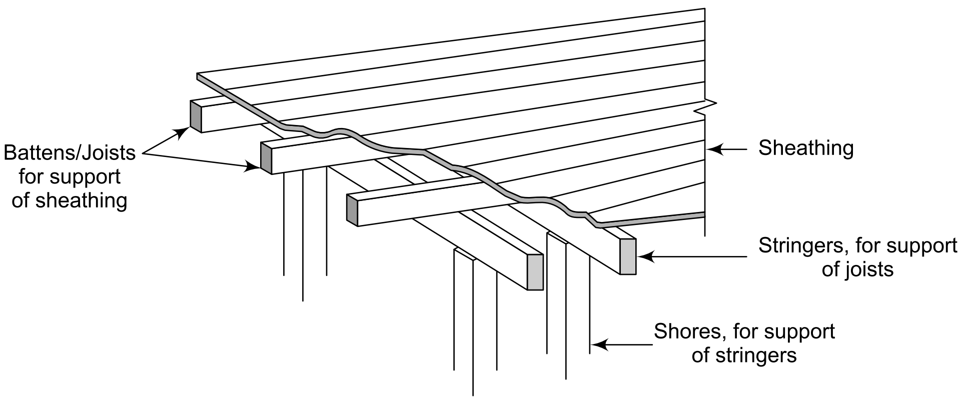


Figure 2.3 Parts of Typical Slab Formwork.

The depth (or thickness) of battens or joists will be based on strength and deflection requirements. Timber members used as joists, beam bottoms, battens supporting floor boards, battens supporting vertical sheeting of walls act primarily in bending. The maximum permissible unstiffened length is 50 times the width of the timber batten (joist) for such applications. For example, if the timber batten is 50 mm (approximately 2 inches) wide, the maximum spacing between stiffeners (horizontal or vertical) or cross members shall be $50 \times 50 = 2{,}500$ mm = 2.5 m. This could be even further reduced for practical considerations. As in the case of sheathing, the use of cut-lengths of block boards as battens should be avoided as these have very poor shear stress transfer capability.

2.2.6.3 Requirement for timber posts (Vertical Shores)

Timber board as vertical posts is shown in Fig. 2.3. Timber posts should have sufficient thickness so that they do not buckle. The l/d ratio (where l = unsupported length of a member and d = width of the face of the member under consideration) is one of the important design parameters for timber shore design. The floor to floor height in a normal building is of the order of 3 to 4 m and the shores are provided with horizontal braces at a distance of about 1.80 m from the floor level. The 1.8 m distance is ensured, so that a crew member of average height can easily work under the braces. The end conditions of shores are normally neither restrained against rotation nor restrained against the lateral movement (except due to friction). The effective length of the shores is about 2 times the height between the braces (about 1.8 m as explained above). The l/d ratio even in case of long columns is recommended to be 50. Thus, in order to avoid buckling and to avoid the shores against the collapse, the minimum recommended cross section of timber shores/posts should be taken as 100 mm × 100 mm. The free standing shores (without horizontal braces) of very large heights should be avoided.

2.2.7 Factors Affecting the Reuse of Timber Formwork

Reuse of formwork (discussed separately elsewhere in the text) is an important consideration in formwork. There are several factors affecting the reuse of timber formwork. Ling and Leo (2000) identified five main factors. These are: (1) the materials used to fabricate the formwork, (2) workmen who work with the formwork, (3) design of the completed structure, (4) design, fabrication, and stripping of the formwork, and (5) site management issues. These are shown in Fig. 2.4. The sub factors (total 15) under the five factors are also shown in Fig. 2.4.

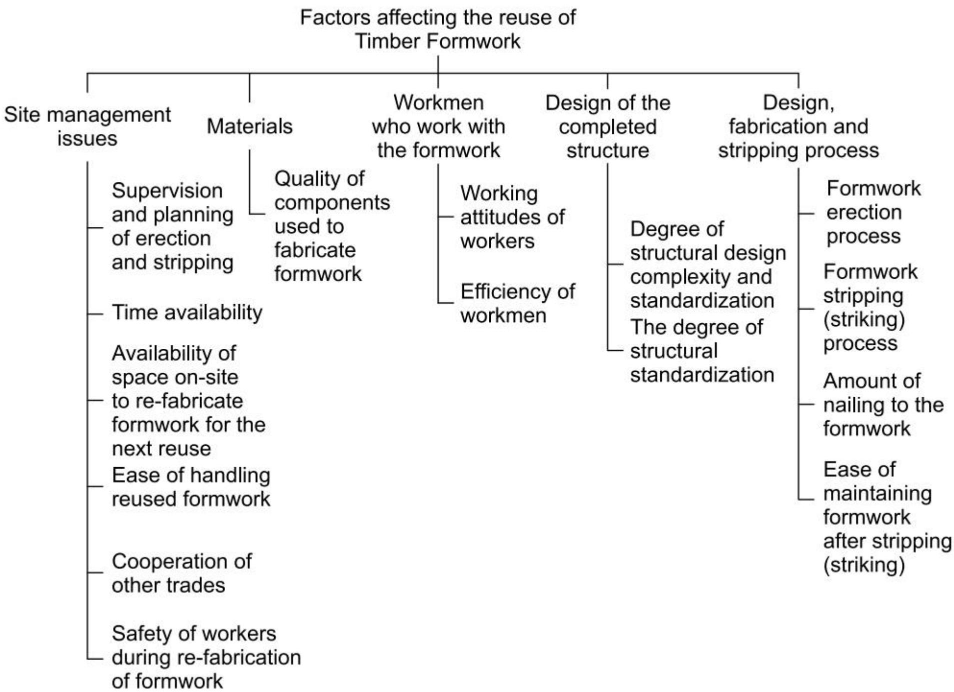


Figure 2.4 Factors Affecting the Reuse of Timber Formwork.

The sub factors: *working attitudes of workers* and *efficiency of workers* under the factor 'Workmen who work with the formwork' and the sub factor: *formwork stripping (striking) process* under the factor 'Design, fabrication and stripping process' were found to have significant impact on the reuse of timber formwork.

Although the study by Ling and Leo (2000) was based on a limited set of data, it clearly pointed that it is the workmen who determine the number of times a formwork can be reused. The workmen need to have the correct working attitude and be able to work efficiently, to ensure that the formwork has a high reuse. The formwork stripping process is also related to the working attitude and efficiency of the workmen.

Based on this finding, it may be possible to increase the numbers of reuse of timber formwork by motivating the workmen, instead of concentrating on the design or other factors. The study recommended that monetary reward be used to improve the work attitude and crew efficiency. Efficiency of erection and stripping of formwork is further enhanced if such rewards are directly associated with the performance of the workmen.

2.3 PLYWOOD

Plywood is used as the sheathing material directly in contact with the concrete. With proper care and treatment of form surfaces and panel edges, many reuses are possible with plywood. Plywood also involves relatively lesser cost of making of the formwork because of the ease of working with it.

The plywood is built up of odd number of layers with the grain of the adjacent layers perpendicular to each other. Generally the grain direction of the outer layer is parallel to the long dimension of the panel and it is stiffer when the face grain is parallel to the span.

Plywood is the most commonly used material for formwork because of the ease with which it can be cut and assembled on the site.

Plywood is available in several thicknesses and it can be made available with the choice of surface treatments to serve various needs and purposes. It provides extremely smooth surfaces to provide desirable concrete surfaces and one can be confident about its physical properties, as it is manufactured in factory like conditions unlike timber.

When using plywood, it is usual to frame up the materials into the largest size of the panel that can be handled by the available equipment on the site, or is convenient for handling by men.

For large smooth areas like walls or floors, or complicated shapes, timber frames with a plywood face are usually more economical than timber boards, especially when a high number of reuses are needed.

Care needs to be taken during the assembly, erection and casting so that their rather soft faces and edges are not damaged; particular care is necessary when striking and during storage.

Eco marked plywood with environment friendly features are also available these days.

IS: 4990–1993 gives specification of the plywood used for the purpose of form lining and sheathing. There are about 20 tests prescribed in IS: 1734–1983 for plywood testing.

The concrete surface obtained by plywood can virtually eliminate the need of plastering. Further, the large size of plywood brings in economy. All this would however depend on nails and screws being used judiciously. Excessive cutting of plywood and excessive use of nails and screws can spoil the plywood unduly.

Plywood can be used in a better way both in the hot and the cold climatic conditions as the external heat or cold does not penetrate through plywood unlike metal sheeting.

2.3.1 Permissible Stresses in Plywood

Based on the permissible deflection in plywood to be 1:270 (1/270th of the span between the bearers), the permissible stresses have been worked out in Table 2.10.

Table 2.10 The Permissible Stresses in Plywood for a Specified Deflection of 1:270

Center Distance of Bearers (Span) mm	The maximum load permitted (in kg/m^2) is for the face grain of the plywood perpendicular to the length of the bearers.		The maximum permissible load (in kg/m^2) is for the face grain of plywood parallel to the length of the bearers	
	9-mm Thickness	12-mm Thickness	9-mm Thickness	12-mm Thickness
300	1,270	1,950	760	1,220
400	540	850	320	540
450	320	640	195	390
600	170	360	110	170

The above loads apply when the concrete is laid on concrete shuttering plywood as in slabs and beams. The same thickness of concrete in a wall can be held without excess deflection by thinner boards. According to IS: 4990–1993, the maximum load should be reduced to 75 % if wet boards are used.

Plywood in general is used for the formwork in a number of ways. For example, the plywood panels are nailed with supporting timber members either in frame form or without the frame form. The frame form is mostly used for wall and column application while in the case of slab, the frame form may not be there. The frame can be made with timber as the supporting member. In some applications, plywood boards are embedded in all steel frames. The frames are made out of steel angle or channel sections and plywood is nailed into it. Sometimes on a frame made out of timber, a thin ply veneer is also placed. Some manufacturers combine plywood with aluminum frame also.

2.3.2 Commonly Available Sizes of Plywood and Their Properties

The common plywood sizes for concrete shuttering works are given in Table 2.11.

Table 2.11 Common Sizes of Shuttering Plywood

Sl. No.	Length (mm)	Width (mm)
1	2,400, 2,100, 1,800, 1,500, 1,200,	1,200
2	2,400, 2,100, 1,800, 15,00, 1,200, 900	900
3	1,200	600

The plywood for the shuttering application is available in various thicknesses. A leading plywood manufacturer of the country offers plywood in 4 mm, 6 mm, 9 mm, 12 mm, 16 mm, 19 mm, and 25 mm thicknesses. The 12 mm and 19 mm plywood thicknesses are the most commonly used. A minimum thickness of 12 mm with proper frame work is suggested if plywood is used. For heavy load applications such as bridge decks, thick plywood members may be required. The smaller thicknesses of plywood are used for applications such as casting curved wall, dome etc. where it is desired to bend the plywood. The data on the weight in kg/m^2 of plywood from a leading Indian plywood manufacturer is given in Table 2.12.

Table 2.12 Weight Data of Plywood

Thickness	Weight (kg/m^2)
6 mm	3.90–4.62
9 mm	5.85–6.93
12 mm	7.80–9.24
18 mm	11.70–13.86

The code IS: 4990–1993 also suggests the minimum bending radii for the plywood of different thicknesses. They are given in Table 2.13 purely for guidance purposes.

Table 2.13 Bending Radii

Thickness mm	Across the Grain of the Outer Plies m	Parallel with the Grain of the Outer Plies m
6	0.9	1.25
9	1.65	2.15
12	2.55	3.30

Plywood without any plastic coating or a suitable overlay, may be bent to still smaller radii by soaking in cold or hot water (temperature up to 70°C) before fixing.

2.3.3 Requirement for Plywood for Formwork Application

IS: 4990–1993 specifies the requirements for plywood for formwork applications. One of the important requirements for the plywood is its moisture absorption property. The moisture absorption should be as low as possible, otherwise the plywood would absorb the moisture from the concrete. This would leave insufficient water near the plywood surface which would hamper the cement hydration process and the concrete may not gain the required strength.

Another problem due to moisture absorption is the loss of binding ability of the resins used in the manufacturing of plywood. The moisture results in peeling off of the veneers of plywood. It is thus recommended to use hard wood veneers in conjunction with water insoluble resins.

While using the forms, care should be taken in storing them. It is imperative to avoid the drying of forms either due to direct sun or due to hot dry winds. Thus, as far as applicable, both the used and unused forms should be stored in shade. Also, the application of the form release agent or form oil helps to prevent undue drying of forms when the forms are laid in position and concrete pouring gets delayed.

The plywood must also be durable under alternate wetting and drying conditions. Although this requirement makes the plywood a little costlier, nevertheless it is required, and is one of the guiding parameters for the selection of plywood to be used as formwork material.

Phenolic resin bonded plywood having an overlay of resin impregnated polymer film on both sides is also available these days to the construction industry. Commercially, they are also referred to as 'film faced shuttering plywood'. The surface films in such a plywood have the following advantages to offer:

1. Makes the plywood invulnerable to moisture.
2. Protection from deterioration caused by concrete and slurry.
3. Smooth finish to the concrete.
4. Large reuses are possible.
5. Very good for fair faced concrete application.

2.4 STEEL

Compared to timber formwork, steel formwork provides smoother concrete surfaces and is found to be economical if there are a large number of uses. It has adequate rigidity and strength. These forms can be erected, disassembled, moved and re-erected at a faster pace, provided suitable handling equipments are available. The steel formwork system facilitates in maintaining accurate alignment, levels and dimensions.

The cost advantage would turn into disadvantage if there were less number of reuses of the steel form. Also, these forms offer little or no insulation protection to the concrete placed during cold weather.

2.4.1 Commonly Used Steel Sections for Formwork and Their Properties

Steel plates of 3.15 mm and 5.0 mm thicknesses are commonly used for fabricating the sheathing. IS Angle sections $50 \times 50 \times 6$ and $65 \times 65 \times 6$ are quite commonly used for making steel panels to be used in the wall, column, and slab and beam applications. Channel section ISMC 100 is the common fabrication material for fabricating walers and soldiers. It is very common to use 40 NB circular steel pipes for shoring and scaffolding works. Some of the steel components are even galvanized for durability and to reduce maintenance.

The cross sectional area (A), moment of inertia (I), section modulus (Z), and radius of gyration (K) of steel tubes of different diameters are provided in IS: 2750–1964. For example, for a 40 mm nominal bore steel tube (frequently used), the cross sectional area (A), moment of inertia (I), section modulus (Z), and radius of gyration (K) are 557 mm^2, 1,37,700 mm^4, 5,700 mm^3, and 157 mm respectively.

2.4.2 Permissible Stresses for Steel

The design calculations for the steel formwork system can be made accurately for the imposed loads to ensure minimum material use to achieve the desired results. This is because of the known characteristics of steel and the controlled environment in which steel is produced.

Axial stress in compression— The permissible stress in compression can be obtained from Figs. 2.5, 2.6, and 2.7 for different *l*/*r* values. Figures 2.5, 2.6, and 2.7 correspond to Grade YSt 22, YSt 25, and YSt 32 respectively. On the horizontal axis, *l*/*r* is specified while on the vertical axis permissible stresses are specified. For some select *l*/*r* values the permissible stresses are specified on the figures as well.

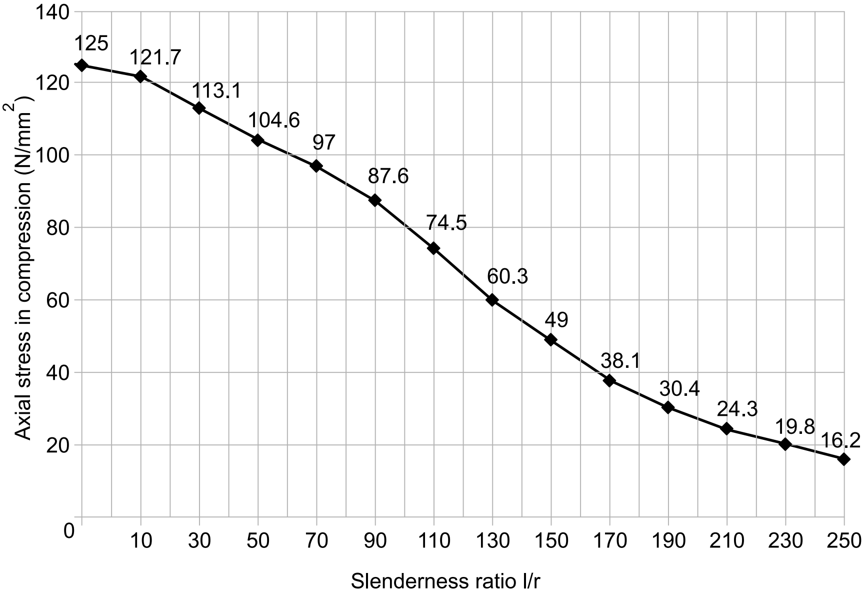


Figure 2.5 Permissible Axial Stress in Compression for Different *l*/r for Grade YSt 22.

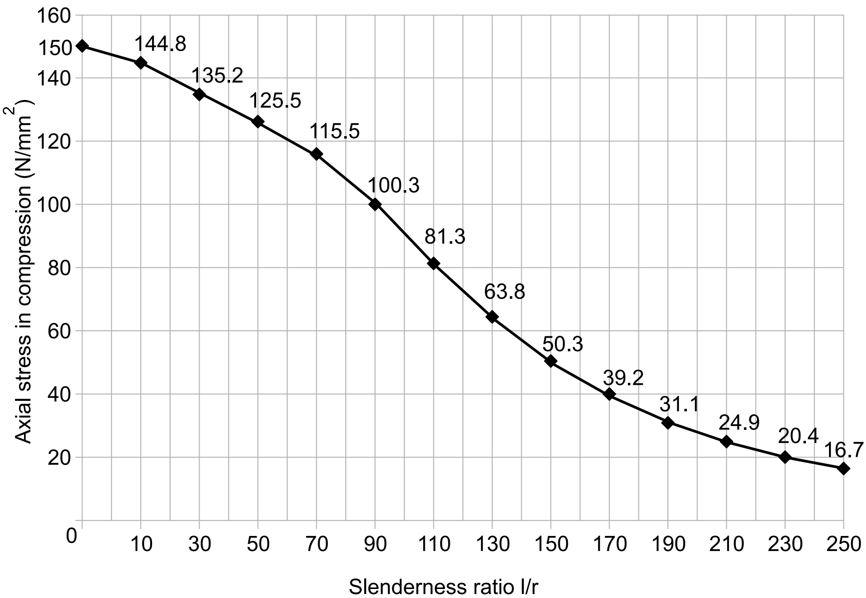


Figure 2.6 Permissible Axial Stress in Compression for Different *l*/r for Grade YSt 25.

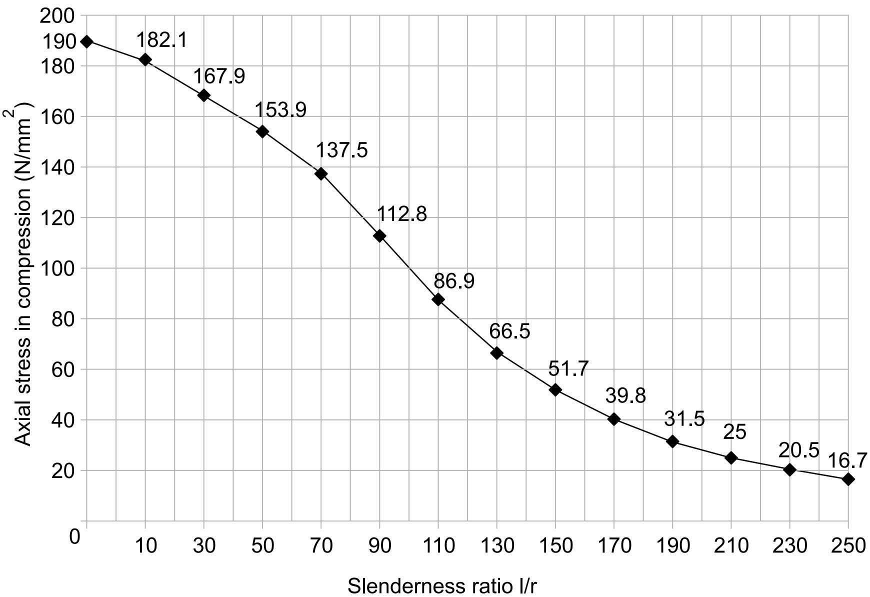


Figure 2.7 Permissible Axial Stress in Compression for Different *l*/r for Grade YSt 32.

Axial stress in tension—The direct stress in axial tension on the net cross-sectional area of tubes shall not exceed the values of F_t specified in column 2 of Table 2.14. The F_t values for Grades Yst 22, Yst 25, and Yst 32 are 125 N/mm^2, 150 N/mm^2, and 190 N/mm^2 respectively.

Bending stress in extreme fibers in tension and compression—The permissible stress in extreme fibers in tension and compression F_v are provided in column 3 of Table 2.14 for the three grades of steel: Yst 22, Yst 25, and Yst 32.

Shear stress—The permissible shear stress F_s values are given in column 4 of Table 2.14 for the three grades of steel: Yst 22, Yst 25, and Yst 32.

Bearing stress—The permissible maximum bearing stress values F_p are given in column 5 of Table 2.14 for the three grades of steel: Yst 22, Yst 25, and Yst 32.

Table 2.14 Permissible Axial Stress in Tension F_t, Permissible Bending Stress in Extreme Fibers in Tension and Compression F_v, Permissible Maximum Shear Stress F_s, and Permissible Maximum Bearing Stress F_p

(1) Grade	(2) F_t (N/mm^2)	(3) F_v (N/mm^2)	(4) F_s (N/mm^2)	(5) F_p (N/mm^2)
Yst 22	125	140	90	170
Yst 25	150	165.5	110	190
Yst 32	190	205	135	250

Some of the relevant IS codes providing specification on steel forms are IS: 2062–2006, IS: 8500 –1991, IS: 1977–1996, IS: 800–2007, and IS: 1161–1998.

2.4.3 Application of Steel in Formwork

Steel is used for formwork sheathing, joists, beam, horizontal and vertical shores, and formwork accessories and hardware. The various applications of steel in formwork and their brief features are given in Table 2.15. A number of proprietary designs of steel formwork are currently available throughout the world both for vertical as well as horizontal concreting.

Table 2.15 Application of Steel in Formwork

Sl. No.	Formwork application	Features
1.	As sheathing material	Steel plates can be used as sheathing materials. The commonly used thicknesses are 3.15 mm and 5.0 mm.
2.	In formwork centering	Light weight structural steel sections, such as angles, tees, channels, rods, tubes, sheets and plates are commonly used in the fabrication of steel formwork and in centering.
3.	As horizontal and vertical shores/ formwork support	Steel is also used extensively for making horizontal as well as vertical shores/formwork supports. The shores are available in different sizes to cater to different ranges of heights and different load capacities. It is very common to use circular steel pipes as they have the best geometric efficiency for axial compressive loads; some of these are also galvanized for durability and to reduce maintenance. The formwork supports in steel are designed specially to cut labor cost in erection and stripping and to make them versatile by incorporating an adjustability feature in most cases.
4.	In form accessories and hardware	Steel has been an important material for fabrication of form accessories and hardware. Tie rods, insert, coil bolts and nuts are very conveniently manufactured using steel.
5.	All steel system wall, column, and slab formwork	Steel wall, column, and slab formworks are also available these days by various manufacturers. These system formworks essentially comprise of a standard modular steel panel with angle or rolled steel sectional frame with sheet. In case of wall formwork, the panels are used in combination with strong back members called soldiers, spaced at regular intervals and wall ties. In case of column formwork, the above mentioned panel has additional accessories called adjustable column clamps. In case of slab formwork, the panels rest over walers which in turn are resting on steel shores.
6.	With other formwork system	Some steel components such as framing and bracing are used in conjunction with wood and plywood panel systems as well.

2.5 ALUMINUM FORM

The aluminum formwork is more or less similar to steel in a formwork application. In some cases they prove to be a better alternative to steel. It has lighter density when compared to steel and thus it is necessary to use larger sections when forms are made of aluminum because of their lower strength in tension and compression compared to steel. The aluminum formwork requires comparatively less labor time in erecting, handling, and disassembling when compared to other materials such as steel. The handling of aluminum formwork is also easier. Besides, the transportation and handling costs are also lower. Some manufacturers (http://www.wallties.com, http://www.hi-lite-systems.com) claim up to 2,500 repetitions with aluminum form, provided reasonable care, cleaning, and maintenance are carried out at reasonable intervals. In some residential cases, light commercial and precast concrete construction aluminum formworks are found most suitable among the various possible alternatives. Some more features of aluminum formwork are given in Box 2.2.

BOX 2.2: Features of Aluminum Formwork

1. The aluminum forms are similar to steel forms.
2. Because of lower density and being lighter than steel, it is necessary to use larger sections when forms are made of aluminum as they have lower strength in tension and compression compared to steel.
3. Very less labor required. Handling is easy.
4. Pure aluminum is attacked chemically by wet concrete but certain aluminum alloys are found resistant to wet concrete as well as atmospheric corrosion, and they are used for making aluminum forms.
5. Lightweight props in aluminum alloy tubes are also used.
6. The aluminum forms are highly favored in countries where mechanical handling equipment are used on a large scale, labor is in short supply, and the cost of engaging labor is also prohibitive.

2.6 PLASTIC FORMS

Plastic has given us a better alternative to wood. Plastic formwork is not only eco-friendly but it also gives better finish, easy transportation etc. Moulds made of plastic are light, they do not rust, and cleaning after use is easy. These moulds resist mechanical handling; they do not deform easily and can be quickly assembled and stripped. The jointing of plastic formworks is also easily achievable. They can also be re-used a large number of times. Where smooth concrete surfaces are required, plastic sheet linings are generally used. Plastic forms allow greater freedom of design. Using this form, we can mould even unusual textures and designs. Also, there is no size limitation.

The best known plastic sheathings are those made of PVC, neoprene, and polyester strengthened with glass fiber.

Glass fiber reinforced plastic moulds are particularly suitable for the forming of architectural shapes. Technologies in the plastic industry have advanced so much that just by injection methods, perfect moulds are produced. Polypropylene moulds for waffle slabs are increasingly being used. These are very common in the USA and Europe. Such moulds are light, easy to transport and handle. They require no extra maintenance, and they can be welded also. In India, development of plastic moulds is in the preliminary stage.

Fiber glass forms are strong and lightweight, capable of producing high quality concrete surfaces, and generally last through many reuses. They also can easily be moulded into shapes. Trough and waffle units in fiber glass are used in the construction of large floor areas and multi-story office buildings.

2.7 OTHER MATERIALS

2.7.1 Plaster of Paris Forms

These forms are primarily used for architectural ornamental purposes. Plaster of Paris is poured over some base materials such as timber and then left to solidify. Once set, the concrete is poured onto this form. Finally the moulds are destroyed after the concrete has set.

2.7.2 Hard Boards

Hard board is a board manufactured from wood fiber under the controlled combination of pressure, heat, and moisture. Hard boards are tempered by impregnating with drying oils which are stabilized by baking or heating after impregnation. Tempered hard boards have improved strength properties, lower rate of water absorption, better abrasion resistance and therefore, these are used for formwork lining. Generally, hard boards being thinner— about 6 mm — are flexible, and are used for lining of the curved surfaces. Hard board is to be treated as a structural material unlike plywood and timber, and therefore, it needs proper backing either by plywood or timber.

2.7.3 Lost Forms

Precast concrete planks, pressed fiber planks, cardboard tubes, precast reinforced concrete joists and clay filler blocks, ferrocement planks etc., are examples of lost forms. These forms are left along with the laid *in-situ* concrete and form an integral part of the construction. Precast concrete planks of various thicknesses are usually produced in factories and are transported to building sites. Precast concrete planks are designed as slabs to act in one-way or both ways. This system of utilizing precast concrete form itself as shuttering, is largely in vogue in the USA and Germany. This system provides a flat ceiling similar to flat slabs. Various patented systems are marketed in the USA and Germany.

In India, Structural Engineering Research Centre, Madras, has developed a similar system of "*plate floors*", where reinforcements are so detailed that they form truss-like stiffeners. Their tests have shown good agreement in performance compared to calculated values. Use of architecturally finished concrete, PVC liners and even wood wool slabs is prevalent as a practice of using lost forms. Here the surface texture is obtained automatically on vertical surfaces, as otherwise it is produced from the use of liners on built up forms.

2.7.4 Fiber Forms

Fiber forms are generally used as lost forms especially for concrete walls, roofs and slabs. These are left in place on the exposed face of the concrete where it gives an architectural look and also improves the acoustical and insulating properties. Fiber boards are made from such substances as glass fibers or wood fibers. Recommendations published by the fiber board manufacture should be followed in setting up and spacing of supports for the form boards.

Fiber form manufacturers also supply standard circular fiber tubes for concreting circular columns in standard sizes. The tubes are water proofed with a plasticized treatment. They can be removed after the concrete hardens.

2.7.5 Gypsum Boards

Gypsum moulds are generally used to provide artistic design or ornamental pattern for the exposed concrete face. Gypsum mix is reinforced with organic fiber or coir or sisal to get structure toughness. Concrete quality may improve due to the absorption of superfluous water in the concrete. The fragility of gypsum surfaces impedes its widespread use.

2.7.6 Asbestos Tiles

Corrugated asbestos tiles like corrugated sheets have been used as the re-usable shuttering material where corrugated face is required. Asbestos tubes cut into two halves and then bound together have also been used for forming cylindrical columns. However, the strength of the asbestos tubes itself being insufficient, it is likely to break during transportation and moreover, asbestos material is likely to stick to the concrete or even bind to it and therefore, it requires sufficient lubrication or releasing agents to be applied before concreting.

2.7.7 Wire Mesh

The wire mesh is a kind of lost form. The wire mesh may act as a form to take up plaster. One such application of wire mesh in the construction of ferrocement water tanks is shown in Fig. 2.8. In the figure, the wire mesh is wrapped on a stiff wire frame. The cement rich mortar is applied separately for the inside and outside surfaces. Though not commonly used, the technique could be useful for constructing curved surfaces such as shells.

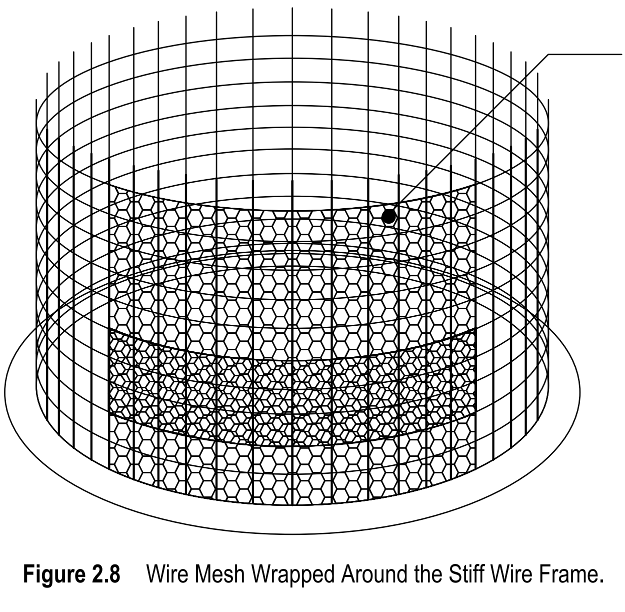


Figure 2.8 Wire Mesh Wrapped Around the Stiff Wire Frame.

2.7.8 Inflated Membranes

Inflated membranes are used (1) occasionally to form structures of varied shapes such as spherical, elliptical, cylindrical etc. (2) for providing block-outs or cavities in areas where access is difficult and the application of other form materials is difficult. In the two cases mentioned above, the inflated membranes can prove to be an economical solution of formwork compared to other systems.

The membranes are inflated by different ways. The principle here is to anchor the membranes to some supporting structures such as a foundation slab or a ring beam, and then to inflate the membrane. After the membranes have taken the desired shape, they are coated with plaster by shotcrete. The plaster is reinforced with wire mesh to bring rigidity to the inflated forms.

The dismantling of the form is performed easily by deflating the membranes. The application of inflated membranes can provide considerable saving in the construction and stripping of forms.

2.7.9 Fabric Formwork

Fabric formwork uses a flexible textile membrane in place of the rigid formwork panels normally used in concrete construction. When wet concrete is contained by a thin formwork membrane, the flexible fabric container naturally deflects into a repertoire of precise tension geometries (West and Araya 2009). Fabric formworks can be used to form columns, walls, beams, slabs and panels in both pre-cast and *in-situ* construction. They can also be used to produce complicated structural curves and sculptural forms. The advantages and disadvantages of fabric formworks are given below:

Advantages of fabric formwork

1. Fabric formwork allows air bubbles and excess mix water to bleed out through the mould wall and thus retaining a cement-rich paste at the surface of the cast concrete element. This results in a very good concrete surface quality and improved strength of the concrete element. This also eliminates the requirement of expensive treatment of the concrete surface.
2. Complicated shapes such as shells and vaults can be easily formed using fabric formwork.
3. Fabric formworks are extremely lighter (100 to 300 times) than the conventional wooden formwork.
4. The light weight also results in reduced material, storage, and transportation cost. The cost of fabric formwork is about one tenth of the cost of the formwork plywood per unit area.
5. The low cost and reusability of fabric formworks provide the options of both sacrificial (lost) and reusable formwork.

Disadvantages of fabric formwork

1. The fabric formwork lacks the required rigidity.
2. There is a lack of knowledge and experience in using fabric formwork and there are limited manufacturers for fabric formwork.
3. Fabric formwork does not retain moisture well during concreting.
4. There are limited literatures available for designing the fabric formwork.
5. The testing facilities for formwork are also limited.

2.7.10 Insulating Concrete Forms

Insulating concrete form (ICF) is an example of stay in place type (lost) formwork. It serves the purpose of a formwork as well as an insulator. The insulating concrete forms are available in modular units which can be assembled quite easily at the site just like interlocking bricks. ICFs are made up mainly from foam materials such as Polystyrene and Polyurethane. This results in a light weight formwork which not only reduces labor cost but also saves the time of installation of formwork considerably. After the installation of insulating concrete forms at site, rebars are placed and concrete is poured. After the curing of concrete, the forms are left in place permanently. The left out forms help in thermal and acoustic insulation primarily. ICFs are versatile formwork materials, and by using them, it is possible to incorporate curves, angles, slopes, and arches in the concrete elements to be cast.

2.8 FORM COATINGS AND MOULD LININGS

Mould surfaces of wood and steel in due course, after repeated use, warp and rust, respectively, making the moulds unserviceable, if they are not protected.

It is common to treat the sheathing materials with some coating or releasing agent for ease in their striking off. Only a few special form face materials, such as expanded polystyrene, do not need to have a release agent.

The coating or release agents are temporary coatings, and are composed of fatty acids which react with the alkali in cement. The reaction produces a soap-like substance on the contact surface which helps in the removal of forms in an easy manner. Coating agents can also help enhance the mould life. The form surface will remain smooth and provide good abrasive resistance, make the wooden surface moisture resistant and retard the rusting of steel.

There are many chemical compounds that have been developed for use as the coatings for wood and steel. It is therefore important to make sure that the right one is being used.

The three most common types of release agents are:

- Neat oils with surfactants: mainly used on steel faces, but can also be used on timber and plywood.
- Mould cream emulsions: for use on timber and plywood — a good general purpose release agent.
- Chemical release agents: can be used on all types of form face recommended for all high quality work.

The release agents could be oils, emulsified wax, oil phased emulsions with water globules, petroleum based products, catalyzed polyurethane foam, etc. The waste oil is also used as a release agent.

The type and composition of the coating or releasing agent is dependent on the following:

- type of sheathing materials,
- conditions under which it is to be applied,
- type of concrete,
- quality of finish,
- area of form, and
- ease of application.

The primary objective of the coating and releasing agents is to ensure easy striking off of the form material without injuring the form and the concrete. The concrete surface should also not get any stain from the application of release agents. It should be possible to apply the form release agent in an even manner on the form surface. The form release agent should not react with the concrete and produce some undesirable substance in the process.

Coatings on all type of forms are employed with the following objectives:

- Protection of the form for durability.
- On timber, it reacts with organic constituents and provides a uniform surface on each use. Penetration of the chemicals control grain or edge effect as well as it fills pores in timber.
- Chemically active coating reacts with free lime from the fresh concrete and produces water insoluble soaps. When dried, these soaps act as positive concrete release agents. When wet, they help in the movement of air voids along the form.

Coatings including release agents commonly used are: Straight oils: emulsified wax applied at a temperature of about 10°C; oil phased emulsions with water globules encased in a continuous phase of oil; petroleum based products; catalyzed polyurethane foam; epoxy resin.

As liners, fiber glass, silicone rubber, styrophors, rubber sheetings, vitreous clay, etc. are used. They give excellent results in forming architectural concrete. Liners and form release agents may not be used together, as undesirable chemical reactions may take place.

Because of their absorbency, new and untreated timber and plywood should always be given a coat of the appropriate release agent at least 36 hours before being used. A second application should then be made before using for the first time. For all further pours, a normal application is all that should be necessary.

Release agents are prepared by the manufacturers to suit various requirements and their instructions must be followed carefully. The most common fault with release agents is far too much to be put on, which can stain concrete. On the other hand, if not enough is applied, striking is made difficult and both the concrete and the form face can be damaged.

The release agents should normally be applied as a thin film with brush, roller or spray. If by mistake, too much is applied, the excess should be wiped off with a clean rag.

2.9 FORM ANCHORS

Form anchors are devices embedded in previously poured concrete and are used for securing the formwork for the subsequent lifts. The anchors are placed in position during the placement of concrete. The anchors are basically in two parts, viz. the embedded part and the bolt or other external holding device which is removed after use. The strength of the anchor depends upon the strength of the concrete, the area of contact between the concrete and the anchor, and the depth of the embedment. Various proprietary designs of embedment are available for the anchors. Some of these are: open metal spirals called screw-anchors or loop anchors consisting of coil nut to which one or more metal loops are welded, and pigtails with one end crimped and the other end threaded. Bolts or holding devices for the anchors are usually simple bolts with one end threaded to suit the coil nut profile or 'she' type bolts tapered with one end having internal threads and the other end with square threads with a nut and washer plate either tilting or with plain ends.

Usually, pigtail anchors or spiral screw-anchors are used for mass concrete construction like dams. The other type viz. loop-type anchors are used for single sided formworks for shallow lifts.

The ties and anchors are made both of mild steel and high tensile steel to cater for various strength requirements. The manufacturers provide complete data regarding the strength and other particulars of ties and anchors depending upon their size, strength of the concrete, depth of embedment, etc. Figures 2.9 to 2.13 show different types of form anchors.

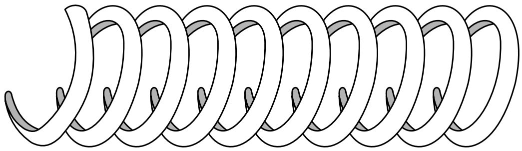


Figure 2.9 Screw Anchor.

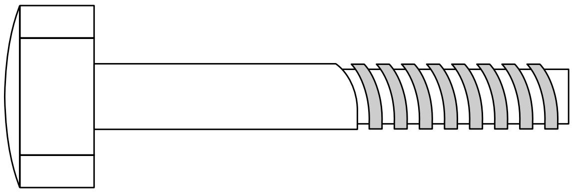


Figure 2.10 Screw Anchor Bolt.

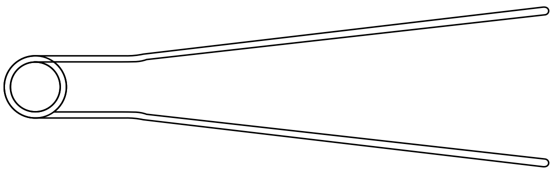


Figure 2.11 Hair Pin Anchor.

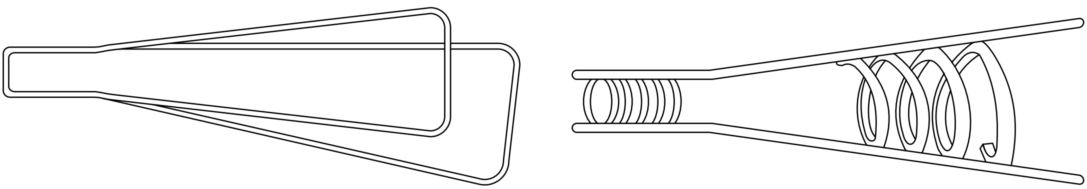


Figure 2.12 Loop Anchor.

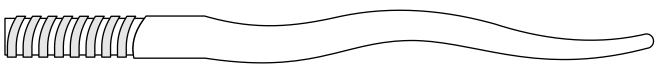


Figure 2.13 Pig Tail Anchor.

2.10 TIE SYSTEM

Formwork for the concrete structures must be secured so that the lateral pressure exerted by the wet concrete does not cause the formwork to shift from its required position. In the traditional method, the holding back of formwork in position was achieved through wooden spreader and wire ties.

One such example of twisted wire loop wall tie is shown in Fig. 2.14. Such ties are simple to use and are popular even in current construction practices.

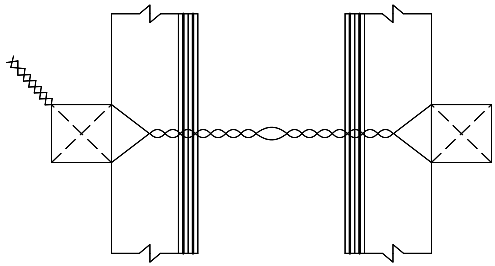


Figure 2.14 A Twisted Wire Loop Wall Tie.

In recent times, various form ties of primarily steel, have evolved. These form ties are tensile units which hold securely the formwork for concrete structures against the above mentioned lateral pressure of wet concrete. The form ties can be made with or without the provision for spacing the forms a definite distance apart (the function of a spreader). Further, the form ties can be made with or without the provision for the removal of ties to a specified distance back from the finished concrete surface.

Modern ties made of special alloys are capable of resisting very high pressure. The form ties are available with safe load ratings ranging from 500 kg to more than 25,000 kg.

The form ties essentially consist of two parts: (1) internal tension unit and (2) external holding device. The form ties are manufactured in the following two basic types:

1. Continuous single member, also referred to as through tie or one-piece tie.
2. Internally disconnecting type, also referred to as lost tie.

The above two types of tying devices are identified commercially by various descriptive names, such as form clamps, coil ties, rod clamps, snap ties, etc. Except for taper ties, the continuous single member type is generally used for lighter loads, ranging up to about 2,500 kg safe load. The internally disconnecting type of tie is available for light or medium loads but finds its greatest application under heavier construction loads (up to about 35,000 kg). Form ties classification has been shown schematically in Fig. 2.15.

Besides the above two types, we discuss form ties for water retaining structures and a patented system of form tie.

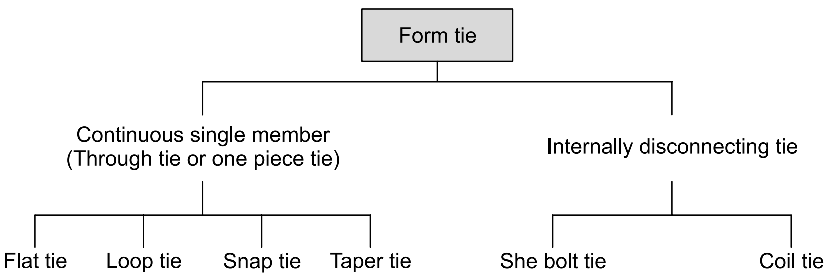


Figure 2.15 Form Ties Classification.

2.10.1 Continuous Single Member

The continuous single member tie is also referred to as through tie or one-piece tie. In the continuous single member, the tensile unit is a single piece. A special holding device is added for engaging the tensile unit against the exterior of the form. Some single member ties may be pulled as an entire unit from the concrete; others are broken back a predetermined distance; some are cut flush with the concrete surface.

The continuous single member tie comprises a threaded bar or tie-rod passed through a PVC tube. The PVC tube needs to be accurately cut to length to reduce grout loss on the surface of the concrete. Simple plastic cones are fitted at the ends of the tube. The tube leaves a hole through the wall which needs to be plugged after removal of the forms. After use, the tie-rod is removed and reused. In other words, such ties are recoverable. The Formwork-Guide recommends that the minimum factor of safety is 2 on these recoverable tie rods. The safe tie load is based on the minimum guaranteed strength of the bars used.

Whenever the concrete face is to be of the exposed type, it is desirable not to have the exposed tie ends at the surface, and therefore, various devices, such as plastic or wooden cones are incorporated with the tie systems. The cones are removable and the small recesses on the face can be neatly patched.

2.10.1.1 Flat tie

Flat tie serves both the functions (1) of securing the form in position, as well as (2) of maintaining the distance (spacing) between forms. The flat ties are available in several configurations and the safe load carrying capacity of these ties may vary from 750 kg to 1,500 kg.

One variant of the flat ties are the adjustable flat ties. Such ties can be used under different job requirements. One such adjustable flat tie is shown in Fig. 2.16.

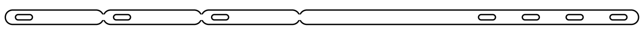


Figure 2.16 Adjustable Flat Tie.

As can be noticed from Fig. 2.16, the adjustable flat tie is made up of a flat strip of mild steel with a series of slots or holes uniformly spaced across the tie. The adjustable ties are designed for adjustments in an increment of about 25 mm. The ties are tightened by a wedge inserted in the slot at a specific position indicated by the wall size.

The tie rods are provided with notches for breakback. At the notches, the width of the flat is reduced for easy break off which can be performed by striking with the hammer. Normally for an inside wall surface, the position of breakback is about 5-6 mm below the wall surface and thus the inside wall surface is to be filled up with grout or mortar. For the outside wall surface, the breakback is flush with the edge of the outside wall and therefore the depression left by the tie rod does not require filling.

2.10.1.2 Loop ties

A loop tie can serve the dual function of securing the form from lateral pressure besides maintaining the spacing between the forms. Figure 2.17 shows a typical loop tie with crimp. The safe load carrying capacity is approximately between 1,000 kg to 1,500 kg.

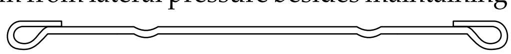


Figure 2.17 A Loop Tie.

2.10.1.3 Snap ties

Snap ties are a type of through tie that allows the ends of the ties that stick out from the concrete face, to be removed by breaking them off, usually by hitting it with a hammer. These ties are provided with a weak point, a short distance from the concrete face at which the break can be made. The snap ties are inserted in the formwork through small holes which have been bored through the sheathing and studs. The spreader washer is set at the correct wall thickness to hold the walls apart. The assembly is tightened on each end by the fastener. After the concrete has cured, the fasteners are removed and the forms are stripped.

A number of variants of snap ties are available. Each of them has its own specially designed ties, fasteners, waler brackets and bracing hardware although they may follow a similar design. Two such designs are shown in Figs. 2.18 and 2.19.

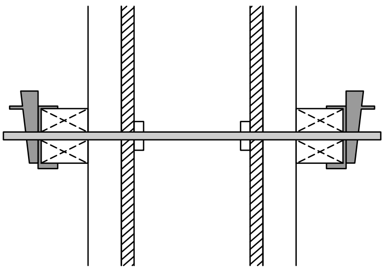


Figure 2.18 Snap Tie.

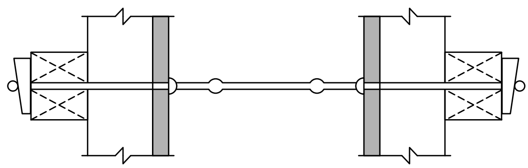
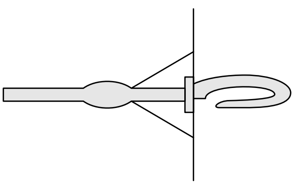
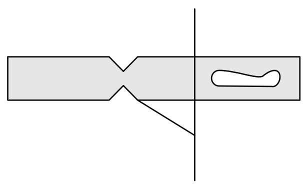


Figure 2.19 Snap Tie with Cone Spreader.

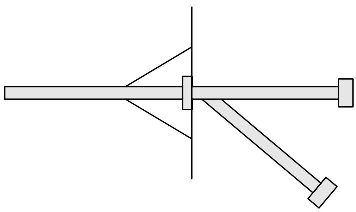
Figure 2.20 shows three designs of snapping arrangement in snap ties. As can be seen from the figures, the snapping action also breaks some of the concrete away. The resulting jagged edges are patched with grout or mortar. When the process is completed, the rod is completely enclosed in the concrete structure. The concrete face is made smooth, with evidences of the tie position. This is the reason why the use of snap ties is discouraged when high quality concrete surface is desired.



(a) Flattened rod -to be twisted out for snapping



(b) Notched steel fat -to be broken downwards for snapping



(c) Thin section- to be bent and twisted for snapping

Figure 2.20 Different Types of Snap Ties.

2.10.1.4 Taper ties

Taper ties (see Fig. 2.21) are a variation of the 'she-bolt' system. They consist of a taper tie which is a long tapered bolt which passes through the wall. The taper ties have different sized threaded nuts and waler plates on each end. After use, the entire taper tie is removed and reused. The load capacity is governed by the diameter of the smaller size. The safe load carrying capacity is between 3,500 kg to as high as 25,000 kg. These ties are used where the specifications allow complete removal of the form ties from the concrete.

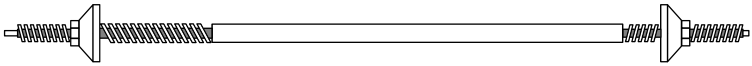


Figure 2.21 A Taper Tie.

2.10.2 Internally Disconnecting Type

The internally disconnecting type of tie is also referred to as lost tie. In such ties, the tensile unit has an inner part with threaded connections to removable the external members, which make up the rest of the tensile unit. In such ties, the tie rod is left inside the concrete structure itself— the reason they are called the lost ties. Since the tie is loaded only once, a typical factor of safety of 1.5 is used on these high tensile steel 'lost' ties.

The internal part may be a plain coil type tie consisting of two rods welded to the coil nuts at the ends, or a plain rod threaded at both the ends or a crimped rod threaded at both the ends. The holding device may be a simple bolt with rope threads to suit the profile of coil nuts or 'she' type tapered bolts having internal threading at one end to suit the threaded ends of the ties. The other end of the 'she' bolts may be provided with a nut and specially designed washer plates either fixed or of swivel type. The washer plates may also have locking device and nail holes for fixing to the timber walers. The washer plate should be large enough to transfer the load to the bearers, particularly in case of timber formwork.

The different forms of lost ties are discussed briefly in the following paragraphs:

2.10.2.1 She-bolt tie

The she-bolts (see Fig. 2.22) help to resist the tensile loads on the forms and they do not perform the function of a spreader. The she-bolt is screwed onto the end of a tie rod. The name she-bolt is due to the fact that the connection in the she-bolt is by female thread. The 'she' bolt does not touch the sheathing materials, thus the grout can leak along the shaft of the bolt even though the hole in the plywood is a tight fit. In order to remove the tie rod easily after the pouring of the concrete, the end of she-bolt at the connection to the tie rod is tapered. The safe load carrying capacity of the she-bolt tie varies from about 2,500 kg to up to about 30,000 kg. She-bolt ties made up of high strength steel can take load of up to about 75,000 kg.

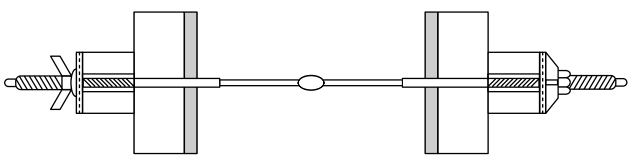


Figure 2.22 A She-bolt.

2.10.2.2 Coil ties

The coil tie (see Fig. 2.23) remains in the concrete. The tie consists of two side rods welded to spring shaped coils. The bolts thread onto these coils. Plastic cones are provided over the outside of the coils at their ends. The cones come in various lengths and provide a good seal against the grout leakage at the form face. The safe load carrying capacity of a two-strut coil tie may vary between 1,500 kg to 6,500 kg.

Figure 2.23 A Coil Tie.

2.10.3 Water Retaining Structures

The ties used for the water retaining structures should be leak proof and for this purpose, water-seal ties are used. This is achieved by crimping the tie rod attaching a metal washer as the water stop in the middle of the ties. When used on liquid retaining structures, proprietary cementitious tie hole fillers are available to pressure seal the hole.

2.10.4 Ties for One-sided Forms

In case of concrete structures for which only one side formwork is in place, a special tie, known as toggle tie, is used (see Fig. 2.24). Such ties are designed to be welded against the existing steel, or attached to wood sheathing by installing the rod through the wood and anchoring it with the toggle. A wire secures the toggle in place when it is attached to the wood so that any further movement of the form does not dislocate the toggle.

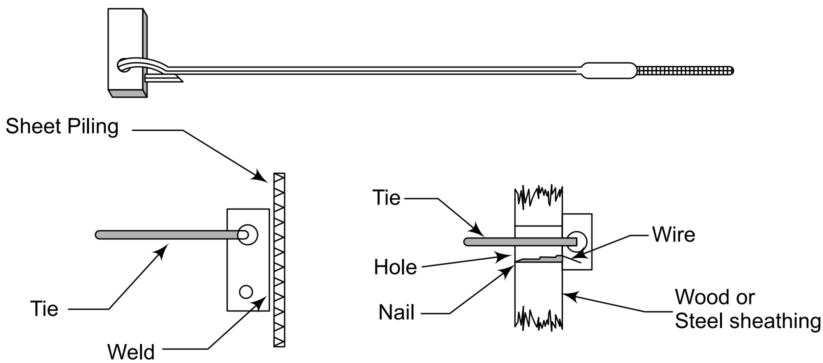


Figure 2.24 The Toggle Tie.

2.10.5 Patented Tie Systems

A large number of patented tie systems have been developed. The tie system developed by PERI, and known as the DK sealing system and the SK sealing system, are shown in Fig. 2.25 and Fig. 2.26, respectively. The DK sealing system of PERI has two reusable DK sealing cones, the reusable tie rod, and a lost spacer tube. A seal between the cone and the formwork compensates for unevenness, and prevents laitance from trickling in.

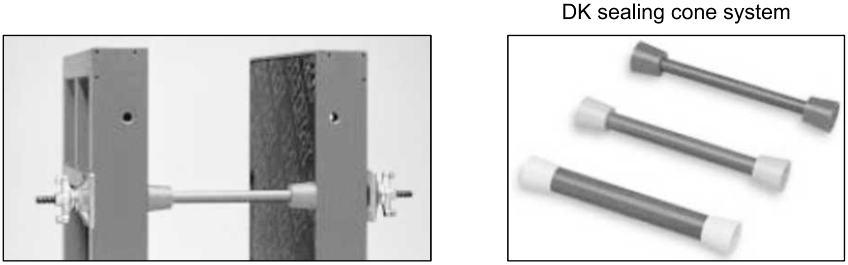


Figure 2.25 DK Sealing Tie System (Courtesy PERI).

The SK sealing system by PERI has two reusable SK tie cones and three tie rods, the middle one of which can be lost (e.g. for vaults and strong rooms and buildings providing radiation protection). For all other applications, a spacer tube is used to enable the middle tie rod to be recovered. The seal is used similar to the DK sealing system.

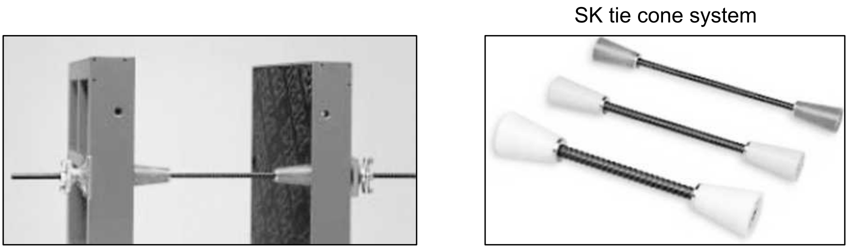


Figure 2.26 SK Sealing Tie System (Courtesy PERI).

The processes of retrieving the tie rod and the operations thereafter are shown in Figs. 2.27 and 2.28, which are self-explanatory.

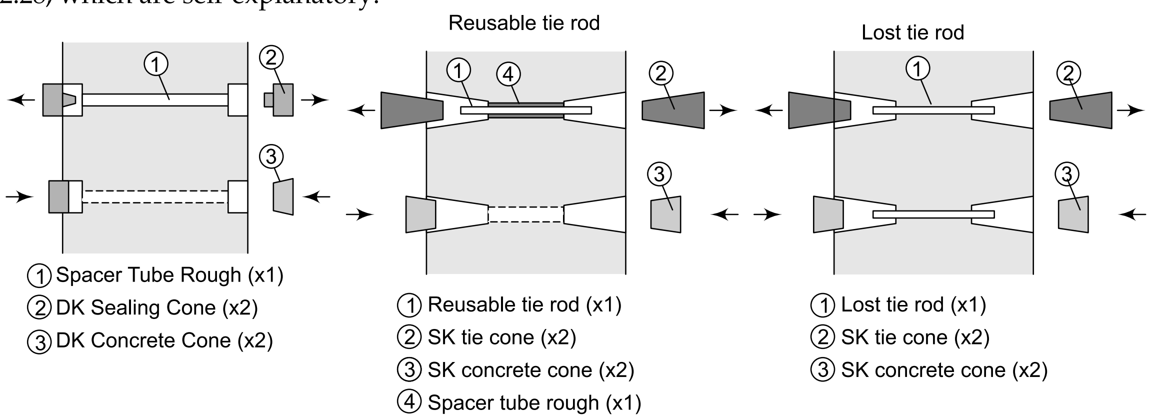


Figure 2.27 Patented Tie Systems (Courtesy PERI).

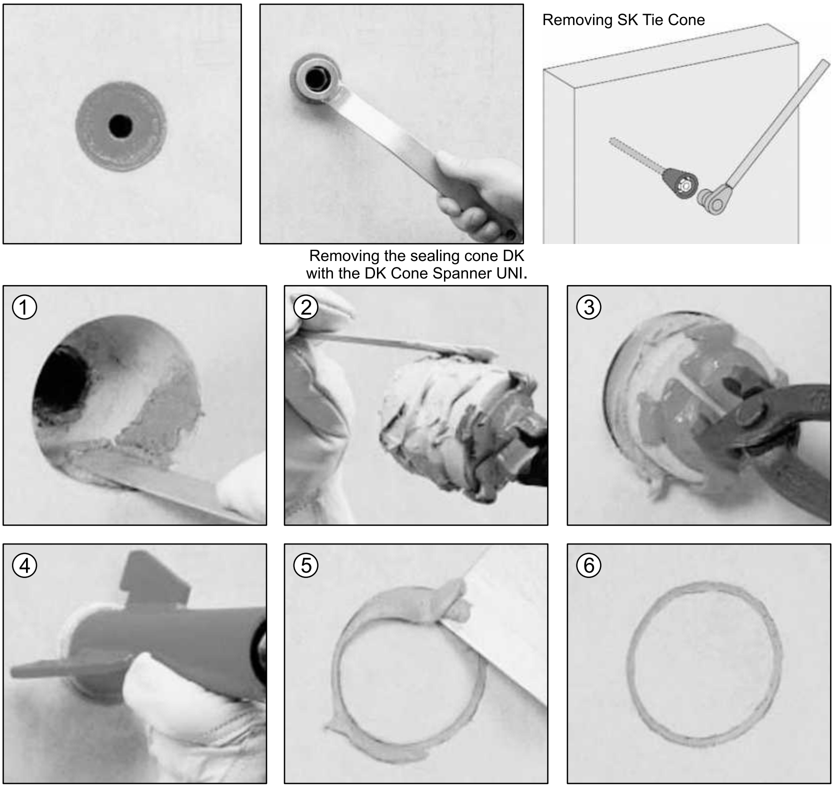


Figure 2.28 Operations after Tie Rod Retrieval (Courtesy PERI).

2.11 SPREADERS, SPACERS

These devices may be used to keep the forms in the proper position and to maintain correct spacing between the vertical form and the reinforcing bars. These may be made of high strength mortar (vibrated or pressed), concrete, various grades of plastic, steel, etc. One typical spreader in a wall formwork is shown in Fig. 2.29.

2.12 FORM LINING MATERIALS

Form liners are an attachment to the sheathing materials. Through the use of the form liner, it is now possible to provide various designs and textures to the concrete. Form liners are available in different shapes and designs. The form liner panel of the desired design is placed on the inside of the sheathing material, usually with the help of adhesives. Liners may even be tacked, screwed, stapled or nailed depending on the type of sheathing material.

Form liners act as mould for the concrete. Once the concrete has set, both the sheathing material and the mould are removed. The form lining material can be used in any form such as sheet, plate or layer and attached directly to the inside face of the form to improve or alter the surface texture and quality of the finished concrete.

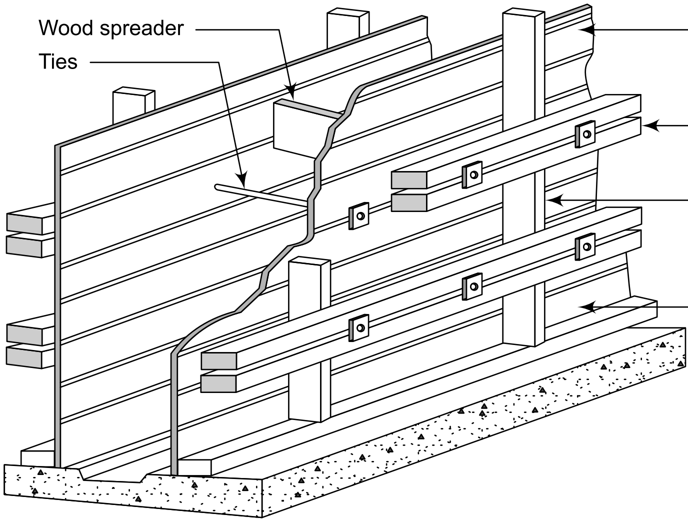
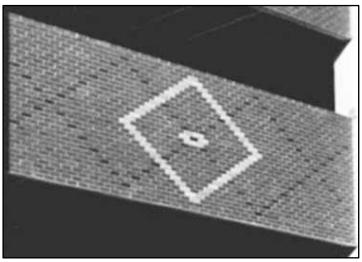


Figure 2.29 Spreader in Wall Formwork.

The design and texture imparted to concrete could be of varied nature such as leaves, animals, birds, clouds, fossils, logos, etc. Brick pattern, stone and rock pattern, and some customized pattern are shown in Fig. 2.30. More such patterns are available at http://www.formliners.com. The form liners essentially act as a reverse stamp. Without the use of form liner, the texturing can only be achieved in two stages: (i) pouring the concrete and (ii) applying the texture tool.



(a) Brick pattern

(b) Stone and rock pattern

(c) Customized pattern

Figure 2.30 Some Patterns Made From Form Liner.

The use of form liners has increased in the recent days since more and more end users of concrete are demanding not just plain concrete but the one which is aesthetically pleasing. A large number of proprietary form liners in different patterns such as brick pattern, fluted rib pattern, sand blast pattern, masonry pattern, stone and rock pattern, geometric pattern, wooden plank pattern, etc. are available.

Some form liners can provide only a single use while some form liners can provide as many as 100 uses (repetitions).

The salient features of different types of form liners are given briefly in Table 2.16.

Table 2.16 Salient Features of Different Types of Liners

Sl. No.	Type of Liner	Salient features
1.	Elastomeric form liner	These plastics are flexible enough to be peeled away from the cast concrete surfaces with slightly undercut areas. Peeling capability is largely lost if the liners remain attached to a rigid backing. They require good support and are usually adhered to the form sheathing. Tough, wear resistant. 100 to 200 uses reported possible with great reasonable care.
2.	Rigid	They are stiff enough to be considered self supporting. Such liners are available in standard sheets up to 3 m long, or on special order up to 9 m lengths. These liners are suited to ribbed or fluted wall surfaces.
3.	Fiber glass reinforced	These liners are similar in appearance and function to other rigid plastics. They are much stronger and have longer potential service life. Better quality glass fiber reinforced plastic liners have an extra gel coat of plastic resins at the contact surfaces to keep the glass fibers from blooming through the resin skin.
4.	Foamed or Expanded	Foamed polyurethane and expanded polystyrene can be readily cut or shaped with hand tools to form unique concrete surface texture.
5.	Rubber Liners	Rubber has, in common with the thicker sheet plastic materials, a long life and high reuse value. Molten rubber can be cast against complicated patterns and become the mould or form for ornamental concrete.
6.	Vacuum Liners	Vacuum treatment removes water and air bubbles from the surface layer of the freshly placed concrete. Improves appearance and durability. But it is too expensive.
7.	Absorptive Form Liners	The absorptive linings proved practical in eliminating voids and other common imperfections on concrete surfaces. Some examples of absorptive form lining materials are: cotton cloth, blotter type paper etc.
8.	Control permeability form liner	These form liners can be used for producing high quality durable concrete surfaces. These formwork liners provide a blemish free, dense and low permeability cover zone. The application of these form liners can result in a durability benefit equivalent to about an extra 15 mm to 20 mm of cover. These liners are manufactured with polypropylene fibers. The thickness would be less than 1 mm (0.7 mm offered by a reputed manufacturer). Release agents are not required when using these form liners.

2.12.1 Precautions to be Taken While Using Liners

- Thin sheets of liners are used only for horizontal surfaces. If they are used for vertical forms, they may wrinkle or sag.
- Thicker layers of lining materials are more adaptable to vertical surface because of greater rigidity.
- Extreme smoothness and relative imperviousness of some lining materials give rise to the problem of eliminating air or "bug" holes, particularly when oil or grease is applied.
- Highly absorptive materials used as form liners have eliminated voids and air pockets on the surface of the concrete and improved the durability of concrete.

SOLVED EXAMPLES

Example 1 *Compute the controlling l/d ratio for an unbraced column shown in Fig. S.2.1.1.*

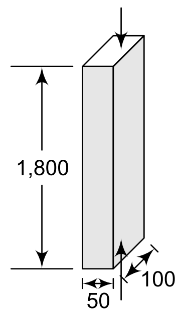


Fig. S.2.1.1 Unbraced Column

Solution

$$l/d \text{ ratio parallel to narrow face} = \frac{1{,}800}{50} = 36$$

$$l/d \text{ ratio parallel to wide face} = \frac{1{,}800}{100} = 18$$

The larger, controlling, l/d ratio = 36

Example 2 *Compute the controlling l/d ratio for a braced column shown in Fig. S.2.2.1.*

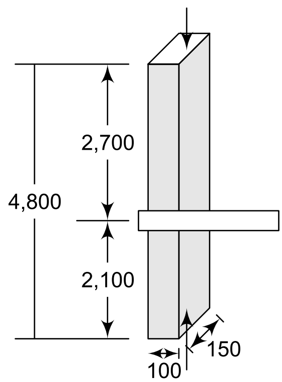


Fig. S.2.2.1 Braced Column

Solution

$$l/d \text{ ratio parallel to narrow face (longest unbraced length)} \ \frac{2{,}700}{100} = 27$$

l/d ratio parallel to wide face $\dfrac{4{,}800}{150} = 38.4$

Thus, the controlling $\dfrac{l}{d}$ is 38.4

REVIEW QUESTIONS

Q1. True or False

(a) Various formwork materials are – timber, plywood, steel, fiber reinforced plastics, plaster of paris, aluminum, form coatings, and mould linings.

(b) Important factors while considering different timber groups are – modulus of elasticity, and bending stress on extreme fiber.

(c) The three timber groups are – A, B, and C.

(d) The different types of anchors are – screw anchor, hair pin anchor, loop anchor, and pig tail anchor.

(e) Various patterns made from form liner are – brick master, fluted rib, fractured rib, sand blast, masonry, custom, geometric etc.

(f) Some advanced formwork materials are – fabric, polysteel, and plastics.

Q2. Match the following for the specification of timber for a formwork application:

1. Modulus of elasticity	(a) No prohibited defects in timber
2. Moisture content	(b) Consider permissible stress while designing
3. Defects	(c) 12 to 20% depending upon climatic zone of construction
4. Durability and treatability	(d) Not less than 550 N/mm^2
5. Design	(e) Suitable quality timber

Q3. Match the following form tie classification—

1. Continuous single member one piece tie	(a) She-bolt tie, coil tie
2. Internally disconnecting tie	(b) Flat tie, loop tie, snap tie, taper tie

Q4. Match the following for 40 mm nominal bore steel tube

1. Cross sectional area (A)	(a) 15.7 mm
2. Moment of inertia (I)	(b) 5,700 mm^3
3. Section modulus (Z)	(c) 557 mm^2
4. Radius of gyration (k)	(d) 1,37,700 mm^4

Q5. Short answer type questions

1. List out the characteristics of good quality timber formwork.
2. List out the features of aluminum formwork.
3. List out the advantages and disadvantages of fabric formwork.
4. Compare and contrast the polysteel formwork and plastic formwork.
5. List out the salient features of various types of liners.
6. What are the three most common types of release agents used in the context of formwork?
7. Under what circumstances will one prefer to adopt fiber reinforced plastic as formwork material and why?

8. What are the advantages of using timber forms?
9. What are the advantages of using steel forms?
10. What are the advantages of using aluminum forms?
11. For what purposes are the plaster of Paris forms used?
12. In what ways do the moulds for precast differ from those of *in-situ* construction?

Q6. What are the factors which govern the selection of form material? Find out their relative importance. Develop a decision tool to select the most economical form material under a given situation.

Q7. What are the factors which affect the reuse of timber formwork?

Chapter

3

Formwork Design Concepts

Contents: Introduction; Loads on Formwork; Dead or Permanent Loads; Imposed Loads; Environmental Loads; The Design Basis (Assumptions Made in Formwork Design); Estimating Permissible Stresses; Maximum Bending Moment, Shear Force, and Deflection

3.1 INTRODUCTION

Design of the formwork, though important, has not received the attention that it deserves. Many formwork failures have resulted on account of either the absence of formwork design or poorly designed formwork. The design concepts specific to formwork design are discussed in this chapter.

Design of the formwork basically involves four steps: estimating the load; forming the design basis; estimating the permissible stresses; and analyzing and designing each of the formwork components.

According to the Indian standard, the formwork should be designed to meet the requirements of the permanent structure using relevant Indian standards for the materials selected for formwork. The formwork design should take into account the conditions of the materials to be actually used for the formwork, as well as environmental and site considerations. The formwork design should address the following requirements:

- Safety
- Overturning
- Overall stability
- Prevention of collapse.

Before designing the formwork, the formwork designer should get familiar with the various details such as the site investigation report, various loads likely to be exerted on the formwork, expected loading schemes, sequence of erection and dismantling of formwork, method of concreting, sequence of concreting, and the total time of pouring the concrete. The sequence of concreting may prove to be a critical input for the formwork design of cantilevers, domes, and so on.

The design should take care of the expected dead load, imposed load, construction load, and environmental load. The design should address various load combinations of dead loads, imposed loads, environmental loads, lateral pressure, and incidental loads arising on account of erection and dismantling of formwork.

3.2 LOADS ON FORMWORK

Formwork for concrete must support all vertical and lateral loads that may be applied until such time as these loads can be carried by the concrete structure itself. The different types of loads exerted on the formwork are given in Fig. 3.1.

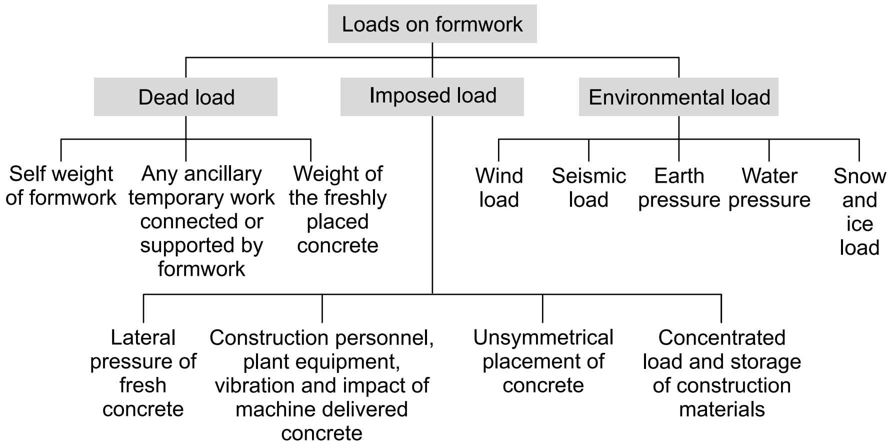


Figure 3.1 Classification of Loads Acting on Formwork.

The loads on the formwork can be broadly classified under three broad heads: (1) dead loads, (2) imposed loads, and (3) environmental loads. These are discussed below:

3.3 DEAD OR PERMANENT LOADS

The dead load comprises essentially of (a) self-weight of formwork, (b) weight of any ancillary temporary work connected or supported by formwork, filling materials (in ribbed slabs), and (c) the weight of freshly placed concrete including reinforcement steel.

The code recommends that the self-weight of formwork be determined on the basis of the actual measurement in accordance with IS: 875 (Part 1)–1987. However, in the absence of actual measurement, it may be assumed as 500 N/m^2 for the purpose of initial calculations.

The weight of any ancillary temporary work connected or supported by formwork, filling materials (in ribbed slabs) should be computed based on the codal provisions. Care must be taken to account for any moisture absorbed by the filling materials as a result of their wetting prior to concreting. For example, the weight of autoclaved aerated concrete blocks, normally (dry) 4.5 kN/m^3, is taken as 6 kN/m^3, which is 30% higher.

BOX 3.1: Extract of Dead Loads or Permanent Loads on Formwork

Sl. No.	Load type	Suggested value of loading
1 a	Self-weight of formwork	Self load shall be determined either by actual measurement or in accordance with IS: 875 (Part 1)–1987. In absence of the data, load may be assumed as 500 N/m^2 for the purpose of initial calculations. Additional weights of fittings shall be included in the self-weight calculation.
1b	Any ancillary temporary work connected or supported by formwork	Actual load shall be evaluated for use in design. Moisture correction to be accounted for.
1c	Weight of freshly placed concrete including reinforcement	The unit weight of wet concrete including reinforcement shall be taken as 26 kN/m^3.

The weight of freshly placed concrete including reinforcement according to Indian Standards is taken to be 26 kN/m^3.

3.4 IMPOSED LOADS

The imposed load comprises of the loads essentially from (a) lateral pressure of concrete, (b) loads from construction personnel, plant and equipment, vibration and impact of machine delivered concrete, (c) unsymmetrical placement of concrete, and (d) concentrated load and storage of construction materials.

3.4.1 Lateral Pressure

When concrete is first mixed, it has properties lying between those of liquid and those of solid substances as shown in Fig. 3.2. It is also defined as plastic material. With passage of time, concrete loses plasticity and changes into solid. Once concrete is set up there is zero concrete pressure.

Internal friction is higher in dry cement than a wet one and it increases with loss of water in the concrete. The speed at which concrete changes from plasticity to solidity has considerable effect on lateral pressure.

3.4.1.1 Factors affecting lateral pressure of fresh concrete

Over the years, various factors which affect the lateral pressure of fresh concrete on vertical forms have been investigated. These factors include rate of placing the concrete, temperature of the concrete, proportion of the concrete mix, consistency of the concrete, consolidation method of the concrete, impact during placing, size and shape of the formwork, amount and distribution of the reinforcing steel, unit weight of the concrete, height of the concrete, ambient temperature, smoothness and permeability of the formwork, pore water pressure and type of cement. According to Johnston *et al*. (1989), the pressure influencing variables are: concrete density, mix design, admixtures, slump, stiffening time of concrete, concrete temperature, pour rate, consolidation method, and formwork dimensions. It is important to know and understand the factors that can affect concrete formwork pressure to ensure a successful pour. When placing concrete, we will consider the different factors that play a role in determining the placement rate and height of pour.

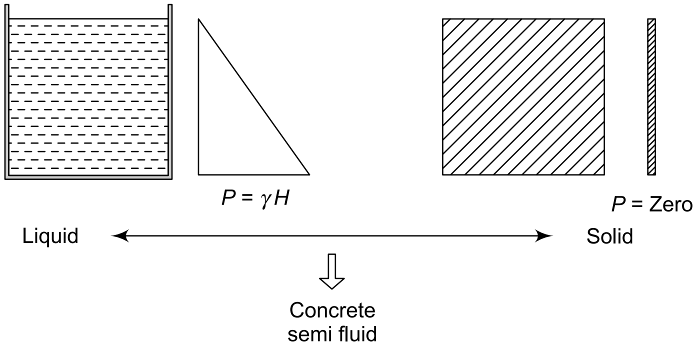


Figure 3.2 Hydrostatic Limit.

In general, in calculating the pressure of concrete on the formwork, the following factors are taken into consideration.

Unit Weight (Density) of concrete (kg/m^3)

The pressure increases in direct proportion to density. Thus, the concrete weighing less will exert a lower pressure, while the concrete weighing more will result in increased pressure on the formwork. In other words, the greater the density of the concrete, the greater will be the pressure exerted on the form face. Indian standards recommend a correction factor on account of the increase in density from the base density considered. This has been discussed later.

Workability of the mix, slump (mm)

The workability of concrete refers to the ease with which the concrete can be used for the purpose for which it is required. The pressure increases in proportion to increases in the workability, or slump, of concrete. The concrete with a low workability will be stiffer and so more self supporting than a mix with a high workability. In other words, a concrete mix with a low slump will exert less pressure than a mix with a high slump.

Rate of placing R (m/h) concrete in the forms

The rate of placement is measured in meters per hour (m/h). For example, if the height of a concrete wall to be poured is 3 m and it takes two hours to pour the concrete, the rate of placing would be 1.5 m/h. Slow rates of placing enable the lower levels of concrete to start stiffening before the pour is complete, and thus results in less pressure. On the other hand, if the rate of placing is fast, the concrete at the base of the pour will still be fluid and therefore the pressure would be more.

Concrete mix chemistry

The chemistry of the concrete mix also affects the concrete formwork pressure. The factors include: cement type, slag or fly ash and retarders — all of which increase formwork pressure. On the other hand, accelerators speed up the setting time and reduce pressure.

Concrete temperature (Deg Celsius)

High temperatures quicken the stiffening and setting of the concrete. Thus, higher temperature results in shorter setting time. On the other hand, low temperatures have the opposite effect. The ambient temperature will affect the concrete temperature; therefore, one would use a slower rate of placement in the winter and a higher rate of placement in summer.

Method of discharge

Discharging concrete from a height causes a surge or impact loading, which exerts more pressure than a discharge closer to the form.

Height of Concrete Pour (lift) H (m)

The height of concrete pour means the total height of the concrete element in which concrete is being placed during one pour. This is measured in m. The relationship of concrete pressure with height and unit weight is given below:

$$\textit{Concrete pressure} = \textit{Unit weight} \times \textit{Height} \tag{3.1}$$

The concrete formwork pressure would get affected if the concrete is not achieving initial set between lifts.

Minimum dimension of the section cast, d (mm)

In theory, the minimum dimension of the section cast (for example, wall thickness in case of concrete wall) does not affect the concrete formwork pressure. However, a narrow wall may produce lower pressures due to bridging effects.

Method of consolidating the concrete

Any consolidation method, internally or externally, should be done by the lift. Disturbing the lift will affect the set-up of previous lifts, increasing the unset height of the concrete, creating greater formwork pressure. High frequency vibration has a tendency to keep the freshly placed concrete in fluid state, so that it behaves as a liquid and thus full hydrostatic pressure is expected. In reality, however, the full hydrostatic pressure may or may not develop depending on whether stiffening or arching of concrete occurs before the lift is finished.

Out of the various factors discussed above, the factors: density of concrete, vibration, size of member being formed, temperature of concrete, rate of concrete placement, slump of concrete, superplasticizer, fly ash, slag cement, admixture and pumped concrete can be considered significant to the lateral pressure problem.

3.4.1.2 CIRIA method of calculating lateral pressure

Concrete pressure on formwork can also be based on CIRIA (Construction Industry Research and Information Association) report. CIRIA has developed a formula. According to this, the lateral pressure $P_{\max}$ is the smaller of the values obtained from the following two expressions:

$$P_{\max} = D \times \left[C_1 \times \sqrt{R} + C_2 \times K\sqrt{H - C_1\sqrt{R}} \right] \tag{3.2}$$

$$P_{\max} = D \times \mathrm{h} \tag{3.3}$$

P_{max} = Maximum concrete pressure on formwork, kN/m^2;

C_1 = Coefficient dependent on the size and shape of formwork;

C_2 = Coefficient dependent on constituent material of concrete;

D = Weight density of concrete kN/m^3;

H = Vertical form height, m;

h = Vertical pour height, m;

K = Temperature coefficient given by the following expression:

$$K = \left(\frac{36}{T+16}\right)^2; \tag{3.4}$$

R = Rate at which the concrete rises vertically up the form, m/h;

T = Concrete temperature at placing,°C.

When $C_1 \times \sqrt{R} > H$, fluid pressure Dh should be taken as design pressure. The term $C_1 \times \sqrt{R}$ incorporates the effect of vibration and workability. The term $C_2 \times K\sqrt{H - C_1\sqrt{R}}$ incorporates the effects of height of discharge, cement type, admixture and concrete temperature. Value of coefficient C_1 is taken as 1 for walls and 1.5 for columns. Value of coefficient C_2 is given in the Table 3.1.

Table 3.1 Value of C_2 in CIRIA Formula

Concrete with	C_2
OPC, RHPC or SRPC without admixture	0.30
OPC, RHPC or SRPC with admixture, except retarder	0.30
OPC, RHPC or SRPC with retarder	0.45
LHPBC, PBFC, PPFAC, or blends containing less than 70% GGBFS or 40% PFA with admixture, except a retarder	0.45
LHPBC, PBFC, PPFAC, or blends containing less than 70% GGBFS or 40% PFA with a retarder	0.45
LHPBC, PBFC, PPFAC, or blends containing less than 70% GGBFS or 40% PFA with a retarder	0.60
Blends containing more than 70% GGBFS or 40% PFA	0.60

Notes:

OPC – Ordinary Portland Cement

LHPBC – Low-Heat Portland Blast Furnace Cement

PBFC – Portland Blast Furnace Cement

RHPC – Rapid Hardening Portland Cement

PPFAC – Portland Pulverized Fuel Ash Cement

SRPC – Sulphate Resisting Portland Cement

GGBFS – Ground-Granulated Blast Furnace Slag Cement

PFA – Pulverized Fuel Ash

Concrete pressure on formwork can be calculated based on CIRIA report 108. Separate values on the basis of dimension of section to be cast e.g. walls and bases and columns have been provided by CIRIA, which is produced in the Tables 3.2 through 3.7. For the purpose of the use of the table, wall or base is defined as a section where at least one of the plan dimensions is greater than 2 m, while column has been defined as a section where both plan dimensions are less than 2 m. In order to calculate the pressure on formwork, we need to know the concrete group, concrete temperature, form height and rate of rise.

Table 3.2 Design Formwork Pressure for Columns for Concrete Groups: (1) OPC, RHPC or SRPC without admixture and (2) OPC,RHPC or SRPC with any admixture except a retarder

	Concrete temp. (10°C) Rate of rise (m/h)				Concrete temp. (15°C) Rate of rise (m/h)				Concrete temp. (20°C) Rate of rise (m/h)				Concrete temp. (25°C) Rate of rise (m/h)			
Form ht (m)	0.5	1.0	2.0	3.0	0.5	1.0	2.0	3.0	0.5	1.0	2.0	3.0	0.5	1.0	2.0	3.0
3	48	57	69	77	42	52	65	74	38	49	62	72	36	46	61	71
4	53	63	76	85	46	56	70	80	41	51	66	77	38	49	63	75
6	61	71	85	95	51	61	76	87	45	56	71	82	41	52	67	79

Table 3.3 Design Formwork Pressure for Columns for Concrete Groups: (1) OPC,RHPC or SRPC with a retarder, (2) LHPBFC,PBFC,PPFAC or blends containing less tthan 70% GGBFS or 40% PFA without admixture, (3) LHPBFC,PBFC,PPFAC or blends containing less than 70% GGBFS or 40% PFA with any admixture, except a retarder.

	Concrete temp. (10°C) Rate of rise (m/h)				Concrete temp. (15°C) Rate of rise (m/h)				Concrete temp. (20°C) Rate of rise (m/h)				Concrete temp. (25°C) Rate of rise (m/h)			
Form ht (m)	0.5	1.0	2.0	3.0	0.5	1.0	2.0	3.0	0.5	1.0	2.0	3.0	0.5	1.0	2.0	3.0
3	59	66	75	78	50	58	70	78	44	53	66	75	40	50	64	73
4	66	74	86	94	55	64	77	86	48	57	71	81	43	53	68	78
6	77	87	99	109	63	72	86	97	54	64	78	89	48	58	73	84

Table 3.4 Design Formwork Pressure for Columns for Concrete Groups: (1) LHPBFC,PBFC,PPFAC or blends containing less than 70% GGBFS or 40% PFA with a retarder, (2) Blends containing more than 70% GGBFS or 40% PFA

	Concrete temp. (10°C) Rate of rise (m/h)				Concrete temp. (15°C) Rate of rise (m/h)				Concrete temp. (20°C) Rate of rise (m/h)				Concrete temp. (25°C) Rate of rise (m/h)			
Form ht (m)	0.5	1.0	2.0	3.0	0.5	1.0	2.0	3.0	0.5	1.0	2.0	3.0	0.5	1.0	2.0	3.0
3	69	76	78	78	57	65	75	78	49	58	70	77	44	54	66	75
4	79	86	96	103	64	72	84	92	54	64	77	86	48	58	72	82
6	94	102	114	123	74	84	97	106	62	72	86	96	54	65	79	90

Table 3.5 Design Formwork Pressure for Walls for Concrete Groups: (1) OPC, RHPC or SRPC without admixture and (2) OPC, RHPC or SRPC with any admixture, except a retarder

	Concrete temp. (10°C) Rate of rise (m/h)				Concrete temp. (15°C) Rate of rise (m/h)				Concrete temp. (20°C) Rate of rise (m/h)				Concrete temp. (25°C) Rate of rise (m/h)			
Form ht (m)	0.2	0.5	1.0	1.5	0.2	0.5	1.0	1.5	0.2	0.5	1.0	1.5	0.2	0.5	1.0	1.5
2	30	35	41	45	25	30	37	41	21	27	34	39	19	25	32	37
3	36	41	47	52	28	34	41	46	24	30	37	42	21	27	35	40
4	40	46	52	57	31	37	44	49	26	33	40	45	23	29	36	42
6	47	53	53	65	36	43	50	55	30	36	43	49	26	32	39	45

Table 3.6 Design Formwork Pressure for Walls for Concrete Groups: (1) OPC,RHPC or SRPC with a retarder, (2) LHPBFC,PBFC,PPFAC or blends containing less than 70% GGBFS or 40% PFA without admixture, (3) LHPBFC,PBFC,PPFAC or blends containing less than 70% GGBFS or 40% PFA with any admixture, except a retarder.

	Concrete temp. (10°C) Rate of rise (m/h)				Concrete temp. (15°C) Rate of rise (m/h)				Concrete temp. (20°C) Rate of rise (m/h)				Concrete temp. (25°C) Rate of rise (m/h)			
Form ht (m)	0.2	0.5	1.0	1.5	0.2	0.5	1.0	1.5	0.2	0.5	1.0	1.5	0.2	0.5	1.0	1.5
2	40	44	48	52	31	36	42	46	26	32	38	42	23	29	35	40
3	47	52	58	62	37	45	48	53	30	36	43	47	26	32	39	44
4	54	59	65	69	41	47	53	58	34	40	46	51	29	35	42	47
6	64	70	76	81	49	55	61	66	39	45	52	57	33	39	46	52

Table 3.7 Design Formwork Pressure for Walls for Concrete Groups: (1) LHPBFC,PBFC, PPFAC or blends containing less than 70% GGBFS or 40% PFA with a retarder, (2) Blends containing more than 70% GGBFS or 40% PFA

	Concrete temp. (10°C) Rate of rise (m/h)				Concrete temp. (15°C) Rate of rise (m/h)				Concrete temp. (20°C) Rate of rise (m/h)				Concrete temp. (25°C) Rate of rise (m/h)			
Form ht (m)	0.2	0.5	1.0	1.5	0.2	0.5	1.0	1.5	0.2	0.5	1.0	1.5	0.2	0.5	1.0	1.5
2	49	52	52	52	38	42	47	50	31	36	42	46	27	32	38	42
3	59	61	68	72	45	50	56	60	37	42	48	53	31	37	43	48
4	68	73	78	82	51	57	62	67	41	47	53	58	34	40	47	52
6	82	87	93	97	61	67	73	78	48	54	61	66	40	46	53	58

3.4.1.3 ACI formula

For Column

The ACI formula for computing the lateral pressure for a column is given below:

$$P = C_w \times C_c \times \left(7.2 + \frac{785R}{T + 17.8}\right) \tag{3.5}$$

The maximum value of pressure (in kN/m^2) calculated from Eq. (3.5) is limited to that given by the following expression:

$$P = 150 \times C_w \times C_c \tag{3.6}$$

The minimum value of pressure (in kN/m^2) calculated from Eq. (3.5) is limited to that given by the following expression:

$$P_{\min} = 30 \times C_w \tag{3.7}$$

The pressure (in kN/m^2) computed from Eqs. (3.5) and (3.6) can in no case be greater than that computed by the following expression:

$$P_{\max} = w \times h \tag{3.8}$$

For Wall

The ACI formula for computing the lateral pressure for a wall is given below:

$$P = C_w \times C_c \times \left(7.2 + \frac{1{,}156}{T + 17.8} + \frac{244R}{T + 17.8}\right) \tag{3.9}$$

The maximum value of pressure (in kN/m^2) calculated from Eq. (3.9) is limited to that given by the following expression:

$$P = 100 \times C_w \times C_c \tag{3.10}$$

The minimum value of pressure (in kN/m^2) calculated from Eq. (3.9) is limited to that given by the following expression:

$$P_{\min} = 30 \times C_w \tag{3.11}$$

The pressure (in kN/m^2) computed from Eqs. (3.9) and (3.10) can in no case be greater than that computed by the following expression:

$$P_{\max} = w \times h \tag{3.12}$$

In the ACI formulae,

P = Lateral pressure in kN/m^2;

h = Depth of plastic concrete from the top of placement to the point under consideration (in m);

w = Unit weight (in kN/m^3);

R = Rate of displacement (in m/h);

T = Temperature of concrete during placement;

C_w = Unit weight coefficient (as per Table 3.8);

C_c = Chemistry coefficient (as per Table 3.8).

3.4.1.4 DIN standard method for calculating lateral pressure

DIN 18218 presents a series of equations (see Eqs. 3.13 to 3.15) to calculate the limiting lateral pressures of internally vibrated concrete of various mobilities at a concrete temperature of 15°C. The rate of concrete placement and the concrete temperature at placing are the essential factors to calculate lateral pressure:

Table 3.8 Values of C_c and C_w

Types of blends used	Chemistry coefficient (C_c)	Unit weight coefficient (C_w)
Blends without retarders containing less than 70% fly ash	1.2	Concrete weighing less than 22.5 kN/m^3 $C_w = \left(1 - \frac{w}{23.2}\right)$ but less than 0.8
Blends without retarders containing less than 70% slag or 40% fly ash	1.4	Concrete weighing 22.50 – 24.00 kN/m^3 $C_w = 1$
Blends containing more than 70% slag or 40% fly ash	1.4	Concrete weighing more than 24 kN/m^3 $C_w = \frac{w}{23.2}$

$$P_{\max} = 21 + 5R \text{ (kPa) for stiff mix} \tag{3.13}$$

$$P_{\max} = 19 + 10R \text{ (kPa) for soft mix} \tag{3.14}$$

$$P_{\max} = 18 + 14R \text{ (kPa) for fluid mix} \tag{3.15}$$

To adjust for concrete temperatures other than 15°C, the limiting pressure must be decreased by 3% for every degree above 15°C and increased by 3% for every degree below 15°C.

3.4.1.5 IS code method for calculating the form pressure

The lateral pressure computation according to the Indian standard IS: 14687–1999 assumes that the lateral pressure due to fresh concrete is dependent on the temperature of concrete as placed, the rate of placing of concrete, and the concrete mix proportion.

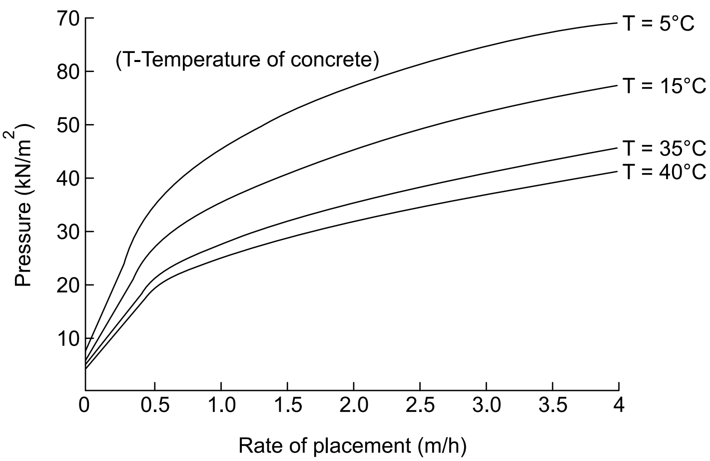


Figure 3.3 Maximum Pressure on Formwork due to Fresh Concrete.

The code outlines a method for calculating the formwork pressure exerted by concrete. It recommends a set of curves for lateral pressure computation (see Fig. 3.3). The parameters affecting the value of lateral pressure according to IS code are predominantly: the rate of placement (height) in m/h; temperature of concrete in degrees Celsius; and the concrete mix proportion. Figure 3.3 is valid for medium degree of workability, and a mix design with cement content of 350 kg/m^3, 33 grade ordinary Portland cement without any addition of admixture. For the pressure calculation of concrete mixes made with different types of cement and workability, correction factors are to be applied as specified in IS: 14687–1999. The different correction factors are discussed in the following sections:

(1) Correction for workability

The correction factors for different degrees of workability of concrete are given in Table 3.9.

Table 3.9 Correction Factors for Different Degrees of Workability of Concrete

Degree of workability	Rate of placement of concrete (m/h)		
	Up to 1	1.5-2	2.5-4
Very low	0.70	0.75	0.80
Low	0.80	0.85	0.90
Medium	1.00	1.00	1.00
High	1.10	1.30	1.50

(2) Cement content

A correction factor for every increase in cement content value beyond 350 kg/m^3 is to be applied. According to the standard for every 50 kg increase in cement content, the rate of placement of concrete may be reduced by 0.5 m/h for obtaining the correction to the pressure.

(3) Density of concrete

The set of curves produced in Fig. 3.3 are based on concrete density of 24 kN/m^3. For other densities the values of $P_{\max}$ shall have to be pro-rated.

(4) Type of cement

For cement other than 33 grade ordinary Portland cement, the code recommends suitable correction factor in the $P_{\max}$. The value of $P_{\max}$ can be increased or decreased depending on the relative setting times of the concrete.

(5) Admixture

Figure 3.3 is valid for concrete without the use of any admixture. In case admixtures are proposed to be used, suitable correction factors to $P_{\max}$ may be applied. The correction factors may be computed on the basis of trials conducted or manufacturers' supplied data.

The pressure distribution along the height of formwork can be assumed as given in Fig. 3.4.

For normal concrete, the maximum pressure may occur at a height h_m below the top as given by the following formula:

$$h_m = \frac{P_{\max}}{d} \tag{3.16}$$

where h_m is in m, $P_{\max}$ is in kN/m^2, and d is the density of fresh concrete in kN/m^3.

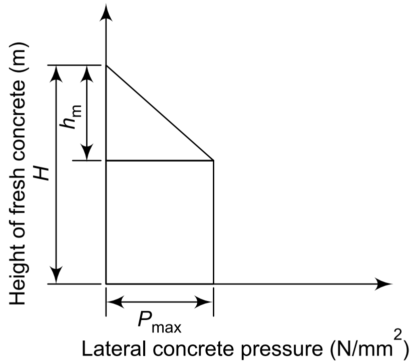


Figure 3.4 Pressure Distribution.

The pressure exerted on back form (that is top form on inclined surfaces) can uplift the formwork. Such situations should be designed and detailed for anchorage and pressure containment without movements.

3.4.1.6 Form pressure on self compacting concrete (SCC)

Formwork pressure represents the perpendicular load of the fresh concrete on the formwork surface. The formwork has to stabilize the concrete from the beginning of the casting approximately until the end of setting. Parameters influencing the pressure on formwork are:

1. Velocity of the casting (Rising);
2. Total height;
3. Consistency;
4. Concrete density;
5. Geometry of the mould;
6. Compaction technique;
7. Degree of reinforcement;
8. Mix design.

Initially SCC was considered as a liquid for determining the pressure on the formwork and was taken as γH (see Fig. 3.5).

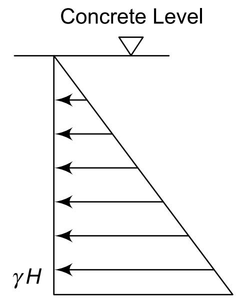


Figure 3.5 Pressure Distribution for Liquid.

But it was found that the formation of initial structure will reduce some pressure on the formwork. The new calculation concept for the formwork pressure was derived from the theory by Janssen.

Experiments were conducted by Carl and Darmstadt on 13 slender columns of 300 mm × 300 mm and of 4 m height, with 10 reinforced columns into three series wherein the influence of the rising velocity, the slump flow and the reinforcement and the pressure with conventional vibration were investigated. The result showed that the column with reinforcement exerted less pressure on the formwork compared to the other.

During casting, the majority of the SCC that is already in the formwork does not move anymore. Therefore, the particles in the motionless layers are able to coagulate and build up an internal structure. Consequently, the resulting pressure using a slow casting speed is relatively low. Regarding the mix design, the use of organic and inorganic admixtures can have an influence on the pressure as they affect cement hydration and thixotropy. If external energy is added, the internal structure of SCC is destroyed, leading to an immediate increase in formwork pressure.

3.4.1.7 Form pressure for inclined formwork

In some special cases where the formwork is inclined, the following formula can be used for computation of lateral pressure.

(a) When only lateral pressure acts,

$$P = w \times h \times \sin^2(\phi) \tag{3.17}$$

(b) When the weight of concrete also acts,

$$P = 2{,}500 \times t \times \cos\phi + w \times h \times \sin^2(\phi) \tag{3.18}$$

where

P = Lateral pressure;

ϕ = Angle of formwork to horizontal;

h = Vertical depth;

w = Pressure acting normal to the slope;

t = Thickness of slab.

3.4.2 Loads From Construction Personnel, Plant and Equipment, Vibration and Impact of Machine Delivered Concrete

Loads during constructional operation shall constitute the imposed loads [see IS: 875 (Part 2)–1987] for falsework design. Such loads may occur due to the construction personnel, plant and equipments, vibration and the impact of machine delivered concrete, lateral pressure of fresh concrete, unsymmetrical placement of concrete, concentrated load and the storage of construction materials.

Such loads are computed on the basis of recommendations given in IS: 875 (Part 2)–1987. The value so selected from the code needs to be corrected if:

1. Concrete is dropped from a height of more than 1.1.m and;
2. Concrete is accumulated for more than three times the depth (thickness) of concrete on the formwork surface with a limit in area of 1 m^2 for any such situation to this height.

If it is necessary to exceed the above limitations, the code recommends allowances for the additional loading in design. Where the allowance has only to be made for access and inspection purposes, the code recommends a loading of 750 N/m^2.

The code further recommends that imposing of any construction load on the partially constructed structures shall not be allowed unless specified in the drawings or approved by the engineer-in-charge. Allowance shall be made in the falsework design to accommodate force or deformation in the post-tensioned members.

The code recommends that the load from the permanent works shall be assessed from the self-weight of the permanent structure to be supported by the formwork, including the weight of plastic concrete which may actually be determined or taken as per IS: 875 (Part 1)–1987. The effect or impact of the surge wherever it may occur, should be suitably considered and catered for. Where pumping is resorted to, additional loads should be considered in the design.

According to ACI 347 recommendations, the formwork should be designed for a live load of not less than 2.4 kN/m^2 of horizontal projection. It further recommends that when the motorized carts are to be used, the live load should not be less than 3.6 kN/m^2. The design load for combined dead and live loads should not be less than 4.8 kN/m^2 or 6.0 kN/m^2 if motorized carts are used.

BOX 3.2: Extract of Imposed Loads on Formwork

Sl. No.	Load type	Suggested value of loading
2a	Lateral pressure of fresh concrete	Based on IS: 14687–1999
2b	Loads from construction personnel, plant and machineries, vibration and impact from machine delivered concrete	Can be taken from IS: 875 (Part 2)–1987
2c	Unsymmetrical placement of concrete	Can be taken from IS: 875 (Part 2)–1987
2d	Concentrated load and storage of construction materials	Can be taken from IS: 875 (Part 2)–1987

3.5 ENVIRONMENTAL LOADS

The environmental load on formwork consists of (a) loads due to wind, (b) seismic loads, (c) earth pressure, (d) water pressure, (e) snow and ice load, (f) thermal load, and (g) miscellaneous loads.

3.5.1 Wind Load

Wind load acting on formwork is treated similarly to wind load acting on permanent structures and is determined according to the pertinent Indian Standard. This is the approach used also by other codes such as ACI and so on. Other horizontal loads are considered similar to ACI guidelines as well. It is worth noting that, according to Chen and Mosallam (1991), ACI requirements with regard to horizontal loads seem to be largely a judgment with no documentation confirming the value; furthermore, wind loads may be more important for falsework than in permanent structures.

Bracings and props should be designed for all foreseeable horizontal loads due to wind, inclined supports, dumping of concrete and equipments. However, in no case should this horizontal load acting in any direction at each floor line be less than 150 kg/linear meter of slab edge or 2 % of the total dead load on the form distributed as a uniform load per linear meter of slab edge, whichever is greater.

Wall forms should be designed to meet the wind load requirements of the local building code. Bracing for wall forms should be designed for a horizontal load of at least 150 kg per linear meter of the wall, applied at the top.

All the loads mentioned above may not be of appreciable magnitude in all the cases, but in some specific circumstances, some of the loads may prove to be predominant.

Indian Standards recommend that the formwork should be designed in such a manner that the vertical members are subjected to compressive force only under the action of combined horizontal and vertical loads.

Special loads

Consideration is given to the influence of all foreseeable special loads during the construction.

Generally, the weight of the concrete with reinforcement can be assumed as 25 kN/m^3. Self-weight of the formwork, for ordinary structures, varies between 0.50 kN/m^2 and 0.75 kN/m^2.

BOX 3.3: Extract of Environmental Loads on Formwork

Sl. No.	Load type	Suggested value of loading
3a	Wind loads	Wind loads should be taken for design in accordance with IS: 875 (Part 3)–1987 subject to a minimum horizontal load equal to 3 percent of the vertical loads at critical level.
3b	Earthquake load	According to relevant Indian Standards.
3c	Snow load	Snow loads should be assumed in accordance with IS: 875 (Part 4)–1987.
3d	Ice load	Ice loads are required to be taken into account in the design of members of formwork in zones subjected to ice formation. The thickness of ice deposits may be taken to be between 3 mm and 10 mm depending upon the locations of the formwork. The maximum density of ice may be assumed to be 900 kg/m^3.
3e	Earth pressure	Earth pressure can occur on falsework as in the case of retaining walls and these shall be catered for. The rise in the water table may increase pressure on the falsework.
3f	Thermal load	Shrinkage and early thermal movements in the freshly placed concrete should be assessed and accommodated in the design of formwork.

3.6 THE DESIGN BASIS (ASSUMPTIONS MADE IN FORMWORK DESIGN)

Some of the requirements to be fulfilled by any formwork design are general in nature. For example, formwork needs to be designed in such a way that it is able to produce accurate configuration and position of the concrete cast; it should be easy to erect and remove the formwork; joints should be able to be tightly done; provisions to incorporate chamfers at all corners should be possible; the design should also have provisions for temporary openings for the convenience of cleaning the forms, inspection, and the placement of concrete.

Until and unless the concrete structure is of major importance, very accurate design is unnecessary as too much refinement wastes time without giving results commensurate with the efforts. Absolute precision is unwarranted when so many assumptions have to be made both for analysis and design purposes; for example assumptions about loads, lateral pressures, quality of materials, workmanship at site, and other factors. Hence, the following simplifications can be done for computations of bending moment, shear force, and deflection:

- All loads are assumed as uniformly distributed.
- Beams supported over three or more spans are regarded as continuous and approximate formulas given elsewhere are used.

- The stresses induced in every member of formwork, in bending, in shear and in bearing, should be within the permissible working stress for that material.
- Forms must be so designed that the various parts will not deflect beyond the prescribed limits. The permissible deflection depends on the desired finish as well as the location.
- The strength of nails is neglected in determining the size of main formwork. This does not apply when considering splices, braces, and brackets.

In the absence of detailed specification, acceptable and frequently used values of permissible deflections can be taken as given in Table 3.10.

Table 3.10 Permissible Deflection for Guidance

Sl. No.	Type of member	Permissible deflection
1	For sheathing	1.6 mm
2	For members up to 1.5 m	3.0 mm
3	For members spanning more than 1.5 m	6.0 mm or span/360, or span/270

According to IS: 14687–1999, the deflection permitted in falsework inclusive of the initial imperfections shall not exceed 3 mm in case of beam span less than 3 m, and it should not exceed the least of 3 mm and L/1000 in case of beam span more than 3 m.

In order to translate the design into implementation at site, formwork drawings should necessarily be produced which contain the following information:

1. The assumptions made in the design, for example the concrete temperature, slump, rate of rise of concrete in the form, etc;
2. The types of materials, sizes, lengths, and the connection details;
3. The sequence of removal of forms and shores;
4. The details of anchors, form ties, shores, and braces;
5. The field adjustment of the form during placing of concrete;
6. The working scaffolds and gangways;
7. The details of weep holes, vibrator holes, or access doors for inspection and placing of concrete;
8. The details of construction joints and expansion joints;
9. The sequence of concrete placements and minimum/maximum elapsed time between adjacent placements;
10. The details of chamfer strips or grade strips for exposed corners and construction joints;
11. The foundation details for the shoring;
12. Special provisions if any, such as protection from flood water, ice, and debris at stream crossings;
13. The details of form coatings and release agents;
14. The means of obtaining specified concrete;
15. The location of box outs, pipes, ducts, conduits and miscellaneous inserts in the concrete, attached to or penetrating the forms;
16. The location of spacing of rubber pads where shutter vibrations are used.

3.7 ESTIMATING PERMISSIBLE STRESSES

Permissible stresses should not exceed the values specified in the relevant Indian Standards for permanent structures. In case of reusable components of steel, timber, etc., the values of permissible stresses shall be suitably reduced depending upon the number of uses and the extent of deterioration.

(a) *Timber:* Basic permissible stresses of different species of timber selected out of the timbers listed in IS: 399–1963 shall be taken in accordance with the stresses given in IS: 883–1994.

(b) *Plywood:* Maximum permissible stresses and modulus of elasticity shall be in accordance with the provisions of IS: 4990–1993.

(c) *Steel: The* permissible stresses shall be assumed as given in IS: 800–2007 and IS: 2750–1964, as applicable.

(d) *Tubular Section:* The permissible stresses shall be assumed in accordance with IS: 806–1968. In case of reused steel tubes, the permissible compressive stresses may be reduced by 15 %, provided the maximum reduction in nominal mass (see IS: 1161–1998) is 7.5 percent and the deviation in length is not more than l/600 of the length.

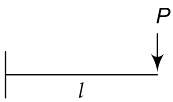
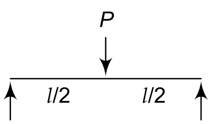
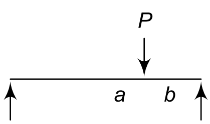
(e) *Brickwork-Stone*: The properties of brickwork, stone masonry and block work shall be as per IS: 1905–1987, IS: 1597 (Part 1)–1992 and IS: 2212–1991.

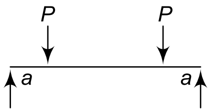
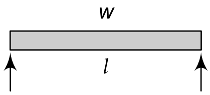
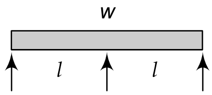
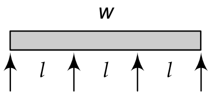
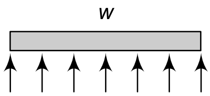
(f) *Concrete:* The concrete should in general comply with the requirements of IS: 456–2000 as appropriate to a concrete member. Blinding concrete, where used, shall have a minimum thickness of 50 mm of grade M10. If concrete of lower strength is used, minimum thickness shall be 75 mm.

3.8 MAXIMUM BENDING MOMENT, SHEAR FORCE, AND DEFLECTION

For the ready reference of the readers, the expressions of bending moment, shear force, and deflection for common loading and support conditions are provided in Table 3.11.

Table 3.11 Maximum Bending Moment, Shear Force, and Deflection for Varying Loading and Support Conditions

Sl. No.	Loading condition	Bending Moment	Shear Force	Deflection
1.		$M = Pl$	$V = P$	$\delta = \dfrac{Pl^3}{3EI}$
2.	 For simply supported beam with concentrated load P at its centre	$M = \dfrac{Pl}{4}$	$V = \dfrac{P}{2}$	$\delta = \dfrac{Pl^3}{48EI}$
3.	 Span = $l = a + b$	$M = \dfrac{Pab}{l}$	$V = \dfrac{Pa}{l}$	$\delta = \dfrac{Pb}{EI} \times \left(\dfrac{l^2}{16} - \dfrac{b^2}{12}\right)$

Sl. No.	Loading condition	Bending Moment	Shear Force	Deflection
4.	 Two point loads of magnitude *P* each. Span = *l*	$M = Pa$	$V = P$	$\delta = \frac{Pa}{6EI} \times \left(\frac{3l^2}{4} - a^2\right)$
5.	 For simply supported beam with U.D.L. Span of length *l*	$M = \frac{wl^2}{8}$	$V = \frac{wl}{2}$	$\delta = \frac{5wl^4}{384EI}$
6.	 Two span uniformly supported beam with U.D.L. Equal span of length *l*	$M = \frac{wl^2}{8}$	$V = \frac{5wl}{8}$	$\delta = \frac{wl^4}{185EI}$
7.	 For continuous beam with U.D.L. over its full length. Equal span of length *l*.	$M = \frac{wl^2}{10}$	$V = \frac{5wl}{8}$	$\delta = \frac{wl^4}{145EI}$
8.	 Continuous beam more than 3 spans with U.D.L. over its full length. Span of length *l*.	$M = \frac{wl^2}{10}$	$V = \frac{5wl}{8}$	$\delta = \frac{wL^4}{145EI}$
		As can be noted from above, the expression remains the same for more than three equally spaced supports.		

SOLVED EXAMPLES

Example 1 *Estimate the total load exerted on the formwork of a RCC slab of thickness 250 mm. Assume a live load of 2.4 kN/m² and self-weight of forms as 0.4 kN/m².*

Solution

$$\begin{aligned} \text{Dead load, concrete and rebar} &= 26 \times 0.250 = 6.5 \text{ kN/m}^2 \\ \text{Construction live load} &= 2.4 \text{ kN/m}^2 \\ \text{Weight of forms} &= 0.4 \text{ kN/m}^2 \\ \text{Total form design load} &= 9.3 \text{ kN/m}^2. \end{aligned}$$

Example 2 *Calculate the lateral pressure exerted by concrete, for a bridge abutment section of dimension 0.8 × 6.0 × 5 m high, proposed to be cast with OPC normal weight concrete, and 10°C temperature at placing. The proposed rate of placement is 24 m³/h and is to be pumped.*

SOLUTION

$$\text{Rate of rise} = \frac{24}{0.8 \times 6} = 5 \text{ m/h}$$

As per CIRIA report 108, P_{max} at height= 4 m = 75 kN/m^2

P_{max} at height = 6 m = 85 kN/m^2

Therefore by interpolating we get, P_{max} at height = 5 m = 80 kN/m^2.

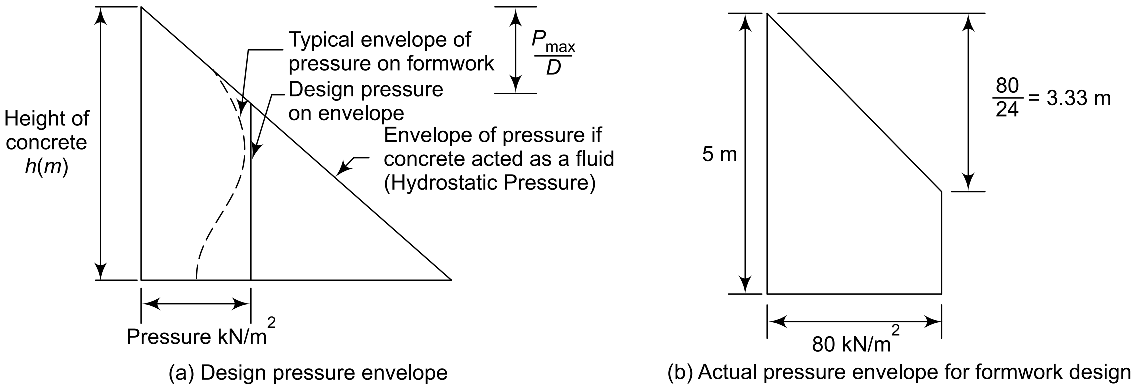


Figure S 3.2.1 Pressure Envelopes.

Please also note that the maximum hydrostatic pressure if concrete acted as a fluid equals $5 \times 24 = 120$ N/m^2 (where 5 is height, and 24 is concrete density in kN/m^3). Therefore, actual maximum pressure (80 kN/m^2) is less than hydrostatic maximum pressure 120 kN/m^2. The design pressure envelope and actual pressure envelope for formwork design are shown in Fig. S 3.2.1.

EXAMPLE 3 *Find the lateral pressure using ACI formula for the wall of size 15 m × 3 m × 0.2 m. Assume rate of pour is 10 m^3/h and ambient temperature as 20°C. The product of C_w and C_c may be considered equal to 1.*

SOLUTION

Pour size = 15 m × 3 m × 0.2 m = 9 m^3

Rate of Pour = 10 m^3/h

Time required for concreting = 9/10

= 0.9 h (say, 1 h)

Therefore, rate of rise = 3 m/h

As per ACI Committee report,

Lateral pressure, $$P = C_w C_c \left[7.2 + \frac{1{,}156}{T + 17.8} + \frac{244R}{T + 17.8}\right]$$

Given $C_w C_c = 1$

$$P = 1 \times \left[7.2 + \frac{1{,}156}{20 + 17.8} + \frac{244 \times 3}{20 + 17.8}\right]$$

$$P = 57.15 \text{ kN/m}^2$$

Example 4 *For the data given below, compute the lateral pressure on formwork as per CIRIA formula and also draw the design pressure distribution.*

D (weight density of concrete)	25 kN/m^3
C_1 (shape constant)	1
R, rate of rise	1 m/h
C_2, concrete constituent factor	0.3
Temperature of concrete (°C)	25
Temperature co-efficient	0.77
H, form height	6.15 m
h, pour height	6 m

Solution

Lateral concrete pressure (as per CIRIA formula)

$$P_{max} = D\left[C_1\sqrt{R} + C_2K\sqrt{H - C_1\sqrt{R}}\right]$$

$$P_{max} = 25 \times \left[1 \times \sqrt{1} + 0.3 \times 0.77 \times \sqrt{6.0 - 1 \times \sqrt{1}}\right]$$

$P_{max} = 38.12$ kN/m^2

$D \times h = 25 \times 6.0 = 150$ kN/m^2

Thus, lateral concrete pressure is the minimum of the above two values = 38.12 kN/m^2

$P_{max}/D = 38.12/25 = 1.52$ m

The design pressure distribution is thus as shown in Fig. S 3.4.1.

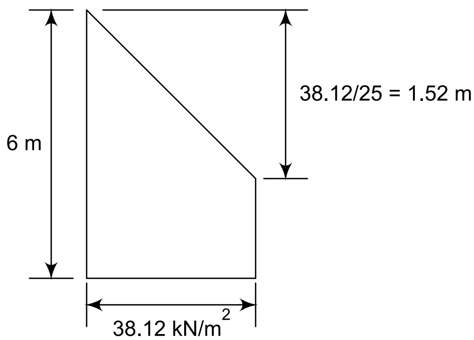


Figure S 3.4.1 Pressure Distribution Diagram for Example 4

Example 5 *Draw the design pressure distribution for a wall of thickness 10.5 m, length of 10 m and height of 6 m. It is proposed to use OPC concrete with retarder. The concrete temperature at the time of placing is 10°C. The concrete pour speed is 15 m^3 per hour.*

Solution

The total concrete quantity to be poured = 0.5 m × 10 m × 6 m = 30 m^3

Estimated time for the concrete pour = 30/15 = 2 hours

Thus the rate of rise R = 6 m/ 2 hours = 3 m/h

$$P_{max} = 90 \text{ kN/m}^2$$
$$h_m = 90/25 = 3.6 \text{ m}$$

The design pressure distribution is shown in Fig. S 3.5.1.

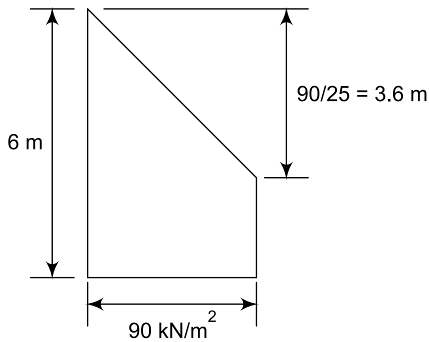


Figure S 3.5.1 Design Pressure Distribution for Example 5

Example 6 *Draw the design pressure distribution for a column of dimension 0.5 m (width) × 1.75 m (length) × 10.0 m (height). It is proposed to use OPC concrete without retarder. The concrete temperature at the time of placing is 10°C. The concrete pour speed is 8.75 m³ per hour. Assume a maximum pressure of 150 kN/m².*

Solution

The total concrete quantity to be poured $= 0.5 \text{ m} \times 1.75 \text{ m} \times 10 \text{ m} = 8.75 \text{ m}^3$

Estimated time for the concrete pour $= 8.75/8.75 = 1$ hour

Thus the rate of rise $R = 10 \text{ m}/ 1 \text{ hr} = 10 \text{ m/h}$

$$P_{max} = 150 \text{ kN/m}^2$$
$$h_m = 150/25 = 6 \text{ m}$$

The design pressure distribution is shown in Fig. S 3.6.1.

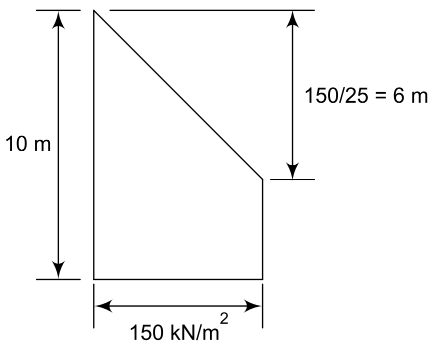


Figure S 3.6.1 Design Pressure Distribution for Example 6

Example 7 *Draw the design pressure distribution for a wall of thickness 0.2 m, length of 2.5 m and height of 3 m. It is proposed to use OPC concrete with retarder. The concrete temperature at the time of placing is 10°C. The rate of rise is 6 m/h and the maximum pressure is 75 kN/m².*

SOLUTION

$$P_{\max} = 75 \text{ kN/m}^2$$
$$h_m = 75/25 = 3.0 \text{ m}$$

Thus in this case, h_m and H both coincide. The design pressure distribution is shown in Fig. S 3.7.1.

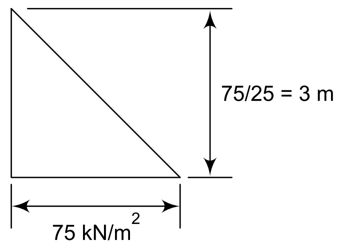


Figure S 3.7.1 Design Pressure Distribution for Example 7

EXAMPLE 8 *Compute the lateral pressure on formwork for a 3.0 m high wall where ordinary concrete is to be placed at 10°C progressively over a 1 hour period. The following inputs (see Table S3.8.1) may be taken for the design:*

Table S 3.8.1 Data for Example 8

C_1	Shape co-efficient	1.0
C_2	Material co-efficient	0.3
D	Concrete density	25 kN/m^3
H	Vertical height of form	3 m

SOLUTION

Since the wall of 3 m height is getting filled in one hour, the rate of rise of concrete can be taken as R = 3 m/h.

$$P_{\max} = D\left[C_1\sqrt{R} + C_2K\sqrt{H - C_1\sqrt{R}}\right] \quad (3.20)$$

For $T = 10°\text{C}, K = \left[\dfrac{36}{T+16}\right]^2 = 1.92$

The lateral pressure $P_{\max}$ according to CIRIA formula is the smaller of

$$P_{\max} = D\left[C_1\sqrt{R} + C_2K\sqrt{H - C_1\sqrt{R}}\right] \quad (3.20)$$

and Dh (3.21)

From Eq. (3.20),

$$P_{\max} = 25 \times \left[1 \times \sqrt{3} + 0.3 \times 1.92 \times \sqrt{3 - 1 \times \sqrt{3}}\right]$$
$$= 25 \times [1.73 + 0.649] = 59.5 \text{ kN/m}^2$$

From Eq. (3.21)

$$P_{\max} = Dh = 25 \times 3 = 75 \text{ kN/m}^2$$

Take the lesser value i.e. 59.5 kN/m^2.

The design pressure distribution is shown in Fig. S 3.8.1.

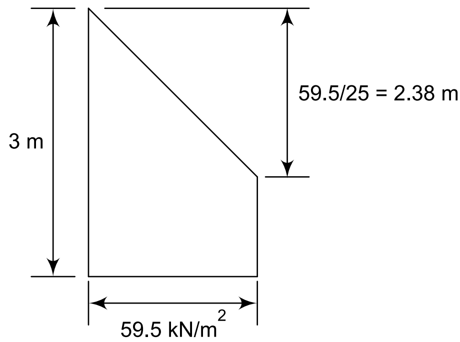


Figure S 3.8.1 Design Pressure Distribution for Example 8

REVIEW QUESTIONS

Q1. True or False

(a) Formwork design should not address safety, overturning, stability, and prevention of collapse.

(b) Parameters influencing the pressure on formwork are - velocity of casting, total height, consistency, concrete density, geometry of mould, and mix design.

Q2. Match the following in the context of formwork loads:

1. Dead load	(a) Wind load, seismic load, earth and water pressure, snow and ice load
2. Imposed load	(b) Self-weight, ancillary work, fresh concrete load
3. Environment load	(c) Lateral pressure, concentrated load, construction personnel load

Q3. Match the following:

1. Concrete pressure	(a) $\left(\dfrac{36}{T+16}\right)^2$
2. P_{max} (Maximum concrete pressure)	(b) Unit weight × height
3. Temperature coefficient K	(c) $D \times \left[C_1 \times \sqrt{R} + C_2 \times K\sqrt{H - C_1\sqrt{R}}\right]$

Q4. Match the following permissible deflections with guidance:

1. Sheathing	(a) 3 mm
2. Members up to 1.5 m	(b) 6 mm or span/360 or span/270 mm
3. Members spanning greater than 1.5 m	(c) 1.6 mm

Q5. Match the following:

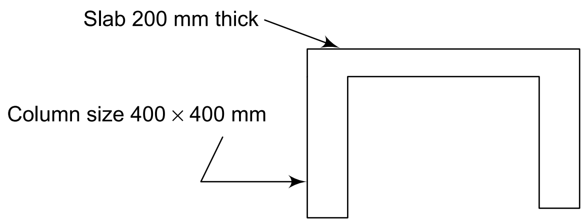
1. Formwork materials	(a) IS Codes for basic permissible stress
2. Timber	(b) IS: 456–2000

3. Plywood	(c) IS: 399–1963 and IS: 883–1994
4. Steel	(d) IS: 4990–1993
5. Tubular section	(e) IS: 1905–1987, IS: 1597 (Part 1)–1992, and
	(f) IS: 2212–1991
6. Brick work stone	(g) IS: 806–1968
7. Concrete	(h) IS: 800–2007 and IS: 2750–1964

Q6. List out various methods for calculating the lateral pressure of concrete.

Q7. Show the formwork arrangement through a neat sketch for a flat slab of thickness 200 mm supported on 400 × 400 mm *RC* column spaced at 4 m centre to centre, by calculating the spacing of primary and secondary timber beams.

Plywood of 12 mm thickness and timber of 50 × 100 mm, 100 × 100 mm, and 100 × 150 mm, are available. Maximum deflection allowed is 1.5 mm. If the capacity of shores is 30 kN, at what spacing will you place them? Assume the concrete density as 25 kN/m^3 and permissible stress in timber as 7 N/mm^2.



Q8. In a construction project, RCC boundary wall of length 120 m is to be built in 12 days time. The height of the wall is 2 m and its thickness is 150 mm. The following shuttering materials are available with the contractor:

Plywood 12 mm thickness, H-16 beam, steel waler of different sizes, tie rod, and alignment props.

The properties of these materials are given in Table Q3.8.1.

Table Q3.8.1 Data for Question 8

Material	Allowable BM	Allowable Shear	Allowable Deflection (cm)	Allowable EI
12 mm thick plywood	0.2 kNm/m	6.16 kN/m	Lesser of L/360 or 0.15	1.07 kNm^2/m
H-16 beam	3 kNm	6 kN	Lesser of L/360 or 0.6	145 kNm^2
Steel Waler	10.2 kNm	103.4 kN	Lesser of L/360 or 0.3	784.14 kNm^2

Assume 40 kN/ m^2 as the design formwork pressure. Tie rod capacity is 50 kN.

Use these formwork materials to prepare the desired formwork scheme economically. Support your formwork scheme with approximate design calculations and sketches. Justify the structural adequacy of the walers with detailed calculations. Calculate the maximum tie-rod forces.

Q9. Design the wall formwork for a wall having 6.0 m length and 3.8 m height. The maximum concrete pressure can be assumed to be 70 kN/m^2. The client's specification limits the deflection to 1/270 of the span of any formwork member. Through ties are allowed. Timber and tie rod of different sizes and plywood of 12mm thickness are available. Prepare a neat sketch showing the cross-section and spacing of different members.

Q 10. (a) Discuss the significance of the various terms and coefficients (C_1 and C_2) used in the CIRIA formula for the computation of lateral concrete pressure on wall forms.

(b) A concrete wall is to be cast with the use of a wall-form made with the following parts:

Walls: Thickness = 800 mm

Height = 3 m

Bay length = 9 m

Assume the wall is located in a sheltered position and no wind loadings need designing for.

Concrete = OPC with retarder

Density = 24.5 kN/m^3

Temperature at placing = 10°C

Uniform volume supply rate:

3 × (6 m^3 trucks) every hour

Using the formulae, determine the lateral concrete pressure distribution on the wall form.

Q11. Find the lateral pressure for the wall of size 15 m × 3 m × 0.25 m. Assume that the rate of pour is 10 m^3/h, ambient temperature is 20°C and slump is 75 mm.

Q12. Given H = 3 m, R = 5 m/h, d = 500 mm, t = 5°C and slump = 75 mm. Calculate the lateral pressure exerted by concrete on the formwork.

Q13. Given H = 6 m, R = 2 m/h, d = 500 mm, t = 20°C and slump = 50 mm. Calculate the lateral pressure exerted by concrete on the formwork.

Q14. Given H = 5 m, R = 2 m/h, d = 250 mm, t = 10°C and slump = 75 mm. Calculate the lateral pressure exerted by concrete on the formwork.

Chapter

4 Formwork for Foundations

Contents: Introduction; Conventional Formwork for Foundation; Foundation Formwork (All Steel); Foundation Formwork Design; Illustration on Foundation Wall Design

4.1 INTRODUCTION

In this chapter, we discuss the conventional and proprietary foundation formwork. Some real life illustrations of foundation formwork in use have also been provided. We discuss the different design steps needed to perform foundation formwork design. Some design aids for getting assistance in the foundation formwork design are also presented. Towards the end, we illustrate few foundation formwork design examples.

The selection of materials for foundation formwork depends on the size and type of foundation. The basic elements in footing formwork are: (1) sheathing materials, (2) stakes, (3) braces, (4) nails, (5) studs, (6) wales, (7) spreader, and (8) ties.

The sheathing material consists of mostly timber planks, plywood, and steel plates. If timber planks are used as sheathing material, their thickness is about 50 mm and width is the same as that of the depth of the footing. Thus, if depth of the footing is 300 mm, the width of the plank to be used is 300 mm. The plywood thickness used is between 12 mm and 19 mm. The steel plate used as sheathing material has a thickness of 3.15 mm to 5 mm and a commonly used size of 600 mm × 1,200 mm. One side of the steel plates is usually kept 600 mm while the other side is in the multiple of 300 mm. For example 600 mm × 300 mm, 600 mm × 600 mm, 600 mm × 900 mm, and 600 mm × 1,200 mm. The sheathing materials are held in place with stakes. The stakes and braces are commonly made up of 50 mm × 100 mm timber. The distance between sheathing materials is ensured with the help of spreaders or ties. The duplex-head nails are used for joining timber materials since they are easy to be removed during formwork removal. Short braces hold the stakes and are useful for alignment of the footing. Sometimes, the excavated earth is dumped outside of the form to prevent the wet concrete from running out.

Since footings are below the ground and are covered, appearance of the concrete surfaces is not that important in case of footing formwork. Thus, used formwork materials are preferable for footing formwork. However, in order to provide straight and smooth concrete surfaces, it is imperative that materials selected for footing formwork are of good quality and straight.

4.2 CONVENTIONAL FORMWORK FOR FOUNDATION

In practice, there are different types of foundations used for reinforced cement concrete construction works. For example: the wall foundation, individual column footings, stepped or tapered footings, offset footings, sloped footings, monolithic footings, round footings, rafts, etc. In conventional formwork for foundations, the formwork materials used are primarily timber boards or planks or plywood.

Forms for small isolated footing

For small isolated footing, a timber plank or a plywood is used as sheathing material which is supported by vertical stakes driven into the ground. Braces which support the stakes and provide rigidity and spreaders which maintain the correct distance between the footing sides are also provided, depending on the requirement. Spreaders could be replaced with steel straps for a small footing. Depending on the site condition, sometimes the excavated earth may be kept behind the forms which may act as support to the sides. In most of the cases, especially if the forms are made of steel, the form height is kept more than the depth of the footing. In such cases, steel straps are located at the proposed finished concrete level.

For setting the footing forms, the center lines of the footing are located by the stretching the chalk lines on stakes. This is shown in Fig. 4.1 in an isometric view. The center of the top of each of the form sides are marked now. The side forms are brought and placed directly below the chalk lines so that the center mark of each of the form sides lies directly under the chalk lines. Footing form is leveled and the location with respect to the center line is cross-checked. The braces as shown in Fig. 4.2 are now installed. In some cases, the excavated earth would be kept behind the footing sides.

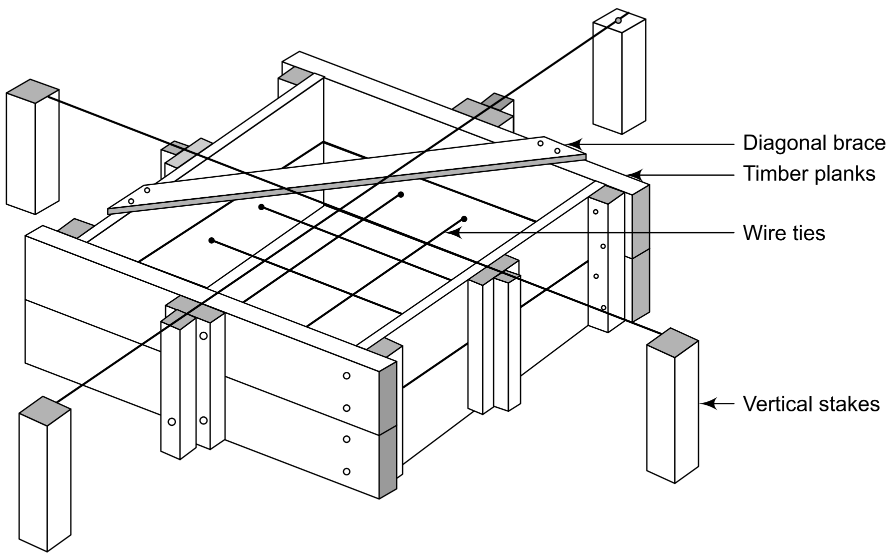


Figure 4.1 Setting of Footing Form by Center Line.

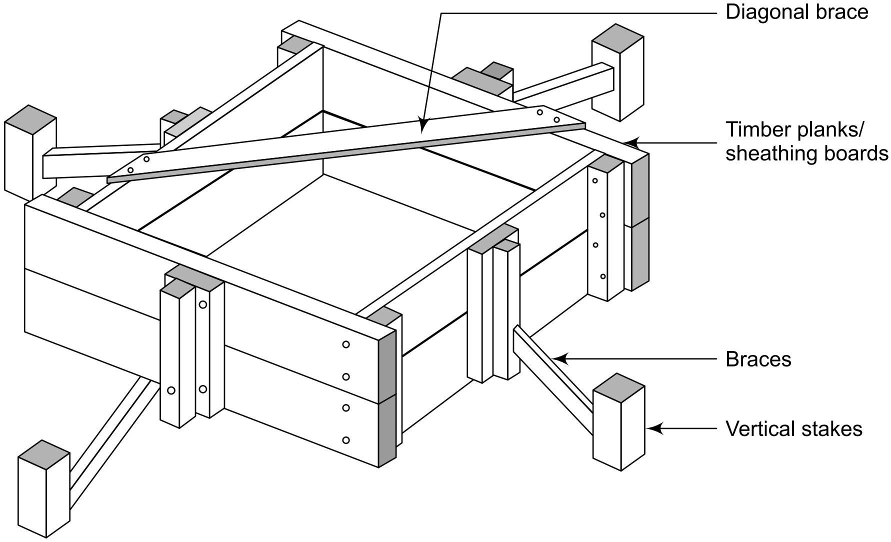


Figure 4.2 Footing Form Showing Arrangement of Braces.

Forms for Large footing

In case of large footings, the sheathing is supported by studs and wales. Sheathing as in case of small footing, could be of timber planks or plywood. One such large combined footing for four columns used at the Delhi International Airport is shown in Fig. 4.3. Timber studs have been used in the longer side to support the plywood sheathing while H-16 beams have been used as studs to support the plywood sheathing for the shorter side. Steel wales have been used to support the studs. Tubular props are used as braces to withstand the pressure exerted on the formwork by the fresh concrete. The braces for the longer sides are supported on the previously cast footing while they are supported on the excavated earth in the shorter side.

Figure 4.3 Formwork Arrangement for a Large Combined Footing Used at Delhi International Airport.

Sometimes the steel form panels used for walls and slabs are also used for forming the foundation. These steel form panels are also known as floor form. The floor forms are usually light gauge mild steel sheets with pressed flanges and stiffeners for rigidity and strength. The floor forms are made in standard modular sizes. Adjuster panels are also available to fill the odd gaps, if found during the form installation. In Fig. 4.4, floor forms made up of pressed bent sheets are shown. The longitudinal box/ rib stiffeners provide rigidity to the floor forms and help them maintain the edges even after considerable number of uses.

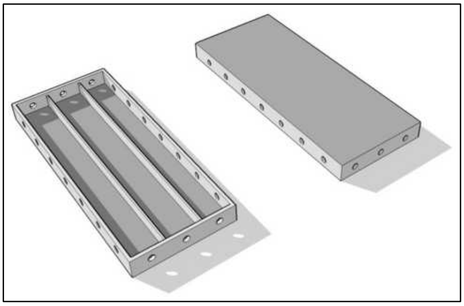


Figure 4.4 A Typical Floor Form.

The floor forms are also known as centering plates in the industry and can also be used for casting of floor slabs, RCC columns, beams, and walls. The quality of concrete finish obtained is reasonable. The cost and time involved in the making, as observed in case of timber formwork, is minimized in case of floor forms. The materials used for manufacturing these floor forms are typically 2 mm sheets with pressed flanges and stiffeners. For heavy applications, floor forms made up of sheet thickness of 2.5 mm and 3 mm are common. Standard floor forms are made in 600 mm widths and 1,150 and 900 mm lengths.

4.2.1 Forms for Column Footing

Forms for column footing are like a box without bottom. The footing form is constructed as four sides. There are two end sections and two side sections. While the end sections are made exactly as per the footing dimensions, the side sections may not be of the exact dimensions. In other words, side sections may appear to be as shown in Fig. 4.5. Ties could be provided internally as shown in Fig. 4.5 or externally as shown in Fig. 4.6. The ties prevent bulging of the form under concrete pressure. For small column footing, diagonal braces made up of timber are nailed across the top of the side shutters as shown in the previous figures.

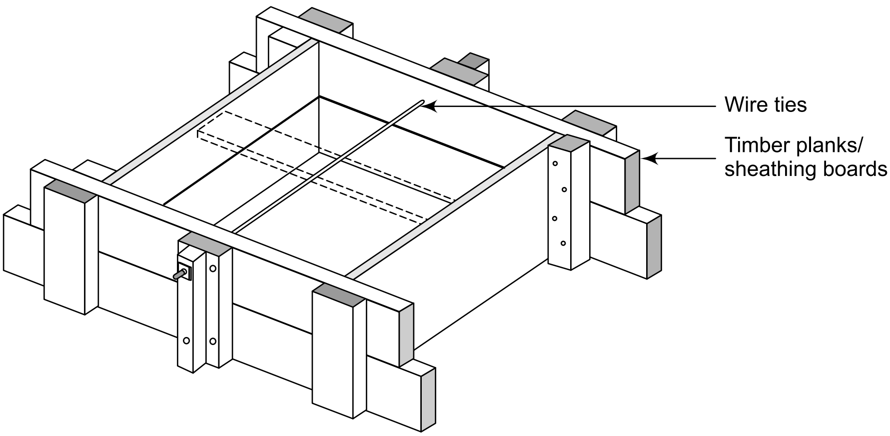


Figure 4.5 Column Footing Showing the End Sections of Exact Dimension and Side Sections of Extended Length. Also Notice the Internal Tie Arrangement.

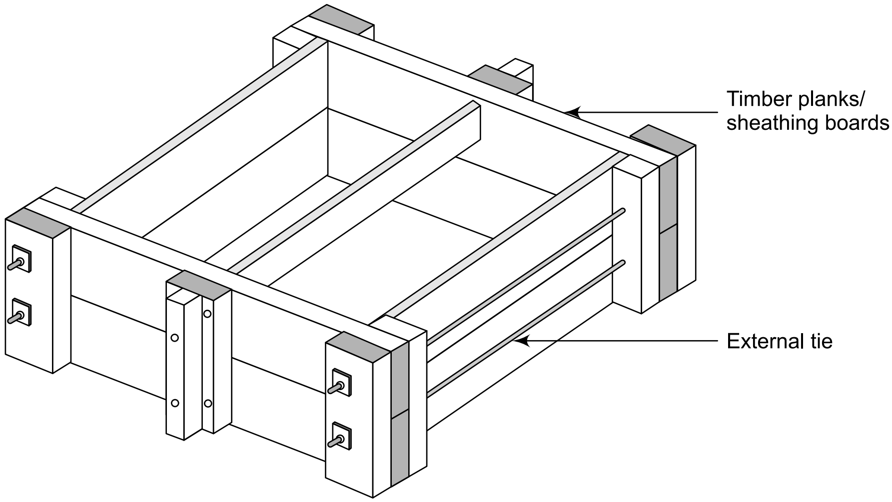


Figure 4.6 Column Footing Showing the External Tie Arrangement.

Large column footings (Fig. 4.7) can be made of shutter panels supported by steel soldiers. The shutter panels consist of plywood and timber. Tie rods may not be provided. The pressure exerted by the fresh concrete is transferred to the tubular steel braces.

Forms for foundation walls

Figure 4.8 shows a typical concrete footing and a foundation wall. The pressure exerted on the sides of the wall footing is comparatively less since the height of a typical foundation wall varies usually between 600 mm to 1,800 mm. Typical materials used in foundation wall formwork are 12–19 mm plywood as sheathing materials, studs and plates of cross section 50×100 mm.

Figure 4.7 Large Column Footing at Delhi International Airport.

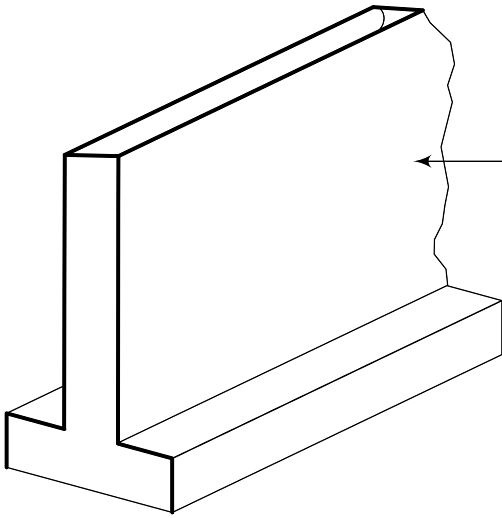


Figure 4.8 View of a Typical Concrete Footing and Foundation Wall.

Figure 4.9 shows a typical shutter panel made up of 19 mm plywood as sheathing material and 50 mm x 100 mm plates and studs. The studs are typically placed at a distance of 300 mm.

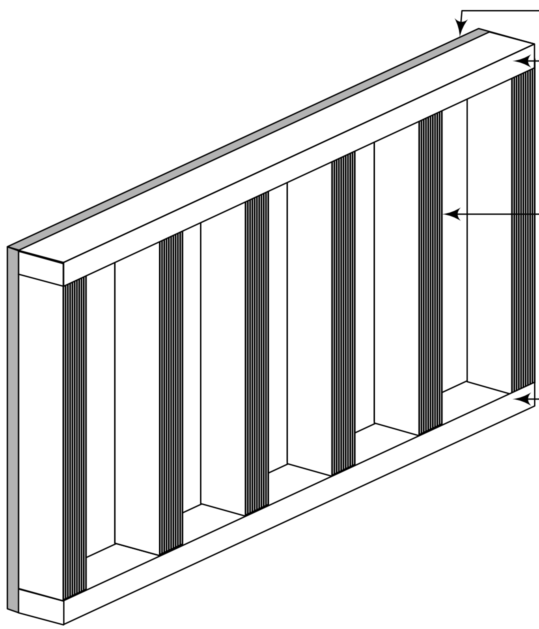


Figure 4.9 View of a Typical Shutter Panel Made up of Timber and Plywood.

In order to allow speedy placement of ties through the wall and the forms, suitable edge distance as shown in Fig. 4.10 is provided.

The step-by-step procedure for erecting formwork for foundation wall is shown schematically from Fig 4.11 through Fig 4.16 and discussed as below:

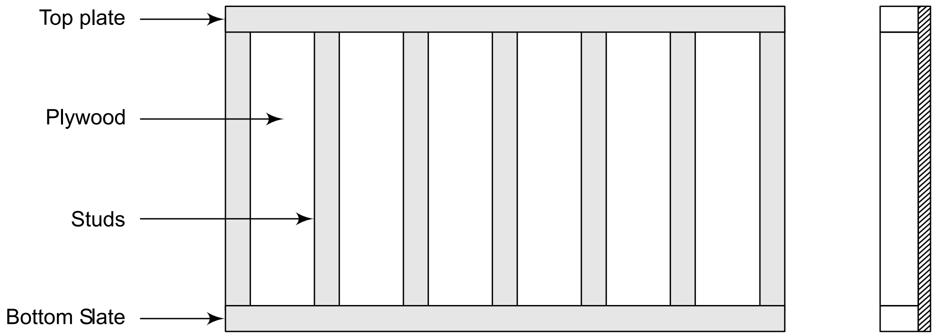


Figure 4.10 A Typical Timber and Plywood Panel.

Step 1

Two sole plates of cross section 100 mm × 100 mm are placed along the footing (see Fig. 4.11). The plates are nailed with the nailer inserts in the footing. The spacing between the plates is equal to the thickness of the wall. The plywood sheathing fixed on the plates is slightly protruded (3–4 mm) for ease in placing the spreader ties at the given spacing.

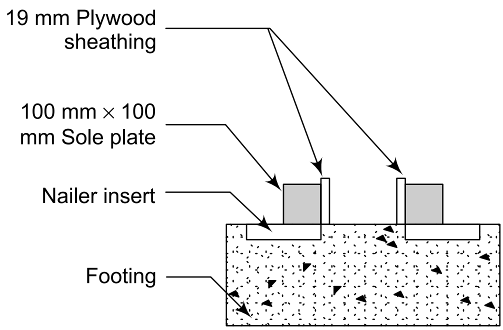


Figure 4.11 Sole Plates Placed in Position.

Step 2

The end spacer ties are fixed at the given spacing across the sole plates with the help of a hammer. The action of the hammer helps the ties to get embedded in the projecting plywood. Alternately, notches can be pre-cut at the tie locations. In such cases, hammers would not be required. Shutter panels are brought one by one and kept on the sole plate. Ends of the shutter panels butt against each other along the footing and they are joined with each other either by nailing or bolting on the end studs. The shutter panels are connected with the sole plates by nailing. The lower ties are now clamped. The arrangement is shown in Fig. 4.12.

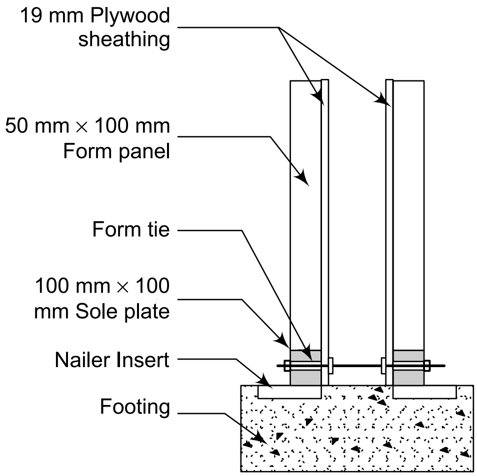


Figure 4.12 Lower Tier Shutter Panels Placed in Position.

Step 3

The form ties at the upper tier of the shutter panels are laid at the given spacing on top of the shutter panels. As described previously, another tier of shutter panels (upper tier) is brought one by one and kept on the previous shutter panels (lower tier). Here also, the ends of the shutter panels butt against each other along the footing and they are joined with each other either by nailing or bolting on the end studs. The upper tier shutter panels are connected with the lower tier shutter panels by nailing. The second level ties are now clamped. The arrangement of step 3 is shown schematically in Fig. 4.13.

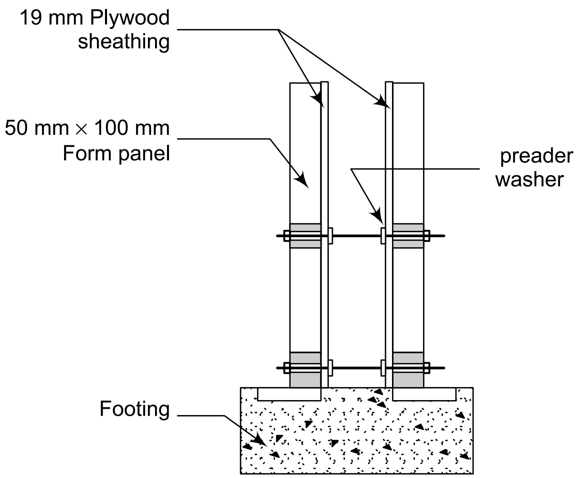


Figure 4.13 Lower Tier Shutter Panel Fixed and Upper Tier Shutter Panel Placed.

Step 4

Form spacers are fixed at the upper tier shutter panels at a given spacing (Fig. 4.14). The spacers could be made up of wood board, metal straps or steel rods, etc.

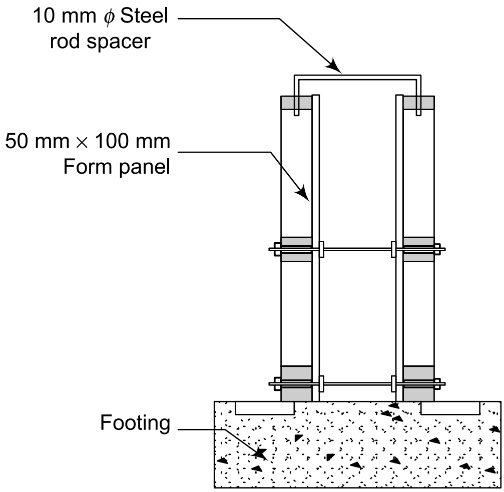


Figure 4.14 Upper Tier Shutter Panel Fixed.

Step 5

Strongbacks at suitable spacing, usually at the end junctions of shutter panels, are installed (Fig. 4.15). The cross section of a typical strongback in a normal case would be 2 number 50 mm × 100 mm timber.

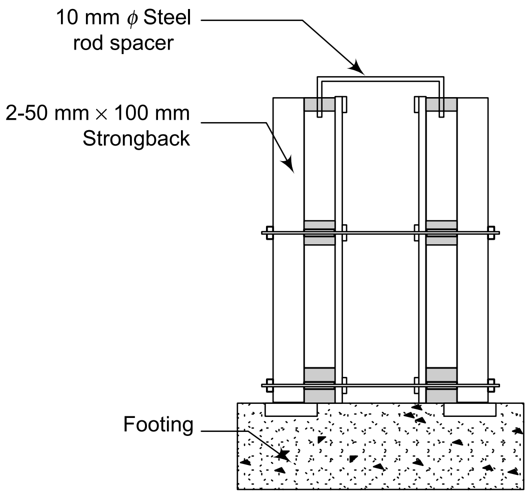


Figure 4.15 Strongbacks Installed.

Step 6

For alignment, braces are fixed with strongback at the top and stakes in the ground (Fig. 4.16). The cross–section of a typical brace in a normal case would be 50 mm × 100 mm timber. Depending on the requirement, a steel alignment prop can also be used.

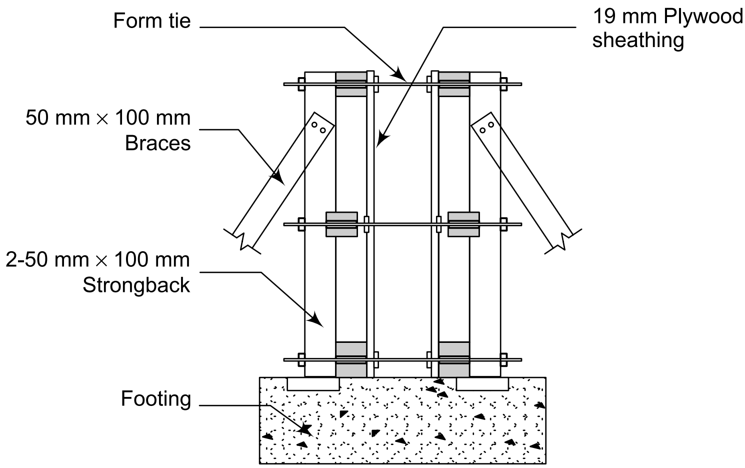


Figure 4.16 Braces Fixed.

The above steps are more or less common even if the form materials change. For example, the wooden shuttering panels can be replaced with the floor forms, the strongbacks could be replaced with channel sections, and timber braces could be replaced with the steel alignment props.

4.2.2 Stepped Footing

Stepped footings are needed for the sites having a sharp slope. These footings prevent the building from sliding. The width of the step is usually equal to that of the depth of the footing. This is shown in Fig. 4.17, where the width of the step is kept at 300 mm, which is equal to the depth of the footing.

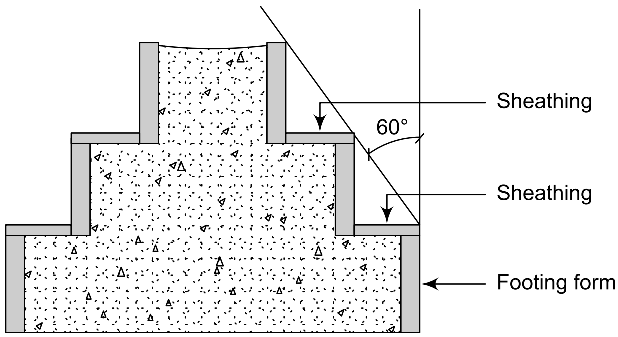


Figure 4.17 Section of a Typical Stepped Footing Form.

Stepped footing form consists of a series of individual sides. These forms are made to the required size of the footing.

The sides of stepped footings are made similar to the sides for the wall footing depending on the size of the footing. The only difference here is that sheathing boards are required to be placed over the openings at the top of each step of the form. In the absence of such sheathing boards, the top concrete surface at the step locations would require to be leveled and finished manually.

The end panels are fabricated to the exact size, while the side panels may not be of precise shape. Panels may be supported by braces or by heaping excavated earth around the sides. Figure 4.18 shows a typical stepped footing form.

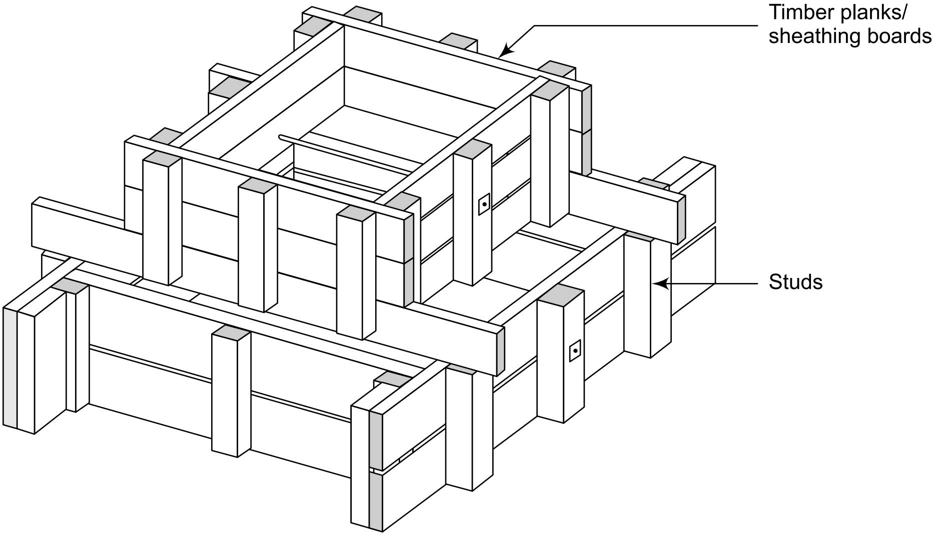


Figure 4.18 Stepped Footing Form.

4.2.3 Sloped Footings

As an alternative to stepped footing, sloped footing is sometimes preferred primarily with the objective to save concrete quantity. However, the formwork construction for sloped footing is difficult and costly compared to stepped footing. Thus, it is debatable whether an overall economy is achieved while preferring sloped footing over stepped footing. The increase in cost is on account of the increase in formwork making cost and making extra provisions for anchorage, besides extra cost incurred in placing and vibrating the concrete. Top forms in sloped footings are required usually when the slope is steeper than 40˚. Figure 4.19 shows the elevation of a typical sloped footing while Fig. 4.20 shows the complete view of a typical sloped footing form.

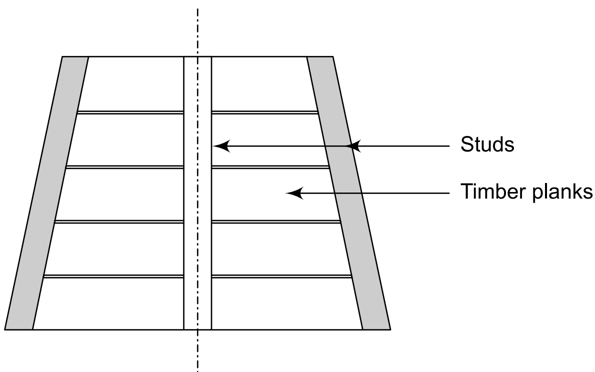


Figure 4.19 Elevation of Sloped Footing.

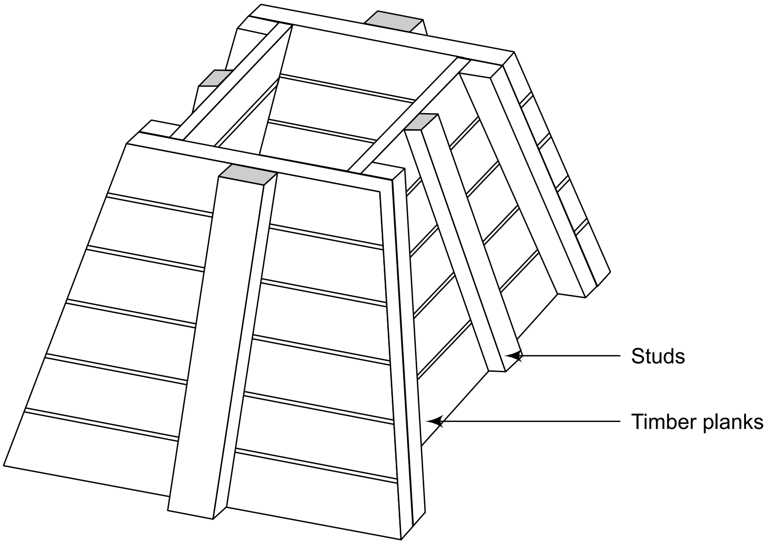


Figure 4.20 Sloped Footing Form.

4.2.4 Forms for Round Footing

The plan and isometric view of a large round footing are shown in Figs. 4.21 and 4.22, respectively. It is not very common to have round footing and only in exceptional cases, round footings are preferred. Formwork for round footings is expensive to construct. Outside yokes in the formwork for round footing are not required since the pressure is resisted by the hoop tension in the form. The materials required for the formwork for round footing are: timber, plywood, aluminum and GI sheets.

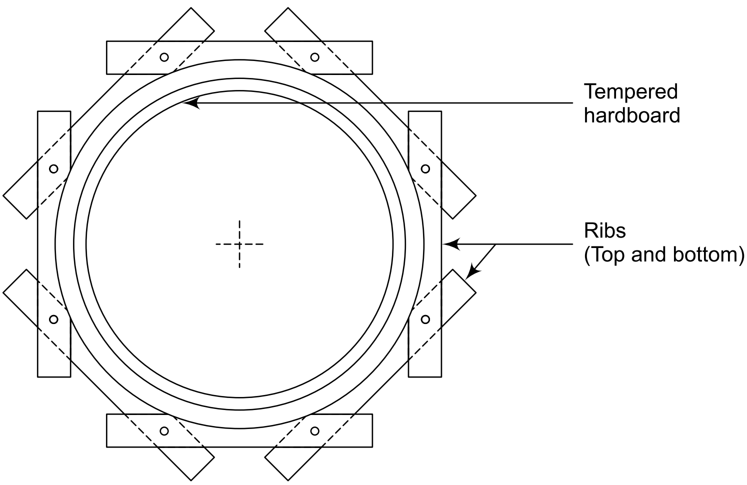


Figure 4.21 Form Plan for a Large Round Footing.

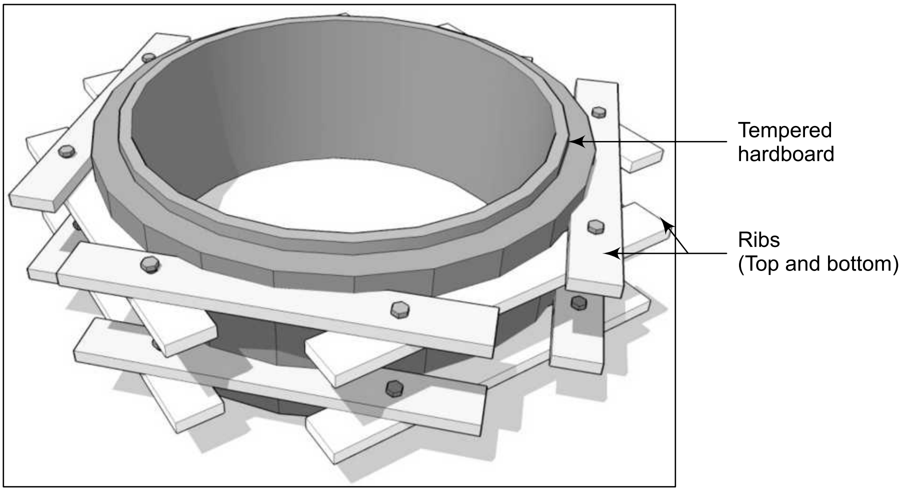


Figure 4.22 Isometric View of a Large Round Footing.

4.2.5 Raft

The plan and section of a typical raft formwork are shown in Figs 4.23 and 4.24. The floor forms mentioned earlier have been used to form the raft. The size of the floor form used is 1,200 mm × 600 mm. The two floor forms are connected with bolts in a proper manner. The floor forms are supported by the walers of 1,200 mm length and spaced at 1,200 mm center to center in the plan. A typical waler is made up of two ISMC 100, placed back to back with about 50 mm spacing between them. The space between two ISMCs is maintained by welding spacer plates at suitable intervals. Supporting brackets are provided at waler locations for resisting the formwork pressure. The brackets are fixed with the help of the head adaptor assembly with the waler, while at the ground, it is connected to the foot adaptor assembly. In some cases, foot adaptor assembly may not be used and the braces may be directly supported on the excavated earth as shown in Fig. 4.25.

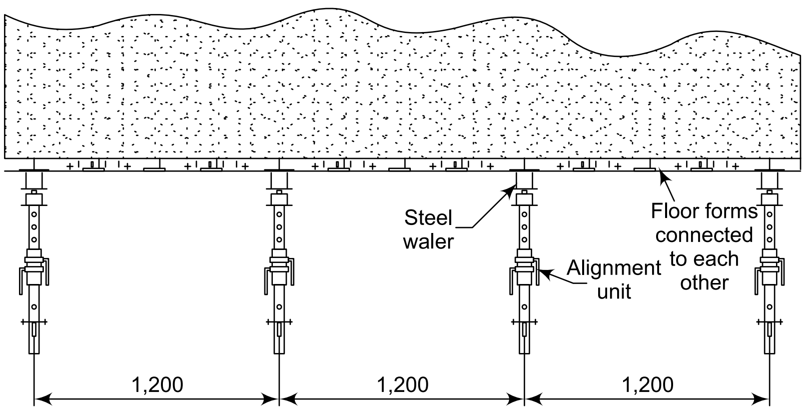


Figure 4.23 Plan of Raft Formwork.

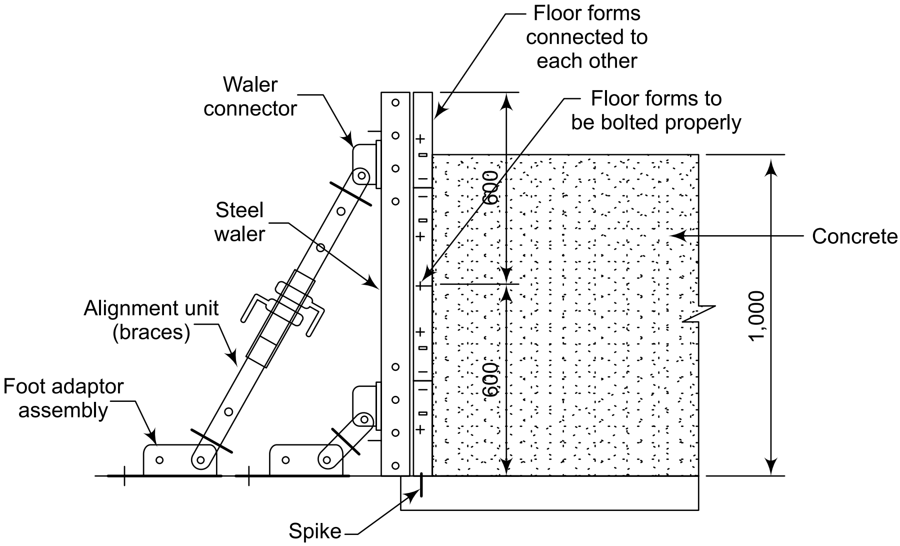


Figure 4.24 Section of Raft Formwork.

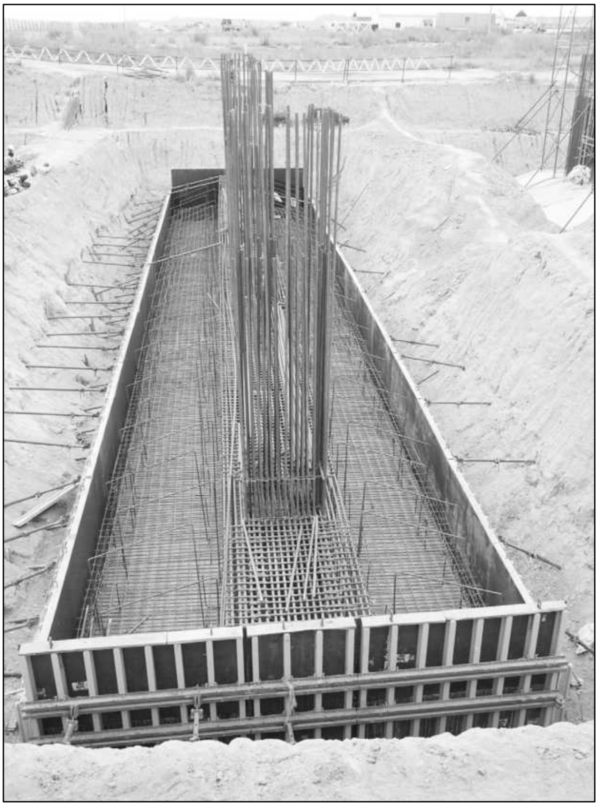


Figure 4.25 Footing Form Arrangement Showing Braces Being Supported by Excavated Earth.

4.3 FOUNDATION FORMWORK (ALL STEEL)

L&T has come up with an all steel foundation formwork. The system is suitable for side formwork of substructures, where the use of timber becomes uneconomical due to less number of repetitions. The maximum permissible concrete pressure suggested by the manufacturer is 25 kN/m^2. In this system, prefabricated steel panels known as floor forms are used, which reduces the need of onsite fabrication. The panels can be assembled easily thus ensuring dimensional accuracy. The plan and section of formwork arrangement for all steel foundation formwork are shown in Figs 4.26 and 4.27, respectively, while an isometric view of the same is shown in Fig. 4.28. The various components used in the system are briefly discussed below:

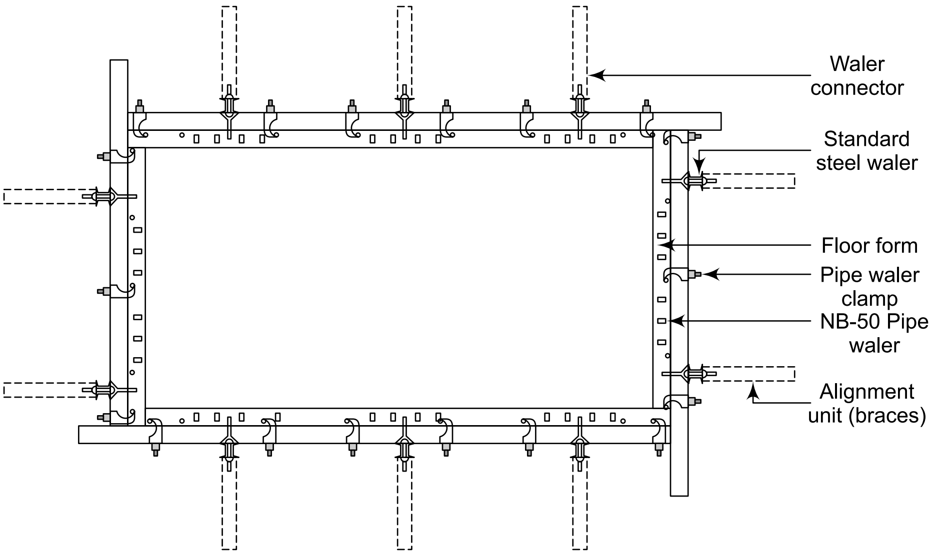


Figure 4.26 Plan View of All Steel Foundation Formwork.

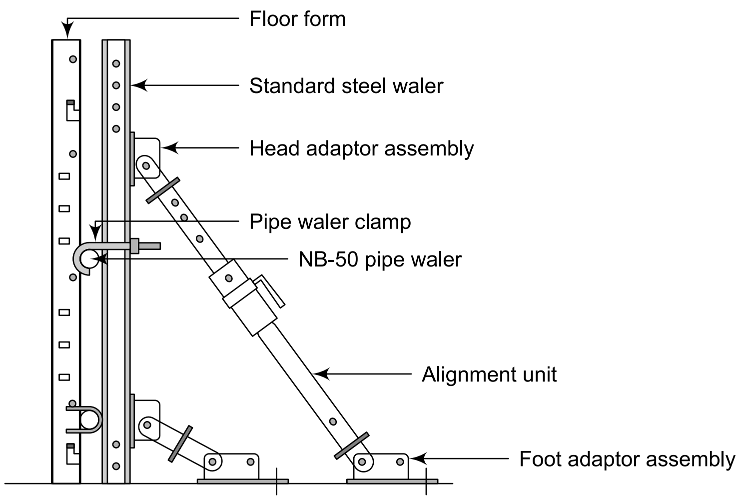


Figure 4.27 Sectional View of All Steel Foundation Formwork.

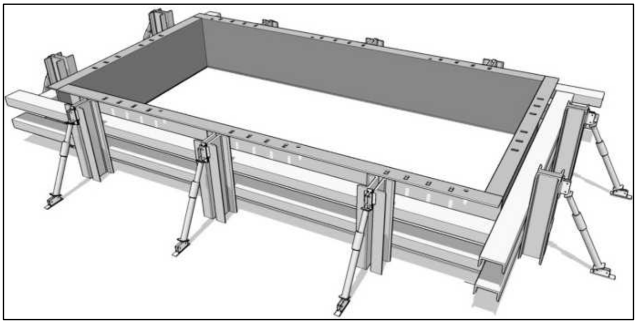


Figure 4.28 Isometric View of All Steel Foundation Formwork.

Floor form: This is the main component of the foundation formwork. Incidentally this is the same component used in slab formwork. The floor forms come in different sizes such as 1,200 × 300, 1,200 × 600, 600 × 300, 900 × 300, and 900 × 600. The weight of a floor form varies from 9.45 kg to 22.75 kg depending on the size.

Floor form corner: The floor form corner is available in different sizes such as 600 mm, 900 mm, and 1,200 mm. The weight of each floor form corner varies from 2.5 kg to 5.0 kg depending on the size.

Lapping plate: These plates are available in three sizes: 600 mm, 900 mm, and 1,200 mm. The weight of each lapping plate varies from 8.31 kg to 18.60 kg depending on the size.

Pipe waler: The pipe walers are available in three lengths: 1,500 mm, 2,000 mm, and 3,000 mm. The weight varies from 9.27 kg to 18.55 kg depending on the size.

Pipe waler clamp and waler connector: The weight of each pipe waler clamp is 1.30 kg while the weight of each waler connector is 0.75 kg.

4.4 FOUNDATION FORMWORK DESIGN

The foundations or footings are relatively low in height. The depth of the concrete is usually small, therefore, the pressure on the forms is relatively low. Thus, design based on strength consideration is generally not needed and the design thumb rules are sufficient to arrive at the thickness of plywood, spacing of studs, and tie rods.

Where the foundation is of a reasonable depth, design calculations need to be made in order to decide the sizes of the various formwork elements. These elements are: sheathing, studs, wales, and tie rods. The design of these elements is more or less similar to the wall formwork design and is covered in detail in the next chapter. For the sake of completeness however, we provide an illustration of the design of the foundation wall.

4.5 ILLUSTRATION ON FOUNDATION WALL DESIGN

The design of a typical foundation wall form is illustrated. The height of concrete foundation wall is 1.8 m. The concrete pressure diagram is given in Fig. 4.29. It can be noticed that the maximum pressure on formwork is 30 kN/m^2.

Let's assume that following materials are available.

1. 19 mm plywood and 50.8 mm × 101.6 mm studs and plates, tie rods of capacity 2,000 kg;
2. For 19 mm plywood, the allowable bending is 0.34 kNm /m, the allowable shear is 9.75 kN/m, and the permissible EI value = 2.73 kNm^2;
3. For timber used, consider the following values:
   $E = 7.0 \times 10^3$ N/mm^2, permissible bending stress = 10 N/mm^2, permissible shear stress $\tau = 1.0$ N/mm^2.

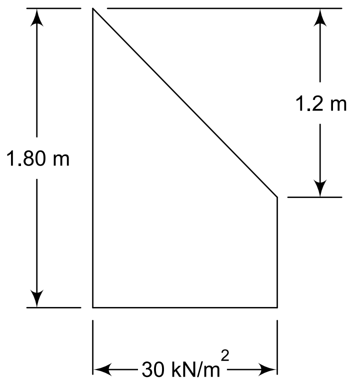


Figure 4.29 Pressure Distribution Diagram for Foundation Wall.

4.5.1 Sheathing Design

The thickness of plywood sheathing is given to be 19 mm. We need to find out the span of plywood which is same as the spacing of the studs. The maximum safe span is found out for bending moment, shear, and deflection considerations for continuous supported condition. The arrangement of the plywood and the stud is shown in Fig. 4.30.

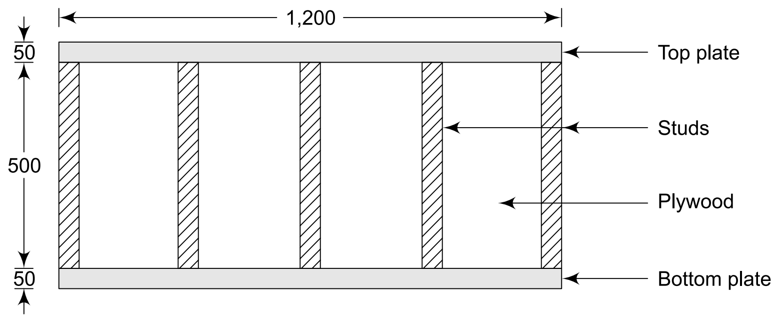


Figure 4.30 Arrangement of Studs and Plate in One Form Panel.

Span based on bending

$M = wl_p^2/10$ (Sl. No. 7, Table 3.11), where l_p is the span of plywood sheathing.

Allowable bending moment for 19 mm plywood = 0.34 kN/m

Thus,

$$M = 0.34 = \frac{w \times l_p^2}{10}$$

Thus, span $$l_p = \sqrt{\frac{10 \times 0.34}{w}} = \sqrt{\frac{3.4}{w}} = \sqrt{\frac{3.4}{30}} = 0.336 \text{ m} \tag{4.1}$$

Span based on shear

$$V = \frac{5wl_p}{8} \text{ (Sl. No. 7, Table 3.11)}$$

Allowable shear for 19 mm plywood = 9.75 kN

Thus, $$V = 9.75 = \frac{5wl_p}{8}$$

Thus, span $$l_p = \frac{9.75 \times 8}{5 \times w} = \frac{15.6}{w} = 0.52 \text{ m} \tag{4.2}$$

Span based on deflection criteria of l/360

$$\delta = \frac{wl^4}{145EI} \text{ (Sl. No. 7, Table 3.11)}$$

Allowable deflection = l_p/360, and allowable EI = 2.73 kNm^2

Thus, $$\delta = \frac{wl_p^4}{145EI} = \frac{l_p}{360}$$

Thus, span $$l_p = \left(\frac{145EI}{360w}\right)^{1/3} = 0.332 \text{ m}$$

Span based on deflection criteria of 1.6 mm

$$\delta = \frac{wl_p^4}{145EI} = 1.6 \times 10^{-3}$$

Thus, span $$l_p = \left(\frac{145EI \times 1.6 \times 10^{-3}}{w}\right)^{1/4} = 0.381 \text{ m}$$

Thus, span based on deflection = 0.332 m (4.3)

The minimum value out of (4.1), (4.2), and (4.3) is taken as the span of plywood which is the spacing of the studs.

It is proposed to space the studs at 275 mm center to center. The arrangement of the studs is as shown in Fig. 4.30.

4.5.2 Stud Design

Each stud is spanning between the top and bottom plates of 50.8 mm × 101.60 mm in a simply supported manner. Here again the maximum safe span of studs shall be determined based on bending, shear, and deflection consideration.

Area of the stud A = 5,161.28 mm^2

Moment of inertia of the stud I = 44,39,801.87 mm^4

Section modulus of the stud Z = 87,397.67 mm^3

Span based on bending stress consideration

The load on one stud w_s will be = 30 × 0.275 = 8.25 kN/m

$M = \dfrac{w_s \times l_s^2}{8}$ (Sl. No. 5, Table 3.11), where l_s is the span of the studs.

$$\text{Extreme fiber stress, } f = \frac{My}{I} = \frac{M}{Z} = \frac{w_S \times l_S^2}{8Z} \tag{4.4}$$

$$\text{Therefore, span} \quad l_s = \sqrt{\frac{8 \times f \times Z}{w_S}} = \sqrt{\frac{8 \times 10 \times 87{,}397.67}{8.25}} = 920.59\,\text{mm} \tag{4.5}$$

Span based on shear stress consideration

$$V = \frac{w_S \times l_S}{2}$$

$$\tau = \frac{VQ}{Ib} = \frac{V \times \dfrac{bd^2}{8}}{\dfrac{bd^3}{12} \times b} = \frac{3 \times V}{2 \times b \times d}$$

$$\Rightarrow \qquad \tau = \frac{3 \times w_S \times \dfrac{l_S}{2}}{2 \times b \times d}$$

$$\Rightarrow \qquad l_s = \frac{4 \times \tau \times b \times d}{3w_s} = \frac{4 \times 1.0 \times 50.8 \times 101.6}{3 \times 8.25} = 834.15\text{ mm} \tag{4.6}$$

Span based on deflection criteria of l/360

The expression for deflection ($5wl^4/384EI$ in case of U.D.L. simply supported condition) is equated with the permissible deflection to get the span of the studs.

$$\delta = \frac{5 \times w_s \times l_s^4}{384 \times E \times I} = \frac{l_s}{360}$$

$$\Rightarrow \qquad l_s = \left(\frac{384EI}{360 \times 5w_S}\right)^{\frac{1}{3}} = \left(\frac{384 \times 7.0 \times 10^3 \times 44{,}39{,}801.87}{360 \times 5 \times 8.25}\right)^{\frac{1}{3}} = 929.72\text{ mm} \tag{4.7}$$

The minimum value of spacing obtained from (4.5), (4.6), and (4.7) is chosen, which indicates the maximum span of the studs. In this case, it is based on shear consideration. The minimum value is = 834.15 mm which is more than 500 + 50/2 + 50/2 = 550 mm (span provided). Hence, O.K.

4.5.3 Tie Rod Spacing

The spacing of tie rods is the span of the plates. The maximum safe span is found out for bending moment, shear, and deflection considerations for continuous supported condition. Let's find the span of the plates at the junction of lowermost shutter and middle shutter. There are two plates of 50.8 mm × 101.6 mm (see Fig. 4.31). The center of the plates is located 1.2 m from the top of the concrete and therefore, the pressure at the center of the plates would be 25 × 1.2 = 30 kN/m^2 and the load per linear m = 30 × 0.6 = 18 kN/m. It is assumed that the plate would be subjected to a load from 300 mm above and below the center of the plates.

The area of the two plates = 5,161.28 × 2 = 10,322.56 mm^2
Section modulus Z_{xx} = 2 × 87,397.67mm^3 = 1,74,795.34 mm^3
Moment of Inertia, I = 2 × 44,39,801.87 mm^4 = 88,79,603.74 mm^4

Span based on bending moment consideration

$$M = \frac{w_{pl} \times l_{pl}^2}{10} \text{ (Sl. No. 7, Table 3.11)}$$

$$\text{Extreme fibre stress, } f = \frac{My}{I} = \frac{M}{Z} = \frac{w_{pl} \times l_{pl}^2}{10 \times Z}$$

$$\text{Therefore, span } l_{pl} = \sqrt{\frac{10 \times f \times Z}{w_{pl}}} = \sqrt{\frac{10 \times 10 \times 1,74,795.34}{18}} = 985.44 \text{ mm} \quad (4.8)$$

Span based on shear consideration

$$V = \frac{5 \times w_{pl} \times l_{pl}}{8} \text{ (Sl. No. 7, Table 3.11)}$$

$$\tau = \frac{VQ}{Ib} = \frac{V \times \left(\frac{bd}{2}\right) \times \left(\frac{d}{4}\right)}{Ib} = \frac{3V}{2bd} = \frac{3 \times 5 \times w_{pl} \times \frac{l_{pl}}{8}}{2bd} = \frac{15 \times w_{pl} \times l_{pl}}{16bd}$$

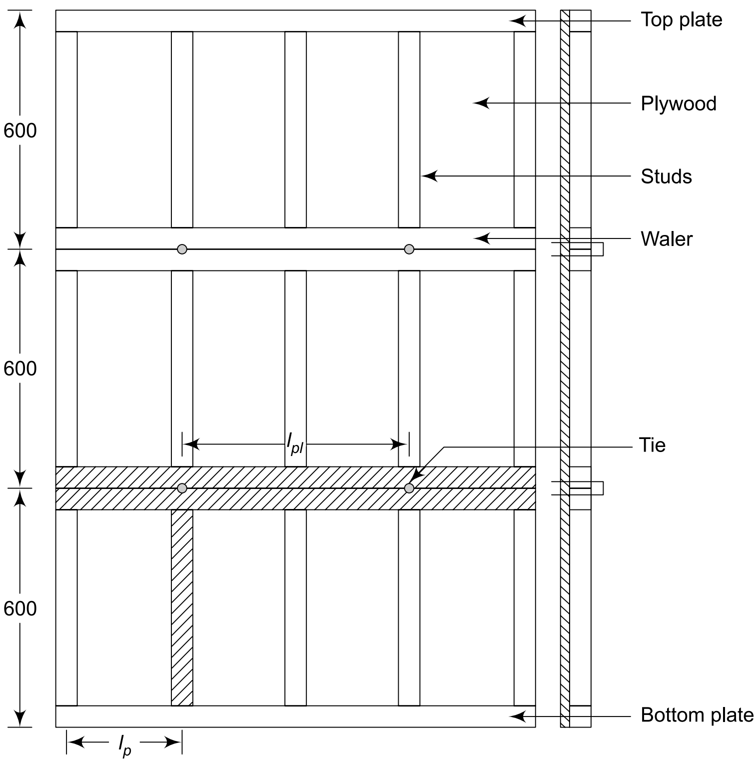


Figure 4.31 The Arrangement of Different Form Panels and Tie Rods in Elevation and Sectional View.

$$\Rightarrow \qquad \text{span } l_{pl} = \frac{16 \times \tau \times b \times d}{15 w_{pl}} = \frac{16 \times 1.0 \times 101.6 \times 101.6}{15 \times 18} = 611.70 \text{ mm} \qquad (4.9)$$

The value of b is taken as $2 \times 50.8 = 101.6$ mm in the above expression.

Span based on deflection criteria of l/360

$$\delta = \frac{wl^4}{145EI} \text{ (Sl. No. 7, Table 3.11)}$$

$$\delta = \frac{w_{pl} \times l_{pl}^4}{145 \times E \times I} = \frac{l_{pl}}{360}$$

$$\Rightarrow \qquad l_{pl} = \left(\frac{145EI}{360 \times w_{pl}}\right)^{\frac{1}{3}} = \left(\frac{145 \times 7.0 \times 10^3 \times 88{,}79{,}603.74}{360 \times 18}\right)^{\frac{1}{3}} = 1{,}116.25 \text{ mm} \quad (4.10)$$

The permissible span is the minimum of the three values given by (4.8), (4.9), and (4.10). Thus the maximum permissible span = 611.70, say 600 mm. Thus, provide the tie rod at a spacing of 600 mm. The tie will be located at a distance of 300 mm from each end of form panel and at 600 mm interval in the intermediate location as shown in Fig. 4.31.

4.5.4 Check the Tie Rod

The load exerted on the tie rod can be computed by multiplying the concrete pressure with the area of load being taken care of by the tie rod. In this case, one tie rod is catering an area of 600 mm × 600 mm.

Thus, area = 0.6 × 0.6 = 0.36 m^2. The load exerted on one tie rod = 0.36 × 30 kN = 10.8 kN. The capacity of the given tie rod is 20 kN; hence it is safe.

REVIEW QUESTIONS

Q1. True or False

(a) Basic materials in footing formwork are timber sheathing materials, stakes, braces, nails, studs, wales, spreader, and ties.

(b) In sheathing design, span is based on – bending, shear, and deflection criteria.

Q2. Match the following:

(i) Sheathing material made of timber	(a) Thickness 3.15 mm to 5 mm, size 600 mm × 1,200 mm.
(ii) Sheathing material made of steel	(b) Thickness 50 mm, size: width approximately equal to the depth of the footing.

Q3. Match the following:

(i) Form for small isolated footing	(a) Sheathing supported by studs and wales.
(ii) Form for large footing	(b) Timber made sheathing supported by vertical stakes.

Q4. Match the following:

(i) Forms for column footing	(a) Needed for sites having a sharp slope and to prevent the building from sliding.
(ii) Stepped footing	(b) Alternative to stepped footing and preferred primarily with an objective to save concrete.
(iii) Sloped footing	(c) Expensive, unusual, no need of outside yokes.
(iv) Round footing	(d) Like a box without bottom; tier preventing concrete bulging.

Q5. Do an illustrative exercise for formwork design of foundation for both:

(a) Low depth design

(b) Reasonable depth design

Q6. List out the steps involved in erection of foundation wall formwork.

Chapter

5 Wall Formwork

Contents: Introduction; Conventional Wall Formwork; Proprietary Wall Formwork System; Large Area Wall Forms; Climbing Formwork; L&T Wall Formwork; PERI Wall Formwork; PERI Climbing Formwork; Doka Climbing Formwork; Wall Form Design; Illustration of Wall Formwork Design Using Plywood and H-16 Beams; Illustration of all Steel Wall Form Design

5.1 INTRODUCTION

In this chapter, we discuss the conventional wall formwork and different varieties of proprietary wall formwork, such as climbing scaffold, traveling climbing scaffold, and automatic climbing scaffold. Some real life illustrations of wall formwork in use have also been provided. We discuss the different design steps needed to perform wall formwork design. Some design aids for getting assistance in the wall formwork design are also presented. Towards the end, we illustrate few wall formwork design examples.

5.2 CONVENTIONAL WALL FORMWORK

The formwork for vertical concreting is usually called wall form. The main components of a wall formwork are sheathing, studs, wales, tie rods, and alignment props. The sheathing could be formed from timber boards or plywood sheets. The plywood sheathing is usually 1.2 m × 2.4 m (4′ × 8′) sheets of plywood in different thicknesses. The most commonly used thicknesses are 12 mm and 19 mm. The studs are normally 50 mm × 100 mm and the wales could be two numbers of 100 mm × 100 mm lumber joined together with a gap between them. The gaps are provided to accommodate the tie rods. The tie rods are metallic rods capable to resisting the tensile forces. Depending on the maximum pressure exerted on the wall formwork, the spacing and dimensions of studs, wales, and tie rods are decided. The alignment props are usually made of metal having the mechanism for pulling or pushing the shutter towards the concrete element to be poured. A sketch of conventional wall formwork is shown in Fig. 5.1. The various form components such as timber planks as sheathing board, studs, wales, spreader, plates, braces (inclined struts), and tie rods are clearly marked in Fig. 5.1.

Inclined struts or shores are to be used in high wall formwork in order to ensure stability of the formwork, and also verticality. The shores will at least be two on one (accessible) side of the formwork so as to ensure a stable configuration. The other side form will be rigidly connected using spacers and bolts to the side which has been shored. Shores in conventional wall form arrangement are usually of timber. However, proprietary metal shores are also very common these days. The shores are connected to the bedplate to distribute the pressure uniformly on the soil.

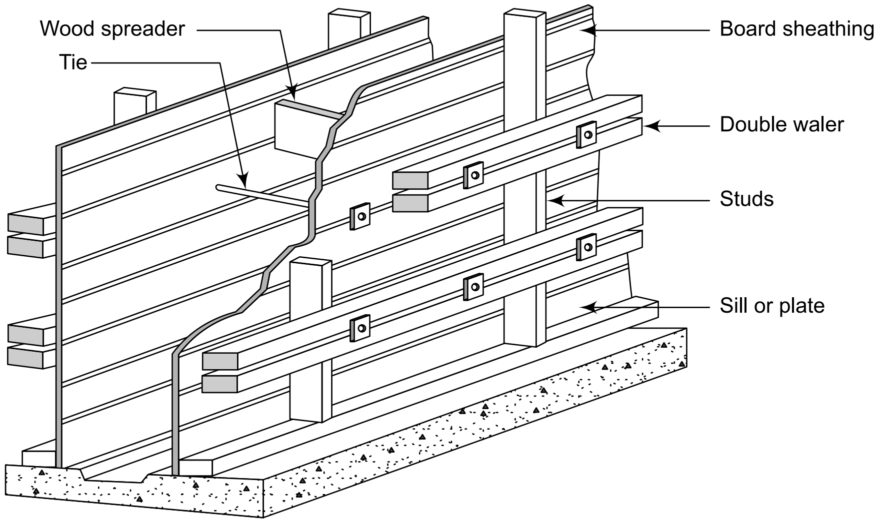


Figure 5.1 A Typical Conventional Wall Formwork.

5.3 PROPRIETARY WALL FORMWORK SYSTEM

A large number of proprietary wall formwork systems are available these days. We shall be discussing a few of them. The wall form system is so named as it is specially designed for wall construction. These systems can be applied to a wide range of wall forming applications irrespective of wall thickness. Some of the application areas for the wall forming systems in wall forming are: single sided wall, double sided wall, single lift or multiple lifts wall, and walls with vertical or sloping faces.

Although the type of components usually vary with each proprietary system, some of the elements which are common in most systems can always be pointed out. For example, a typical proprietary system would comprise of a standard modular steel panel. These panels are manufactured either with angle sections or rolled steel sectional frame with M.S. sheet or plywood for the faces. The panels usually have holes or slots at regular intervals so that the connection of panel to panel or panel to other backing members can easily be made. The connection is accomplished through specially designed clips or bolts.

The other common element is a strong back member, also known as soldier. The above mentioned panels are usually assembled to the soldiers spaced at the designed intervals. The other common element is a system of ties which is provided through the soldiers to resist the concrete pressure.

Some of the proprietary wall form systems have the provision for additional horizontal walers for transferring the load from panels to soldiers. The panels are connected to the walers by means of specially designed clips.

Generally, the wall form system with soldiers is of the climbing type. Since soldiers are longer than the form panel, they can be clamped to the lower lift with or without requiring any additional support. The soldiers climb on the form ties/ anchors embedded in each lift. More details on such climbing forms are presented later in the chapter.

One of the essential features of the proprietary systems is that the entire system is so designed that it can be assembled and dismantled very quickly. The forms can be assembled either in small sections, so that they can be handled manually or they could be formed in large sections. In the latter case, they certainly have to be handled with a crane or some other suitable mechanical devices.

5.4 LARGE AREA WALL FORMS

In a large area wall form, also termed gang form, a number of prefabricated panels are joined together to form a large shutter panel and thereby a large area of wall can be concreted simultaneously. These large panels are erected with the help of a crane. The four basic types of panel forms are unframed plywood, plywood in a metal frame, all metal, and heavier steel frames. The panels are aligned with the help of plumbing frames and special types of hardware.

Reinforcement and concreting operations are done in almost the same manner as performed in other wall forms. Cranes are needed for stripping the forms and shifting them to the next location. The large area wall panels are usually braced with wales, strong backs, and are equipped with special lifting hardwares.

The field application of large area wall formwork in the construction of a Track Hopper at Anpara Super Thermal Power Station Project is shown in Fig. 5.2. The project was executed by L&T-ECC Construction Group. The tower crane in the figure may be noted.

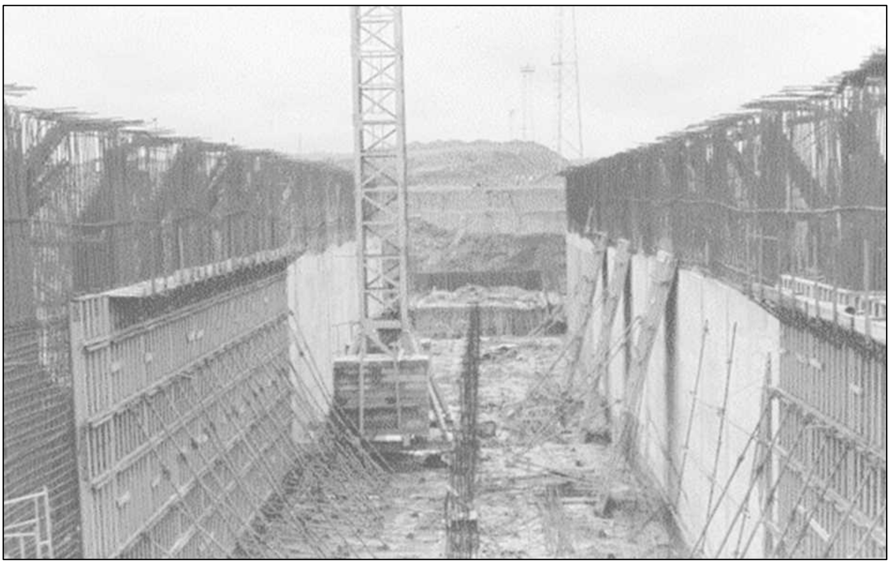


Figure 5.2 Application of Large Area Wall Formwork in Track Hopper Construction.

5.5 CLIMBING FORMWORK

Climbing formworks are also referred to as Jump form. Climbing formwork system is used for high-rise structures. In this system, formwork with working platforms is supported on the lift of concrete already done. The climbing form system is useful in situations where intermediate floors are not available to support the wall formwork. In the absence of intermediate floors, it will be difficult and uneconomical to support the wall formwork beyond the first few lifts from the ground.

The climbing formwork requires three additional components other than those needed for normal wall formwork. They are: supporting bracket with platform, hanging scaffold, and an anchorage device to transfer the loads coming from the brackets and hanging scaffold to the previously cast concrete. The supporting bracket with platform is used by the workers engaged in cleaning and oiling of forms, and in the removal of form ties and fixing of climbing shoe. The hanging scaffold is used for repair and minor touch up works to the already cast concrete. The hanging scaffold is also used by the workers engaged in curing of concrete.

In this system of formwork, the wall forms are raised vertically lifts after lifts. The wall forms for the lift going to be cast are supported on some anchors embedded in the previous lift. It is very important to ensure that the concrete strength gain at the anchors be sufficient at each stage of the operation to resist the various loads that it would be subjected to. In other words, the form is allowed to be supported from the previously cast lift only when the concrete has gained sufficient strength.

Climbing formwork should not be confused with a slipform that moves during placement of the concrete. Support of the climbing form is usually provided by anchors cast in the previous placements.

Climbing formwork essentially consists of two parts: wall form panel and working platform. The working platform is provided for working of crew members and it also is used to store construction materials. The difference between the climbing system and ordinary scaffolding system of wall construction is that in the climbing system, the entire load from form and other construction materials besides the work crew weight, is supported on the bracket fixed to the previous lift and not supported all the way to the ground by means of scaffolding as done in the latter system. This is explained with the help of the following diagrams:

In Fig. 5.3(a), step 1 of the ordinary or traditional wall formwork is shown, where the formwork and scaffolding are made ready for pouring the first lift of the wall.

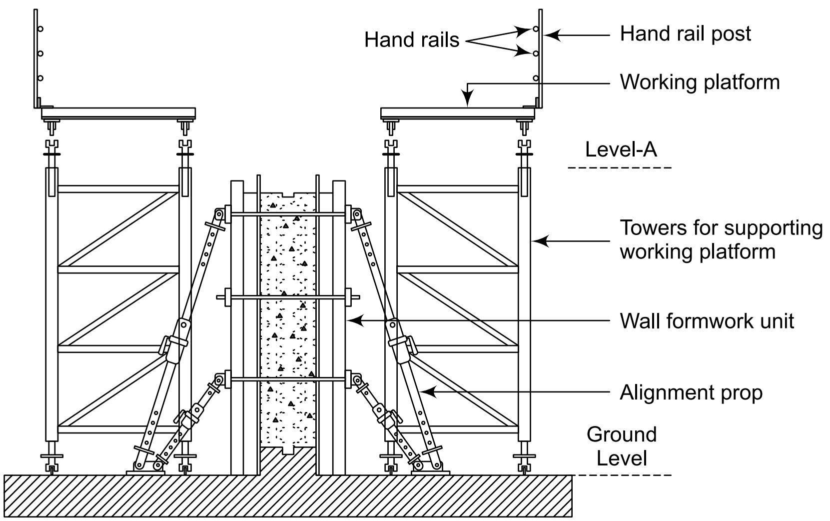


Figure 5.3(a) Step 1 of Traditional Wall Formwork-Casting of First Lift of the Wall.

For the next pour, the scaffolding is erected for greater height. The arrangement for casting the second pour of the wall is shown in Fig. 5.3(b).

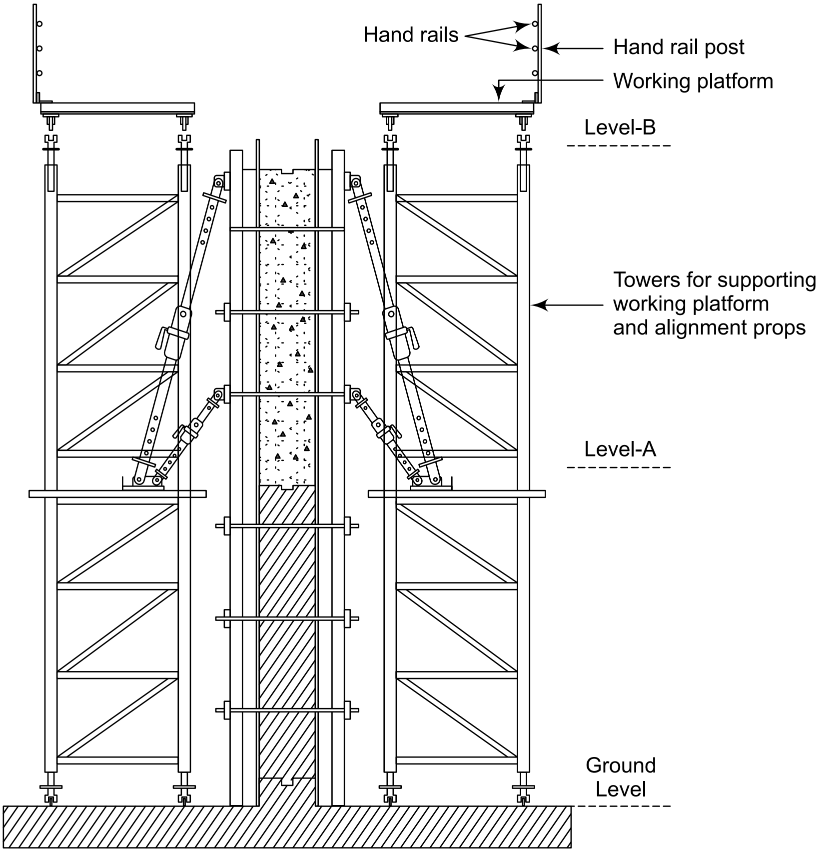


Figure 5.3(b) Step 2 of Wall Formwork— Casting of Second Lift of the Wall.

For the next pour, scaffolding is further raised to the desired height. This is shown in Fig. 5.3(c).

The process is repeated till the desired height of the wall is cast. It can be observed that adopting this method is cumbersome and time consuming.

In order to avoid the above situation in which the formwork is to be supported all the way to the ground, different types of climbing formwork systems are available these days depending on whether the wall form panels are cleaned on the ground or at elevated locations. These can be classified essentially in three types:

Type 1:

In this system of climbing formwork, the wall form panels are to be brought to the ground with the help of a crane for cleaning purposes on completion of each lift. The bracket is to be dismantled and fixed in its next position with the help of a crane. The step wise procedure is given below:

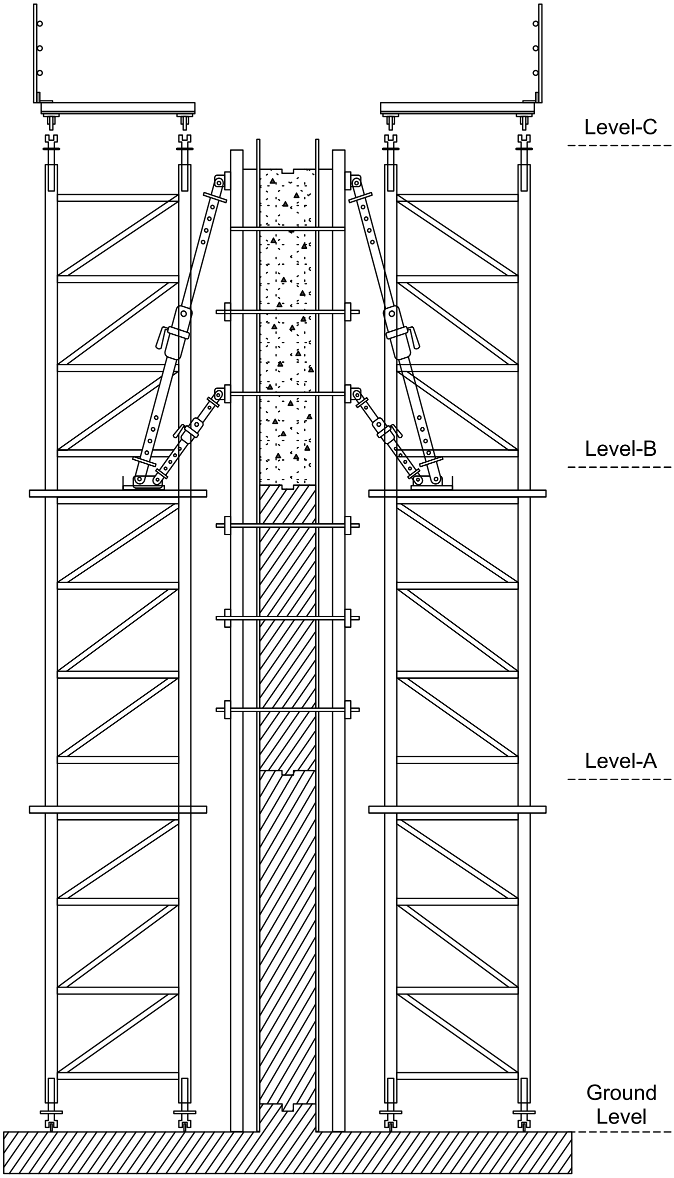


Figure 5.3(c) Step 3 of Wall Formwork— Casting of 3rd Lift.

Step 1

In this step, the first 1 or 2 lifts (depending on the lift height) are cast in a conventional manner upto level A or B as shown earlier in Figs. 5.3(a) and 5.3(b). During the casting of these lifts, provision for a suitable anchorage device is made. These anchorage devices are used to hang the brackets and suspended scaffolds. The brackets support the alignment arrangement of wall shutter besides the formwork crew. The suspended scaffolds are used by masons for finishing works and the curing crew.

Step 2

In this step, the brackets are attached to the anchorage devices left during the casting of the previous pour. The suspended scaffold is attached to these brackets. The wall formwork is assembled along with the tie rods and anchorages devices. This is shown in Fig. 5.4 (a). It can be seen that the formwork is ready to receive the concrete for the second lift. It is assumed here that appropriate height in the first lift has been achieved to hang the suspended scaffold. In case where it is not possible to hang the suspended scaffold, the second lift also needs to be performed in a conventional manner as shown in Fig. 5.3(b).

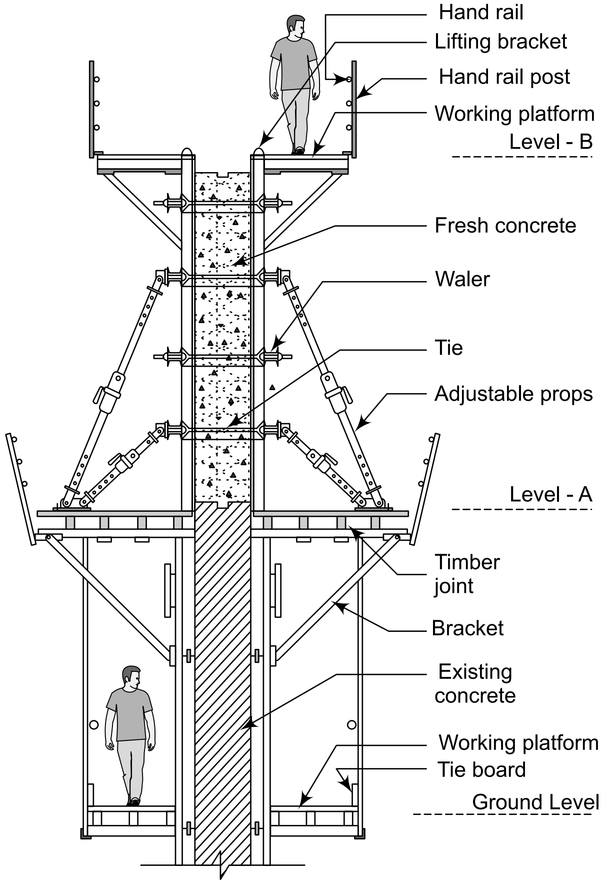


Figure 5.4 (a) Climbing Formwork Arrangement for Second Pour.

Step 3

In this step, after the concrete of the second lift has gained sufficient strength, the wall formwork panel along with the alignment brackets and the top working platform is brought to the ground essentially for cleaning purposes with the help of a crane. This is shown in Fig. 5.4 (b).

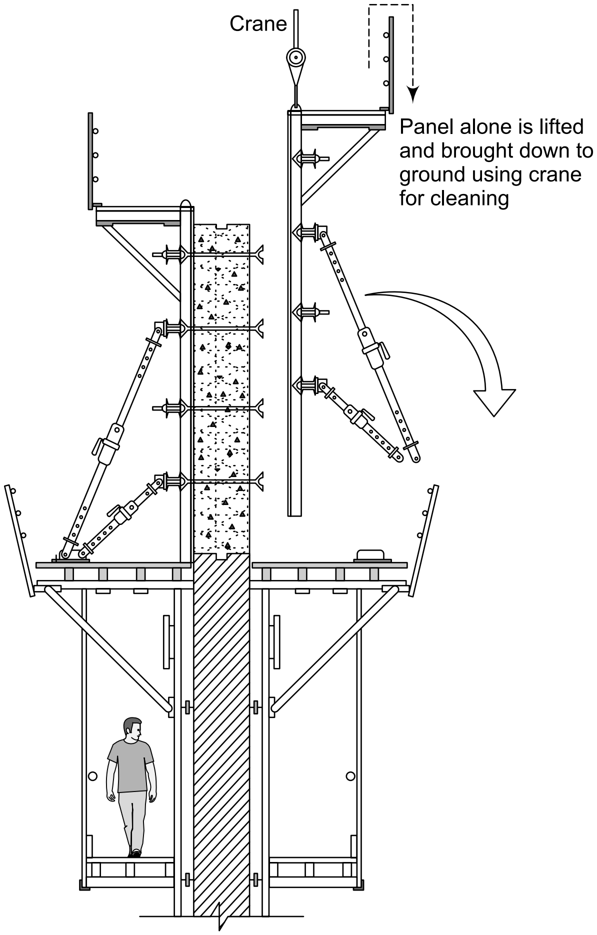


Figure 5.4(b) Stripping of Wall Formwork.

Step 4

The brackets and suspended scaffolds are dismantled, cleaned at the ground level, and attached to the anchorage devices left in the second lift. This is shown in Fig. 5.4(c). The dismantled formwork of the second lift, after cleaning, is brought and placed in position to receive the concrete for the third lift.

Type 2:

In this system of climbing formwork, the cleaning operation of the wall form can be performed at the elevated location itself and there is no need to bring the entire wall form all the way to the ground for cleaning. In this system of formwork, the wall form can be moved away from the concrete surface after the concrete has attained sufficient strength. The distance by which the wall form is moved is sufficient for some crew members to go between the concrete surface and the wall form to clean the wall form panels. Thus, lowering of the wall form to the ground level for cleaning purpose is avoided. After cleaning of the wall form panels, the entire assembly is lifted and fixed to its next position with the help of a crane.

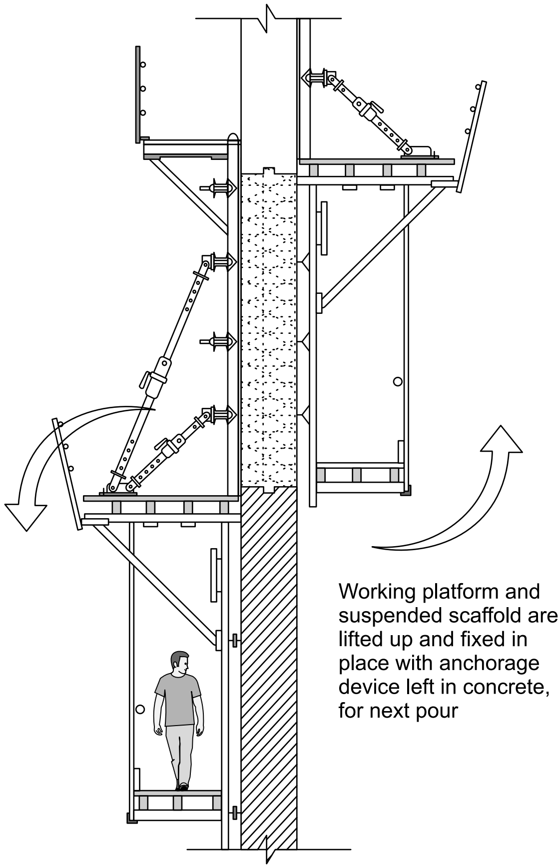


Figure 5.4(c) Fixing of Suspended Scaffold to Receive the Wall Formwork for Third Lift.

The various stages involved in the working of this system of formwork are given below with the help of schematic sketches.

Stage 1

In this stage, first lift of the wall is prepared in a conventional manner like any other wall formwork (see Fig. 5.3(a)). After tying the reinforcement, the wall panels are erected along with the top platforms and tie rods are fixed. The alignment braces are also installed, which are supported on the base slab (it could be ground as well). The accessories to support the climbing shoe are installed at the appropriate locations, normally at the upper tie location. Different kinds of climbing attachments are available with different trade names according to the manufacturer. Concrete is poured and allowed to gain strength.

Stage 2

After the concrete has attained the desired strength, tie rods are released. Formwork panels are moved away from the recently cast wall concrete either by tilting (see Fig. 5.5 (a)) or by rolling (see Fig. 5.5 (b)). Climbing shoe is attached to the climbing accessories left in the previous pour. The entire wall form assembly along with the supporting brackets is lifted with the help of a crane. The supporting bracket is attached to the climbing shoe now. Suspended scaffold can now be attached to the supporting brackets. The reinforcement for the lift is tied and forms are fixed. Similar to the first lift, climbing accessories are installed at the upper tie location. Forms are aligned and concrete is poured.

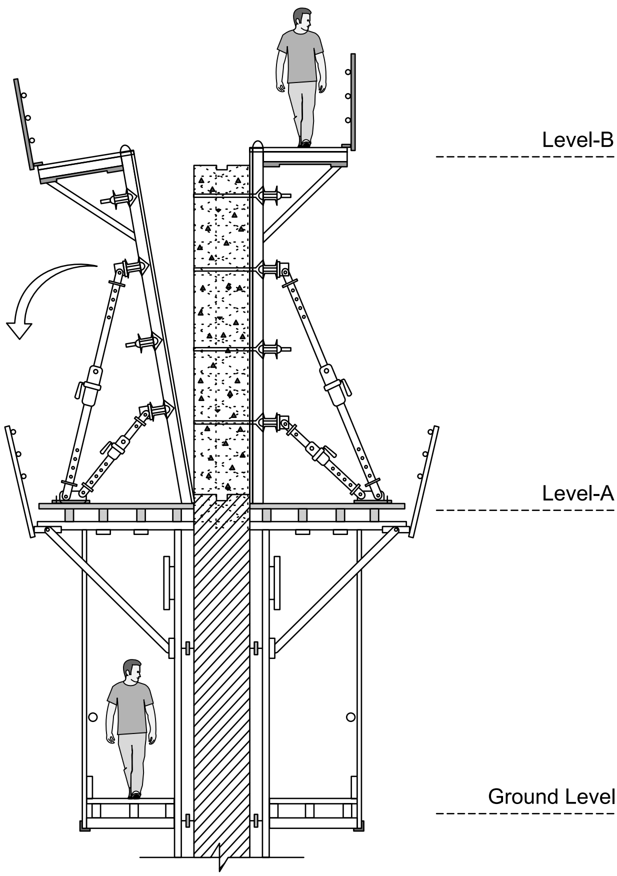


Figure 5.5 (a) View Showing the Gap Between the Concrete and Wall Form (Tilting Type).

Stage 3

After the concrete has gained sufficient strength, the tie rods are released. The climbing shoe for the next lift is fixed at the climbing accessories location left in the previous pour. The entire wall formwork assembly along with the supporting brackets and the suspended scaffold is moved to the third

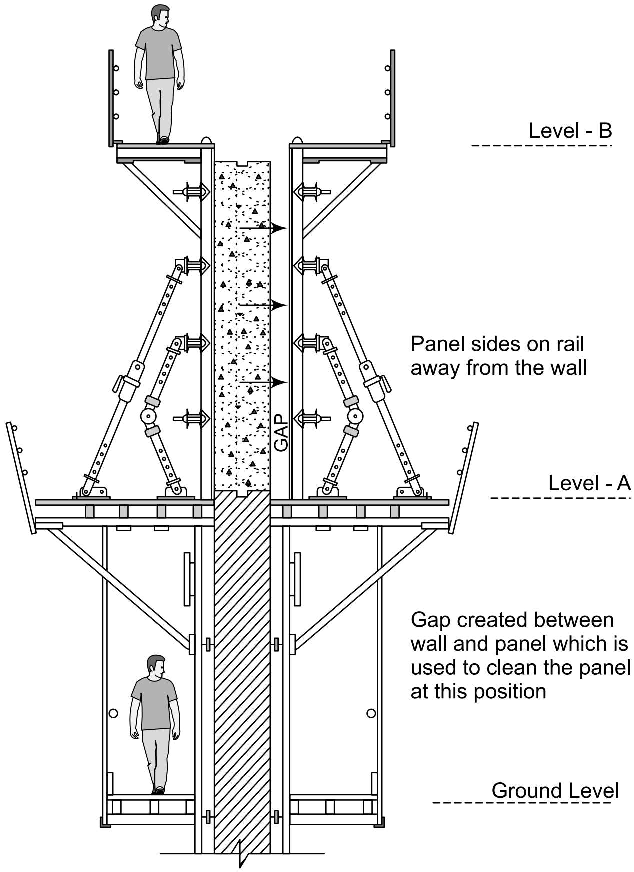


Figure 5.5 (b) View Showing the Gap Between the Concrete and Wall Form (Rolling Type).

lift location with the help of a crane (see Fig. 5.5 (c)). The process continues in a similar manner as described, till the desired wall height has been reached.

Climbing form offers a number of advantages. For example, the cycle time that can be achieved varies anywhere between 2-4 days. The crane time for formwork operation is considerably reduced. The crane is needed only for moving the wall formwork from one lift to the next. The system has in-built safety systems. Thus work can proceed safely at any height. The system can be designed to resist high wind speeds of the order of 150 km/h. All the components in a climbing formwork system are engineered and made in a factory set up, thus the designer can design the system for various loading and site conditions quite confidently.

Some of the limitations of the climbing formwork are: The form panels being of considerable height and width, the site must be properly accessible for ease in maneuverings. Sufficient clearance is also required while lifting the form from one lift to the next. The system's productivity and cycle time suffers if a large number of openings and block-outs are to be left out.

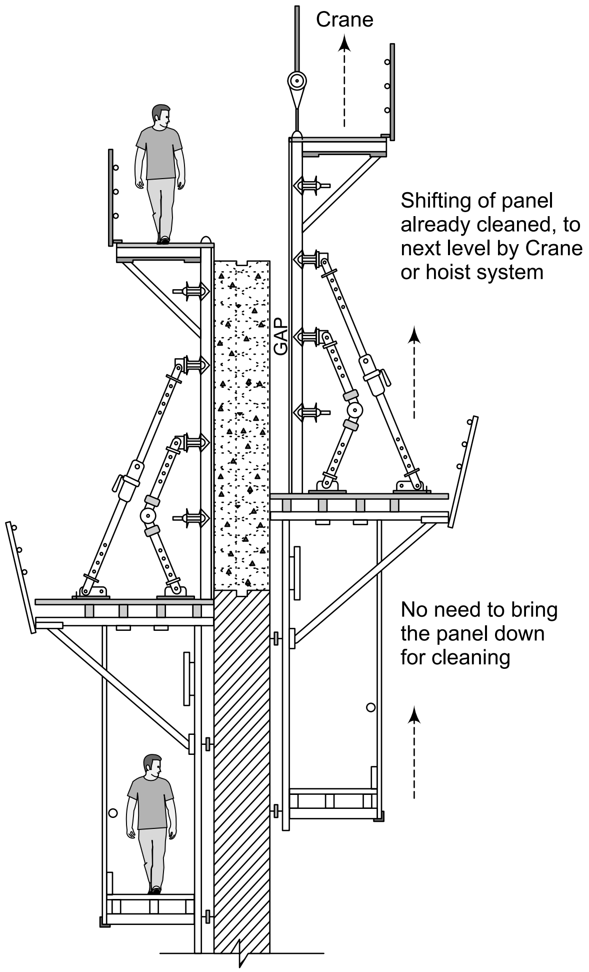


Figure 5.5(c) Shifting of Wall Forms With the Help of a Crane.

Type 3:

This is the most advanced version of climbing formwork. The system is known as *Automatic Climbing Formwork* or *Self-Raising Formwork*. In this formwork system, there is no requirement of a crane for any formwork related operations. The wall form panels need not be brought to the ground for cleaning. The cleaning and other formwork activities are performed at elevated locations itself. The climbing part is taken care by a special device called climber, (see Fig. 5.6) which hoists up the formwork assembly step by step. With standard components used in wall forming, the automatic climbing formwork system can be customized for a wide range of applications. This type of climbing formwork system can be very advantageously used for construction of large tall structures like Pylons, Piers, Lift Shaft, Natural Draught Cooling Towers, Dam Faces etc. The system is getting popular these days due to its independence from the crane and due to increased speed of construction. The form panels could be made of large plywood sheets fixed on metallic or aluminum frames or they could be made up of metal sheets fixed on metal frames.

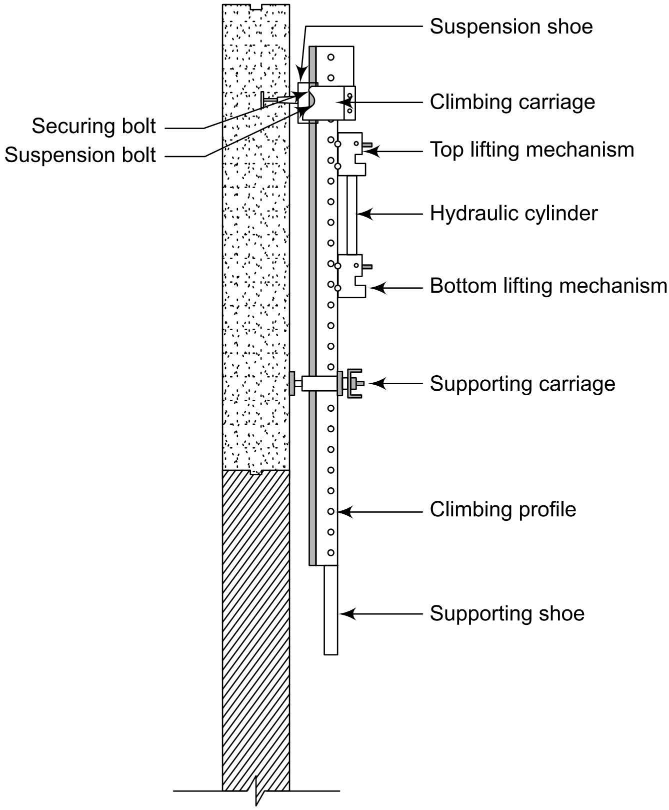


Figure 5.6 A Typical Climber Device.

The following components are in addition to what is used in the normal wall formwork.

Lifting frames

Bottom support of the lifting frame is anchored to the previously cast concrete lift. The hydraulic jacks are attached to the lifting frames. The jacks pull the entire form system to the next lift location.

Platforms at different levels

In an automatic climbing form, a number of working platforms are available for different functions. The lowest platform is used for repair and minor touch-up works of already cast concrete besides being useful to the worker engaged in curing. The next upper level platform is used by the carpentry gang to clean, fix, align, and remove the form panels. The platform above this is used by the reinforcement crew and concrete pouring crew. The interesting thing about these platforms is that the complete wall form along with these platforms, move together and the platforms need not be dismantled after every concrete pour.

Hydraulic jacks

The hydraulic jacks are attached to the lifting frames. The jacks pull the entire form system to the next level. A wide range of hydraulic and electric powered jacks are available. The spacing between the jacks is dependent on the capacity of the jacks and the load to be lifted.

The climbing cycle sequence is explained with the following diagrams.

Stage 1

Formwork on one side is set. The leading anchor is positioned and secured. Reinforcement work is completed. The arrangement corresponding to stage 1 is shown in Fig. 5.7.

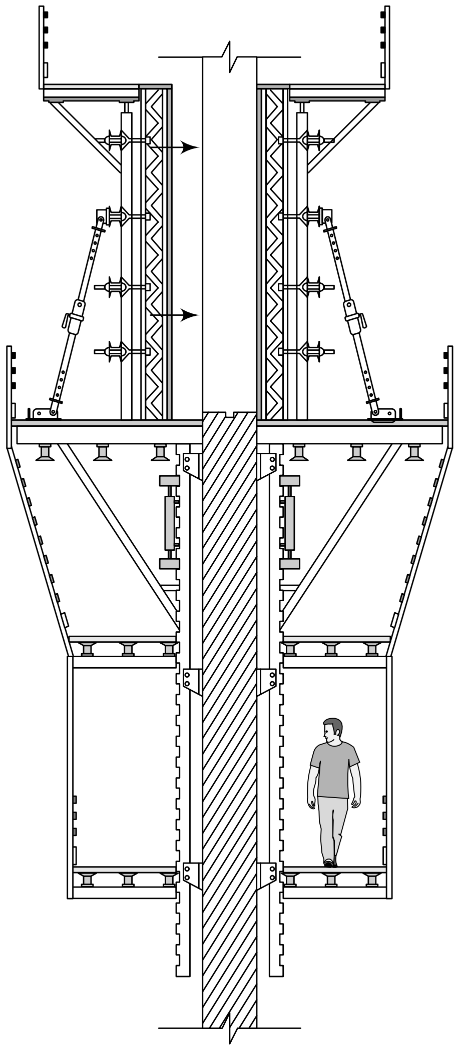


Figure 5.7 Arrangement Corresponding to Stage 1.

Stage 2

Formwork is closed. Concrete is poured. The arrangement at this stage is shown in Fig. 5.8. After the concrete has attained sufficient strength, the striking of formwork is commenced.

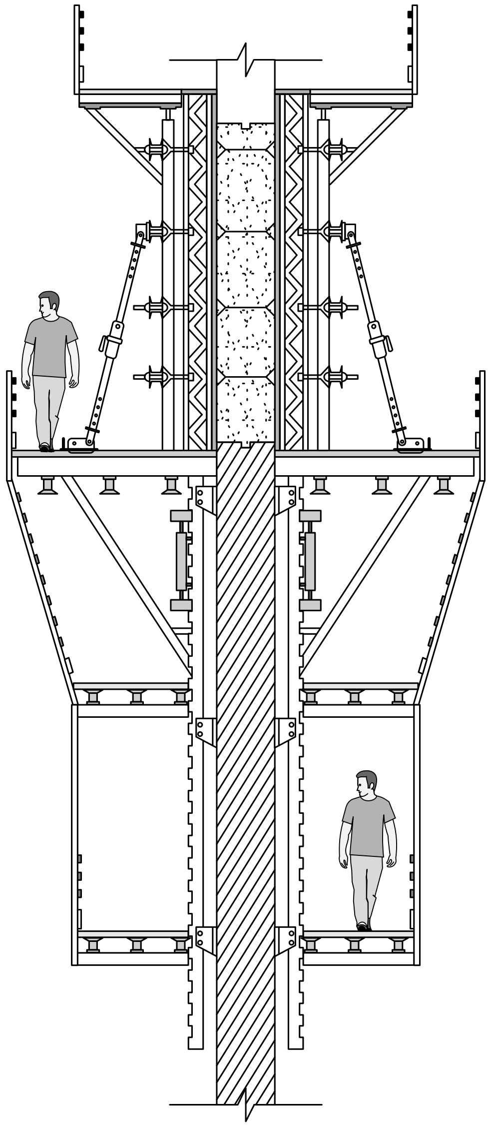


Figure 5.8 Arrangement Corresponding to Stage 2.

Stage 3

Climbing shoe is mounted. Climbing operation starts. The corresponding arrangement is shown in Fig. 5.9.

Stage 4

Formwork for one side is set. Leading anchor is positioned and secured. Formwork is closed. The arrangement is shown in Fig. 5.10.

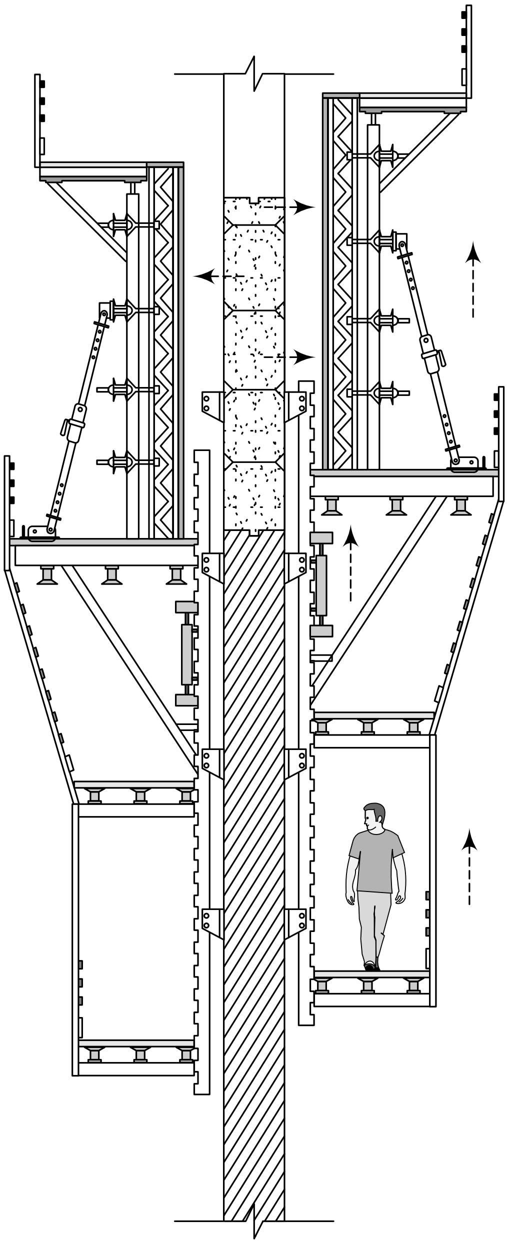


Figure 5.9 Arrangement Corresponding to Stage 3.

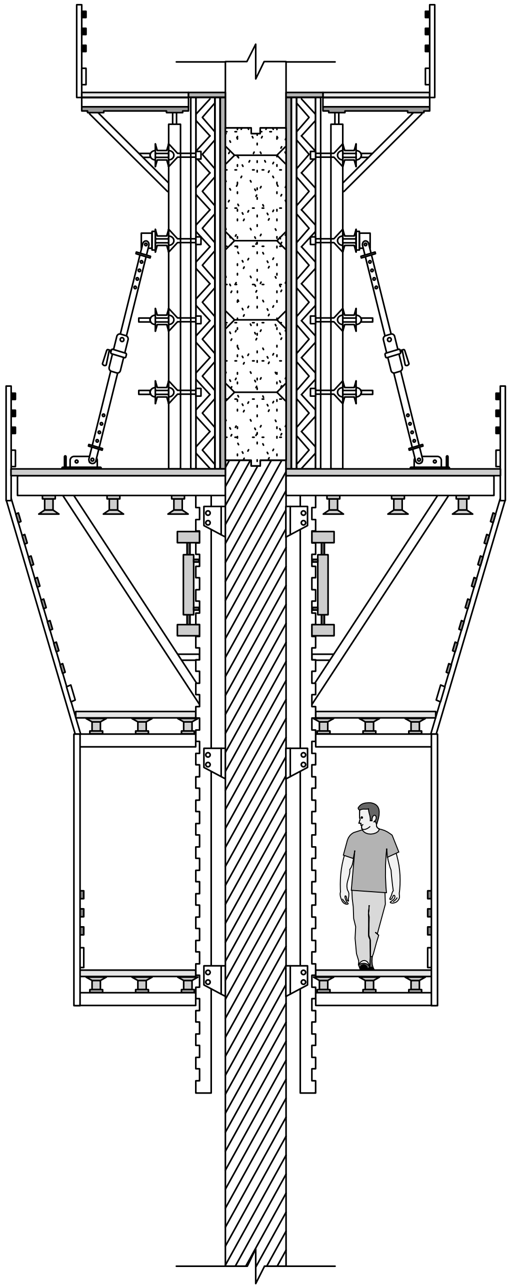
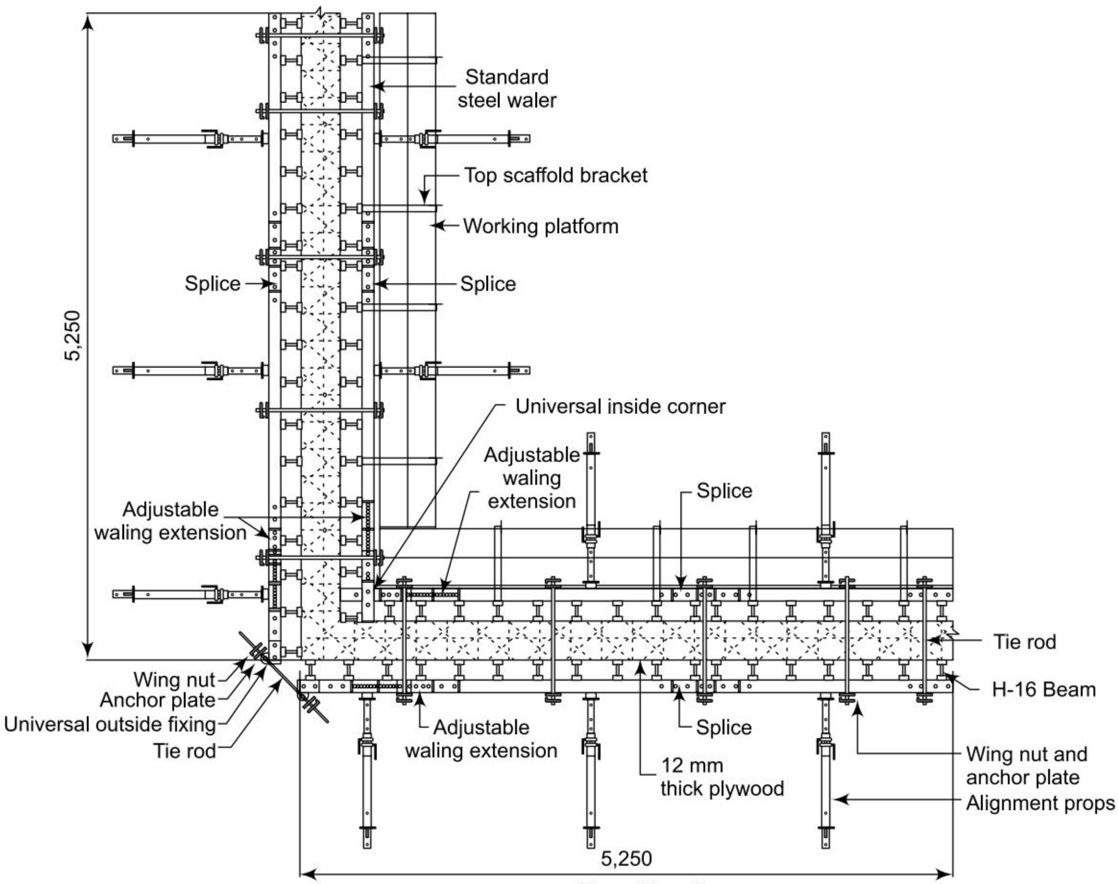


Figure 5.10 Arrangement Corresponding to Stage 4.

5.6 L&T WALL FORMWORK

L&T wall formwork system combines timber beams with steel walers and has standard panels which can be linked with other panels by connecting bolts and element jointers. Standard brackets fixed to H-16 beams are used to make the working platform for concreting/vibration etc. The sheathing panel is made of plywood of 12 mm to 19 mm thickness. The sheathing member is supported by H-16 beams placed at suitable spacing depending on the concrete pressure on the formwork. The depth of the H-16 beam is 160 mm. The H-16 beam is made up of plywood and timber in a factory and can be used for a large number of times. The steel walers are composed of two channel sections placed back to back. Special devices are used to connect the H-16 beams with steel walers. For alignment of wall formwork, props are required. There is a provision to fix the lifting device which can be useful during transportation of the wall formwork. Similarly, to install working platforms, there are arrangements in the wall form panels. The plan and sectional elevation of L&T wall formwork arrangement are shown in Fig. 5.11. Figure 5.12 shows the erection of straight wall formwork in progress, at one of the project sites using L&T wall formwork.



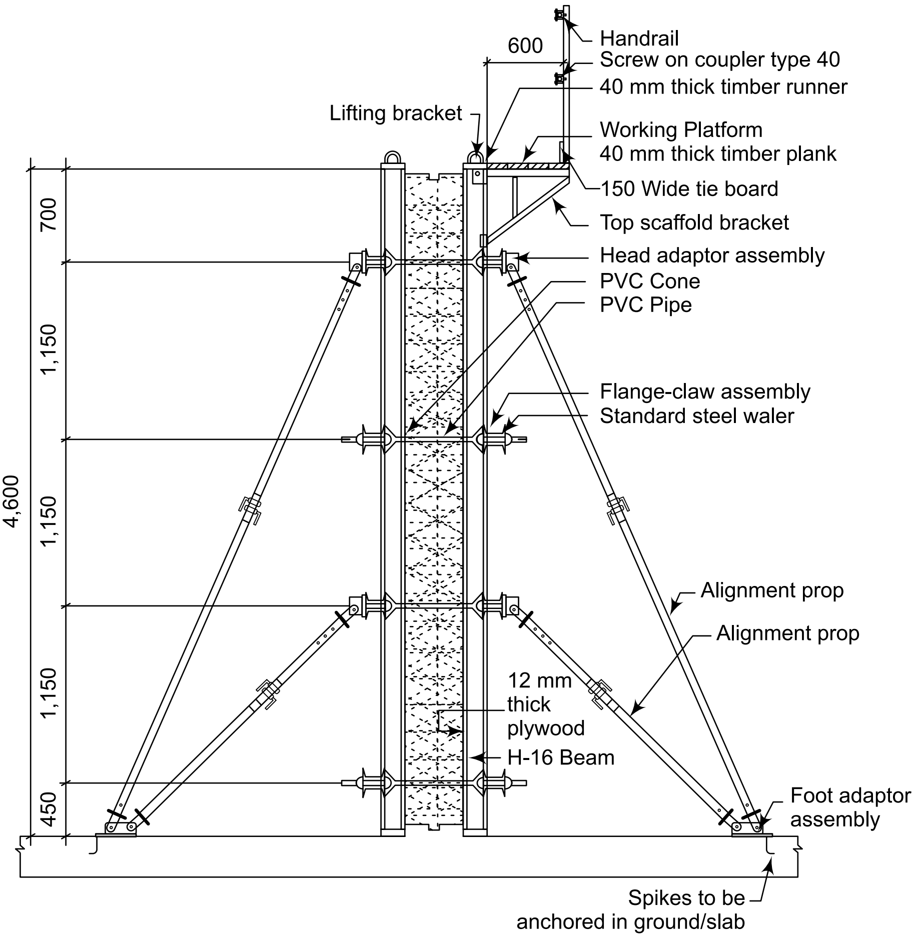


Figure 5.11 Plan and Sectional Elevation of L&T Wall Formwork.

Figure 5.12 Field Application of L&T Wall Formwork System.

In Fig. 5.13, retaining wall construction for a stadium using L&T wall formwork system is shown. A number of steel walers along the wall height can be noticed. The single pour height is of the order of 5-6 m. The tubular props and the foot adaptor assembly can also be noticed. It is possible to cast the wall with slight curvature without any additional component as is clear from Fig. 5.13.

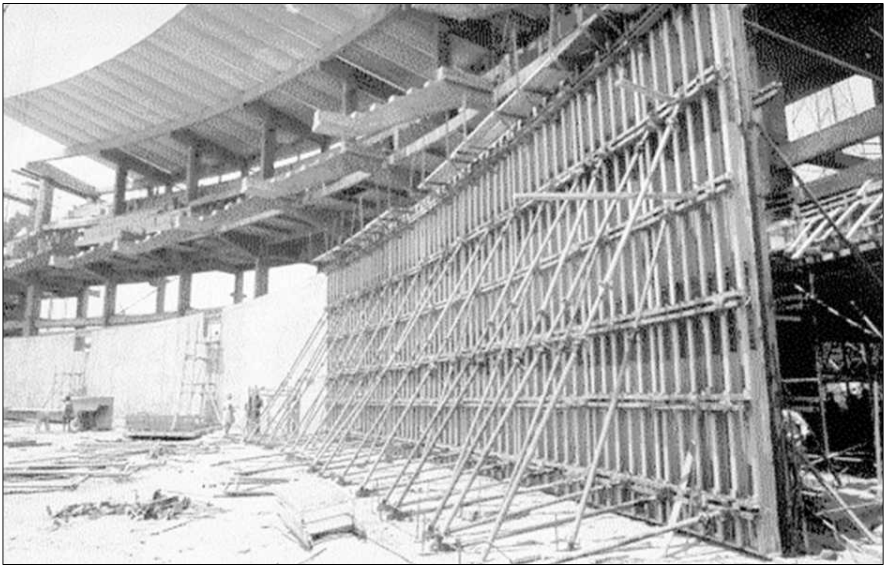


Figure 5.13 Construction of Retaining Wall Using L&T Wall Formwork.

Circular walls can also be formed using the L&T wall formwork components as is shown in Figs. 5.14 and 5.15. The contractor (AFCONS Limited) used L&T wall formwork for casting the caisson.

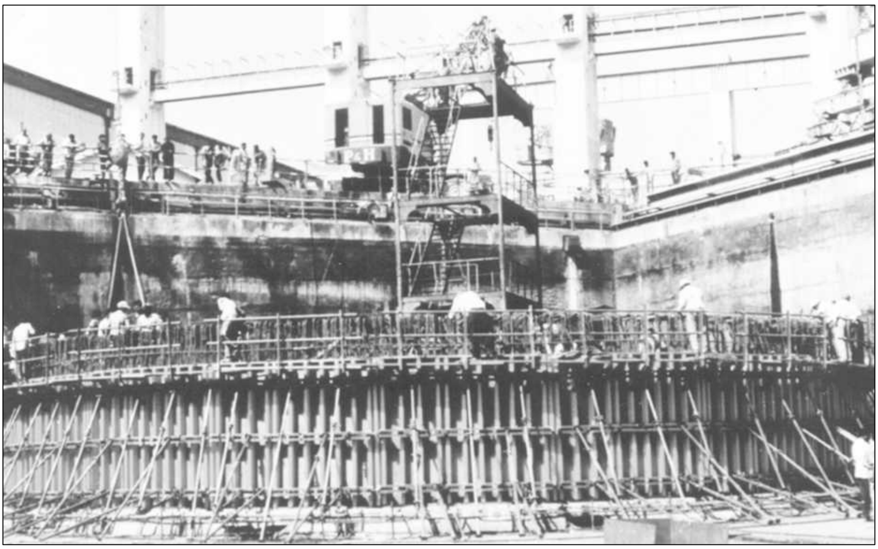


Figure 5.14 Shoe of Caisson for Intake Structure Being Cast in the Dry Dock, Visakhapatnam.

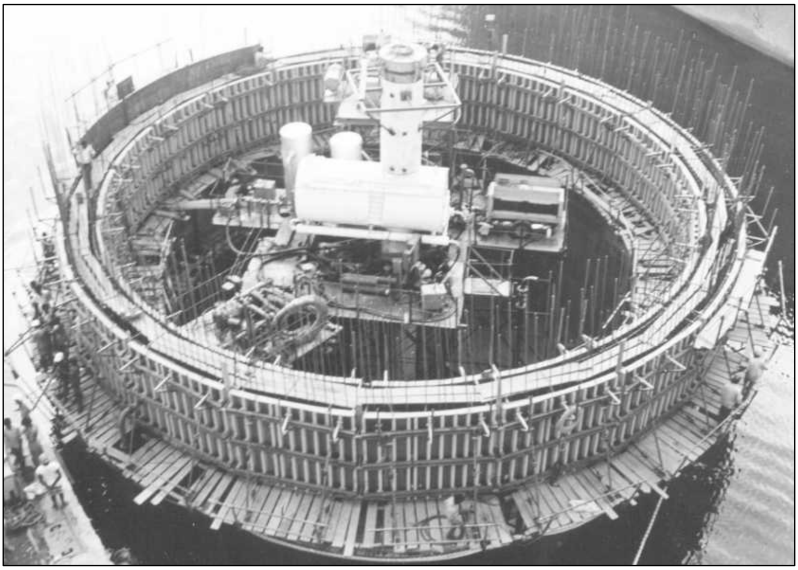


Figure 5.15 Caisson for Intake Structure Being Floated into Deep Sea, Visakhapatnam.

5.7 PERI WALL FORMWORK

Some of the solutions for wall formwork offered by PERI are:

1. Maximo Panel Wall Formwork;
2. Trio Panel Wall Formwork;
3. Handset Panel Wall Formwork;
4. Vario GT 24 Girder Wall Formwork;
5. Rundflex Circular Wall Formwork; and
6. GRV Circular Wall Formwork.

The features of each of these wall formworks are given in the following sections.

5.7.1 Maximo Panel Wall Formwork

PERI offers Maximo panel wall formwork for speedy construction. The system has a conical tie system which allows the wall forming to proceed without requiring spacer tubes. The work takes place from only one side; besides, conventional tie sleeves are not necessary in this system. These features result in saving in time and cost. The uniform pattern of ties also helps in improving the appearance of concrete surface after the removal of forms. A typical Maximo panel wall formwork is shown in Fig. 5.16.

5.7.2 Trio Panel Wall Formwork

The Trio panel wall formwork has very few parts which allow the form to be assembled and dismantled very fast. The Trio panel formworks have few components, and thus the time taken to form is relatively lower. There are many variants of Trio panel formwork. Each of the variants has certain specialized functions. These are described briefly.

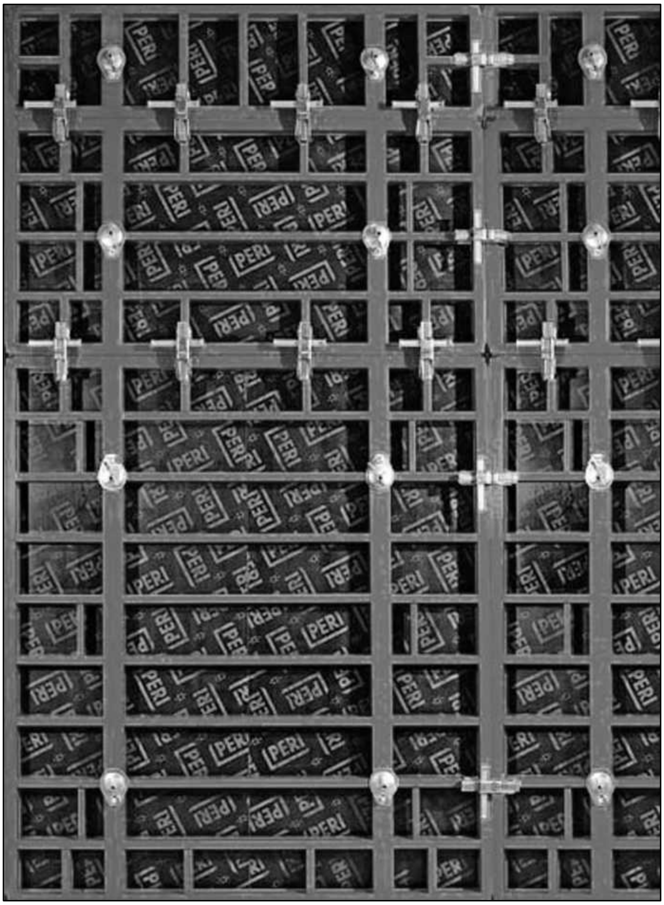


Figure 5.16 Maximo Panel Wall Formwork (Courtesy PERI).

Trio Housing Panel Formwork is the fastest among all the variants of Trio panel wall formwork. This system has a very short cycle time. The plywood board can be screwed on the panel in such a way that there are no indentations of screws and nails left on the concrete surface. The ties can be installed from one side only. The panels could be formed up to 5,400 mm height. There is only a single tie required for concreting a height of 2,700 mm. The form panel is designed to withstand a concrete pressure of 67.5 kN/m^2 for a concreting height of 2,700 mm. Figure 5.17 shows a typical Trio housing panel formwork. The form system is equipped with a working platform and a ladder.

Trio Aluminum Panel Formwork (see Fig. 5.18) is suitable for projects where cranes are not available for formwork operations. The panels in this system of formwork are light in weight and thus can be easily handled by manual efforts. Trio aluminium panel formwork is powder coated and uses the same accessories as used in other Trio steel panel formwork.

Figure 5.17 Trio Housing Panel Form (Courtesy PERI).

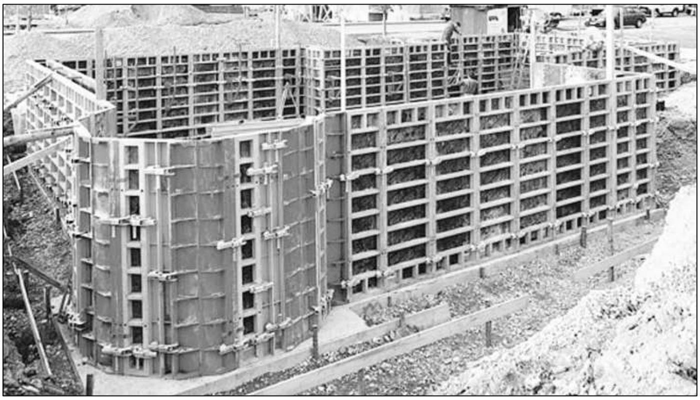


Figure 5.18 Trio Aluminum Panel Formwork (Courtesy PERI).

Trio 330 Panel formwork (see Fig. 5.19) is suitable for wall heights up to 3,300 mm. Two ties are required for this height. The form panels can withstand a concrete pressure of 82.5 kN/m^2.

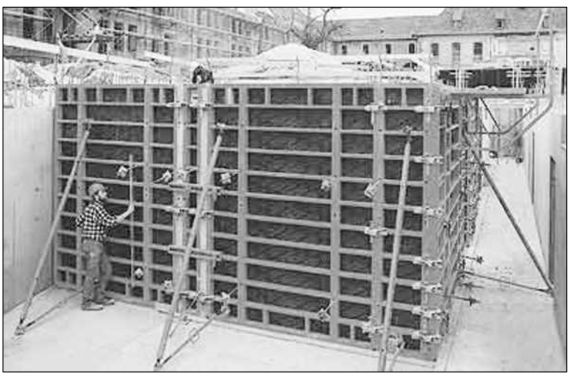


Figure 5.19 Trio 330 Panel Formwork (Courtesy PERI).

In addition to the above mentioned variants of Trio panel wall formwork, there are other variants. For example, Trio structure formwork system (see Fig. 5.20) is useful for situations where farefaced concrete finishes are required; Trio platform formwork system (see Fig. 5.21) provides the highest level of safety, and so on.

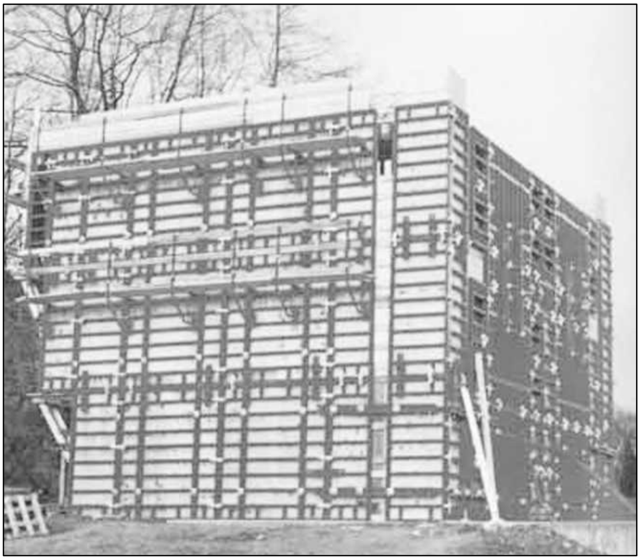


Figure 5.20 Trio Structure Formwork System (Courtesy PERI).

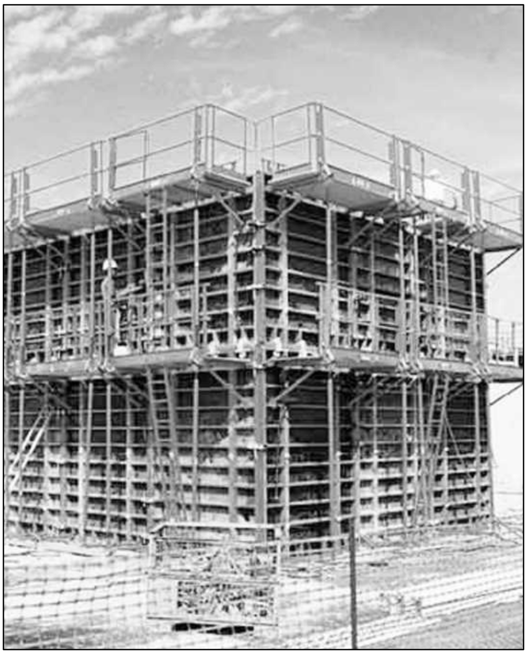


Figure 5.21 Trio Platform Formwork System (Courtesy PERI).

5.7.3 Domino Panel Wall Formwork

The Domino panel wall formwork system consists of formwork elements made up of steel or aluminium. The system is very light and thus suitable for housing and civil engineering applications. The system is useful for wall formwork as well as foundation formwork. The panels are available in four widths: 1,000 mm, 750 mm, 500 mm, and 250 mm. The panels of up to 2,500 mm height are designed to withstand full hydrostatic pressure. For heights of more than 2,500 mm, the permissible pressure on form panels is limited to 60 kN/m^2. Powder coating of various formwork elements make it easier for the system to be cleaned. Figure 5.22 shows a typical Domino panel wall formwork.

Figure 5.22 Domino Panel Wall Formwork (Courtesy PERI).

5.7.4 Handset Panel Wall Formwork

The wall formwork panels offered in this system of formwork are very light and thus can be easily handled by manual efforts. The panels can withstand concrete pressure up to 40 kN/m^2. The system has panels of only three variants of heights and widths which ensure high utilization percentage of the form. The system uses a special clip which is capable to connecting all elements and accessories such as standard panel joints, panel joints with height offsets, external corners, internal corners, articulated corners, and extensions. A typical handset panel wall formwork is shown in Fig. 5.23.

5.7.5 Vario GT 24

Vario GT 24 wall formwork system is a versatile system formwork useful for a wide range of applications, such as building construction, industrial construction, bridge abutments, retaining walls, and so on. It is possible to form any ground plan and any height using this system. The system has continuously adjustable element connections for all designs and applications. The system offers a lot of flexibility in selecting the type and size of plywood, element widths and heights, the vertical and horizontal tie arrangement, and the permissible concrete pressure. The system offers a working platform for safe workings at all heights. A typical Vario GT 24 panel formwork is shown in Fig. 5.24.

Figure 5.23 A Typical Handset Panel Wall Formwork (Courtesy PERI).

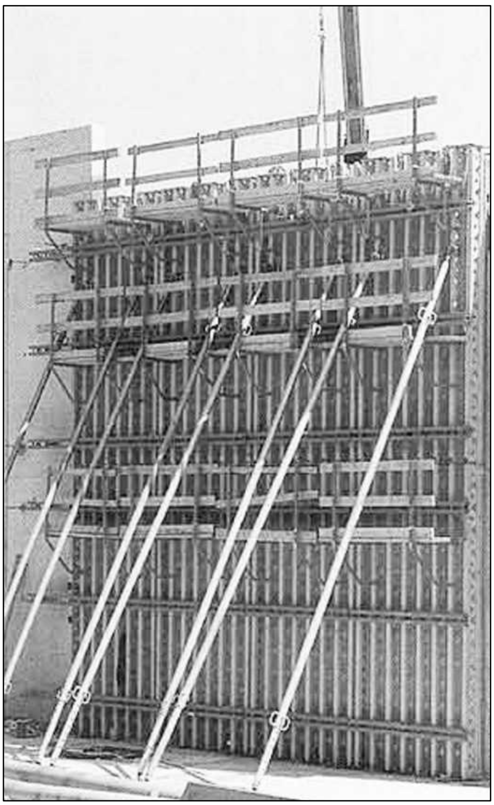


Figure 5.24 A Typical Vario GT 24 Panel Formwork (Courtesy PERI).

5.7.6 PERI Rundflex

PERI Rundflex wall formwork system is a versatile system formwork, useful for structures having a curvature, such as sewage treatment plants, ramps for multi-story car parks, silos, and other circular structures. The elements of the system are designed for adjustments to suit various curvatures, starting with 1,000 mm radius using standard components. The form panels can be quickly connected by means of a specially designed coupler. The formwork has been designed to withstand a concrete pressure of 60 kN/m². The system has only three different element widths for all uses in five corresponding heights. A typical Rundflex wall formwork system is shown in Fig. 5.25.

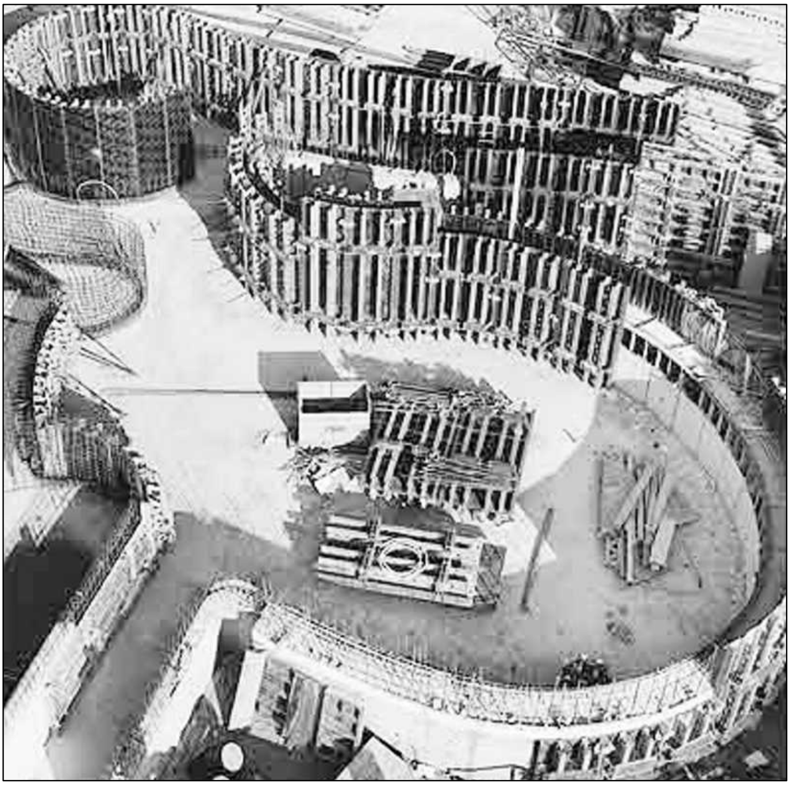


Figure 5.25 A Typical Rundflex Wall Formwork System (Courtesy PERI).

5.7.7 GRV Circular Wall Formwork

The system offers a cost-effective solution to the formwork needs of semi-circular arch, segmental arch, and other arched forms. The system does not require any tie irrespective of whether being used for small or large radii. The system can be used for forming walls having radius as low as 600 mm. The system hinges on a very specialized waler which is capable to resist 300 kN of tension or compression force when in a closed circle. A field application of GRV circular wall formwork can be seen in Fig. 5.26.

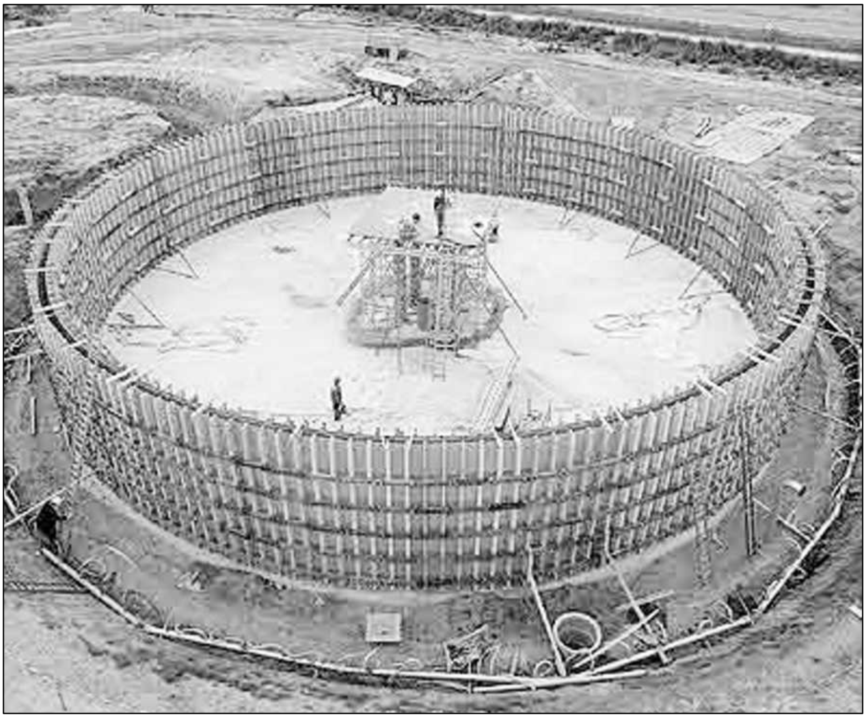


Figure 5.26 GRV Circular Wall Formwork (Courtesy PERI).

5.8 PERI CLIMBING FORMWORK

PERI offers different climbing formwork solutions. Some of them are discussed below:

5.8.1 CB 240 and CB 160 Climbing Systems

In CB 240 system, the formwork is mounted on a carriage, and can be moved around 750 mm. In CB 160 system, the formwork is simply tilted backward when striking. Figure 5.27 shows the arrangements of CB 240 and CB 160 climbing systems. In both the systems, the formwork is moved to the next pour, together with the scaffold, in one crane lift (see Fig. 5.28).

5.8.2 RCS Formwork

RCS formwork system is shown in Fig. 5.29. In this system, the formwork is supported on a mobile carriage which can be retracted up to a distance of 900 mm. The system has an optional self-climbing technology. This system can be climbed either with a crane or lifted by means of mobile hydraulic climbing devices. In this system, a component called climbing shoe guides the climbing rail to the next casting segment. The climbing pawl engages automatically and secures the complete unit after lifting for 500 mm.

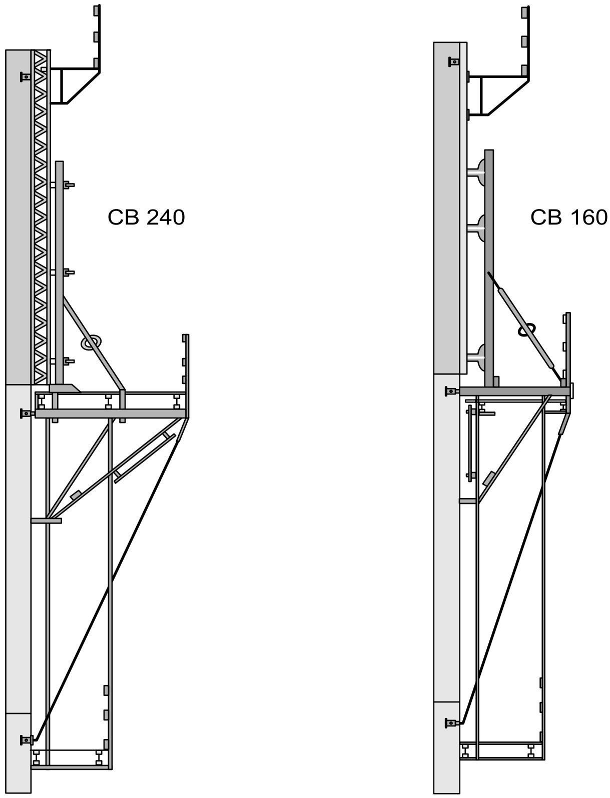


Figure 5.27 CB 240 and CB 160 Systems (Courtesy PERI).

Figure 5.28 CB System Being Transported Through Crane (Courtesy PERI).

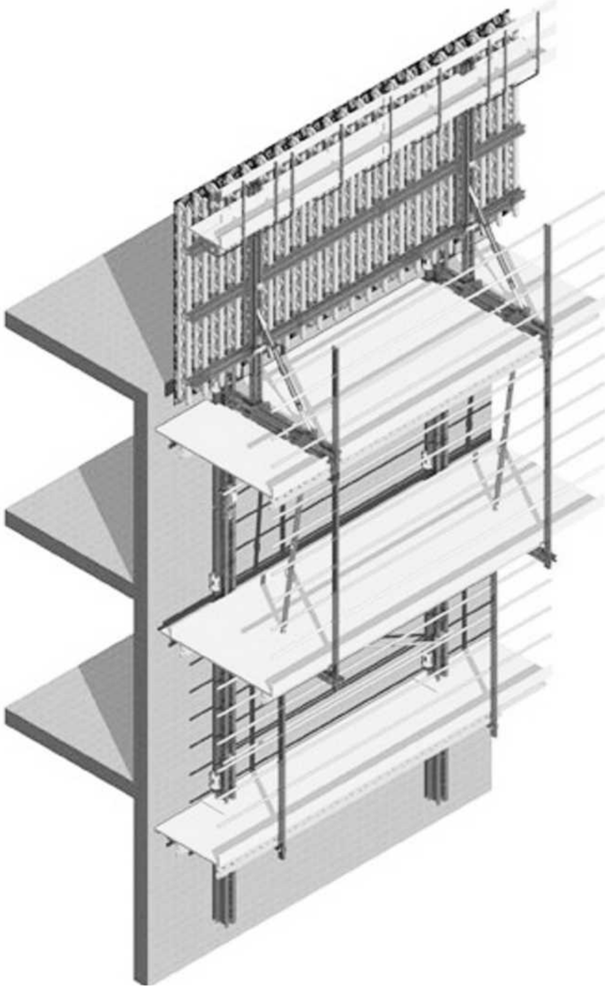


Figure 5.29 RCS Formwork (Courtesy PERI).

5.8.3 ACS— Automatic Climbing System

This system has a climbing mechanism with a lifting power of 100 kN, which raises the climbing unit to the next pour without the need for intermediate anchors.

ACS formwork has three main variants. These are:

ACS R: The arrangement of ACS R is shown in Fig. 5.30. The system is useful in all the regular climbing applications.

ACS P: The arrangement of ACS P is shown in Fig. 5.31. The system is useful for complicated building cores and large working platforms.

ACS G: The arrangement of ACS G is shown in Fig. 5.32.The system is useful for circular buildings and monolithic concreting of slabs and walls.

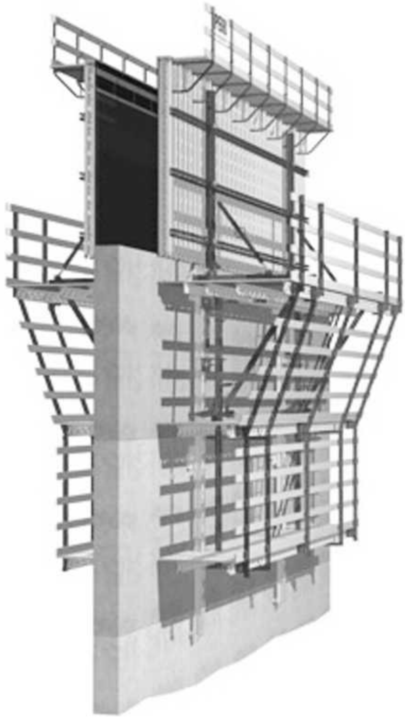


Figure 5.30 ACS R Formwork System (Courtesy PERI).

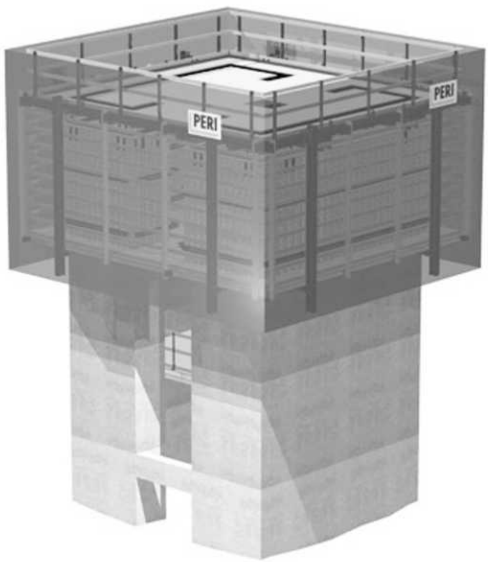


Figure 5.31 ACS P Formwork System (Courtesy PERI).

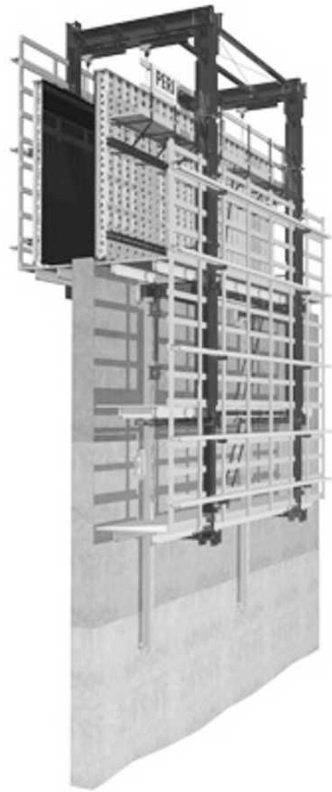


Figure 5.32 ACS G Formwork System (Courtesy PERI).

5.9 DOKA CLIMBING FORMWORK

Doka provides three types of climbing formwork with various degrees of rationalization and mechanization. These are:

1. Climbing Scaffold CB-150A;
2. Traveling Climbing Formwork CB-150F and MF-240; and
3. Automatic Climbing Formwork SKE-50.

5.9.1 CB-150A

This is a crane-assisted, simple climbing wall formwork, and is referred to as a CB-150A system. The system consists of (1) standard wall panels, (2) adjustable props for alignment, (3) standard brackets of 1.5 m width with a provision for extension of width, and (4) suspended working platform. The wall form is brought to the ground with the help of a crane after completion of each lift. This is essentially for cleaning of wall formwork and making it ready for the next lift. The bracket is also dismantled after each use and fixed to the next location with the help of a crane. The details of CB-150A climbing formwork system is shown in Fig. 5.33.

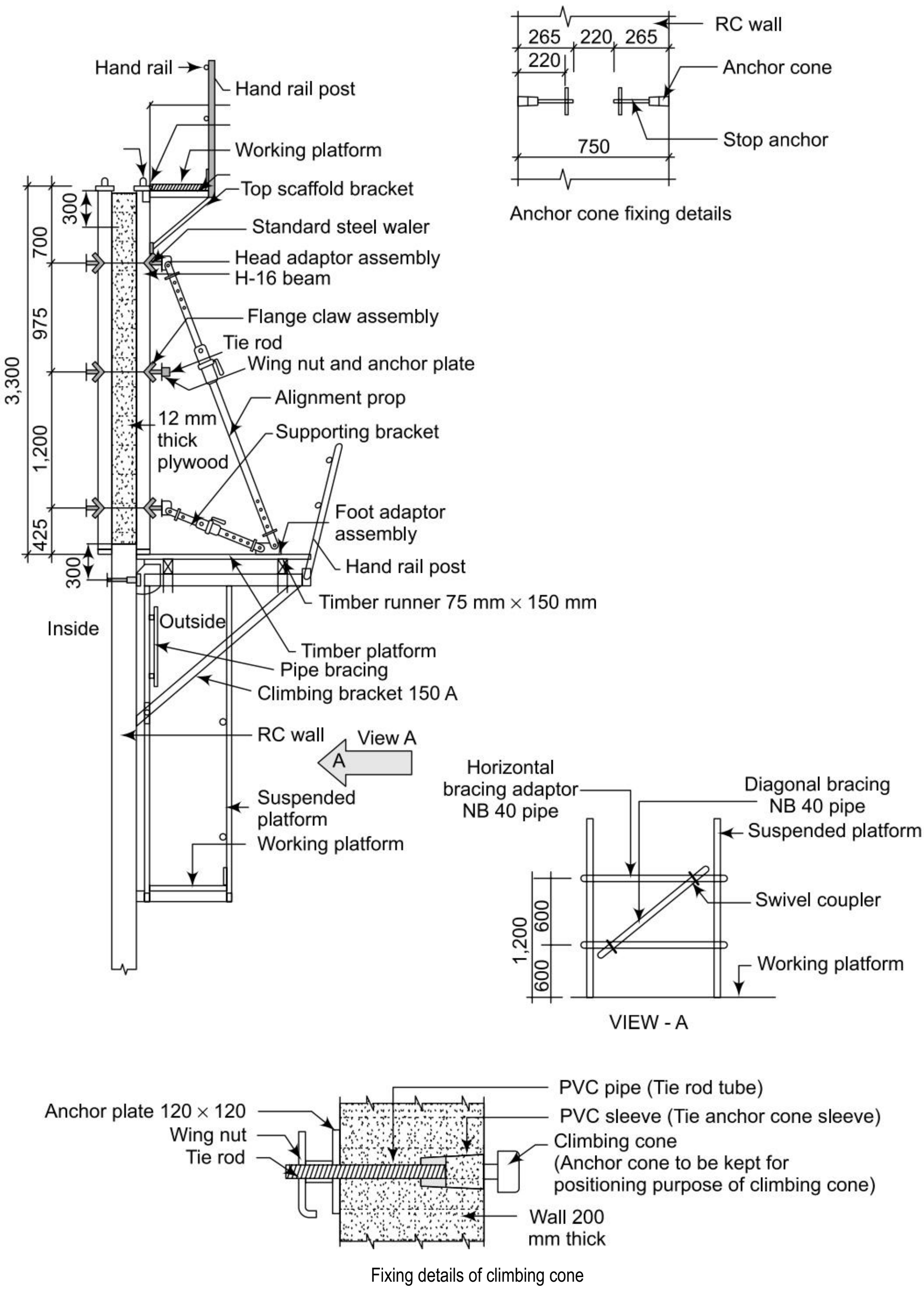


Figure 5.33 Typical Details of Climbing Formwork CB-150A.

5.9.2 CB-150F and MF-240

CB-150F is a traveling climbing formwork, in which wall form is fixed to a vertical walling and scissors action spindle. On completion of a lift, the wall form can be rolled back with scissors action spindle up to 750 mm away from the concrete surface for cleaning. After cleaning, the wall formwork and climbing bracket can be lifted as a unit and fixed to their next position by crane. Thus, lowering of the wall form to the ground level for cleaning purpose is avoided. The platform width in CB-150F is 1.5 m. There is a provision for extension of bracket width beyond 1.5 m.

MF-240 formwork system is also a crane-handled, traveling climbing formwork. The platform width in this system is 2.40 m. In MF-240 system, the brackets can be fitted with automatic climbers like SKE-50 which is a hydraulic system (50 kN capacity). Figure 5.34 shows the schematic arrangement of CB-150F.

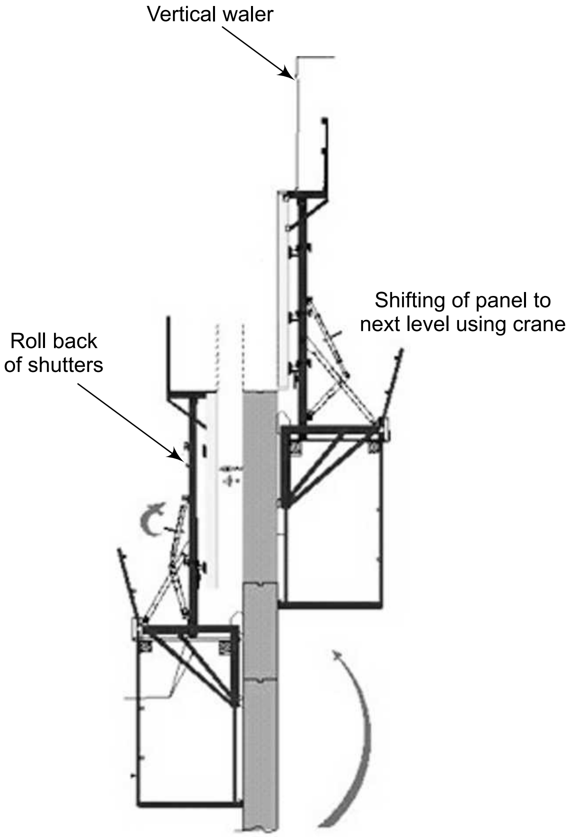


Figure 5.34 Travelling Climbing Formwork System.

The rollback arrangement through which a gap is created can be seen in Fig. 5.34. In Fig. 5.35, the different components used in a CB-150F formwork system are clearly shown. It can be noticed that most of the components are same as that used in ordinary wall formwork.

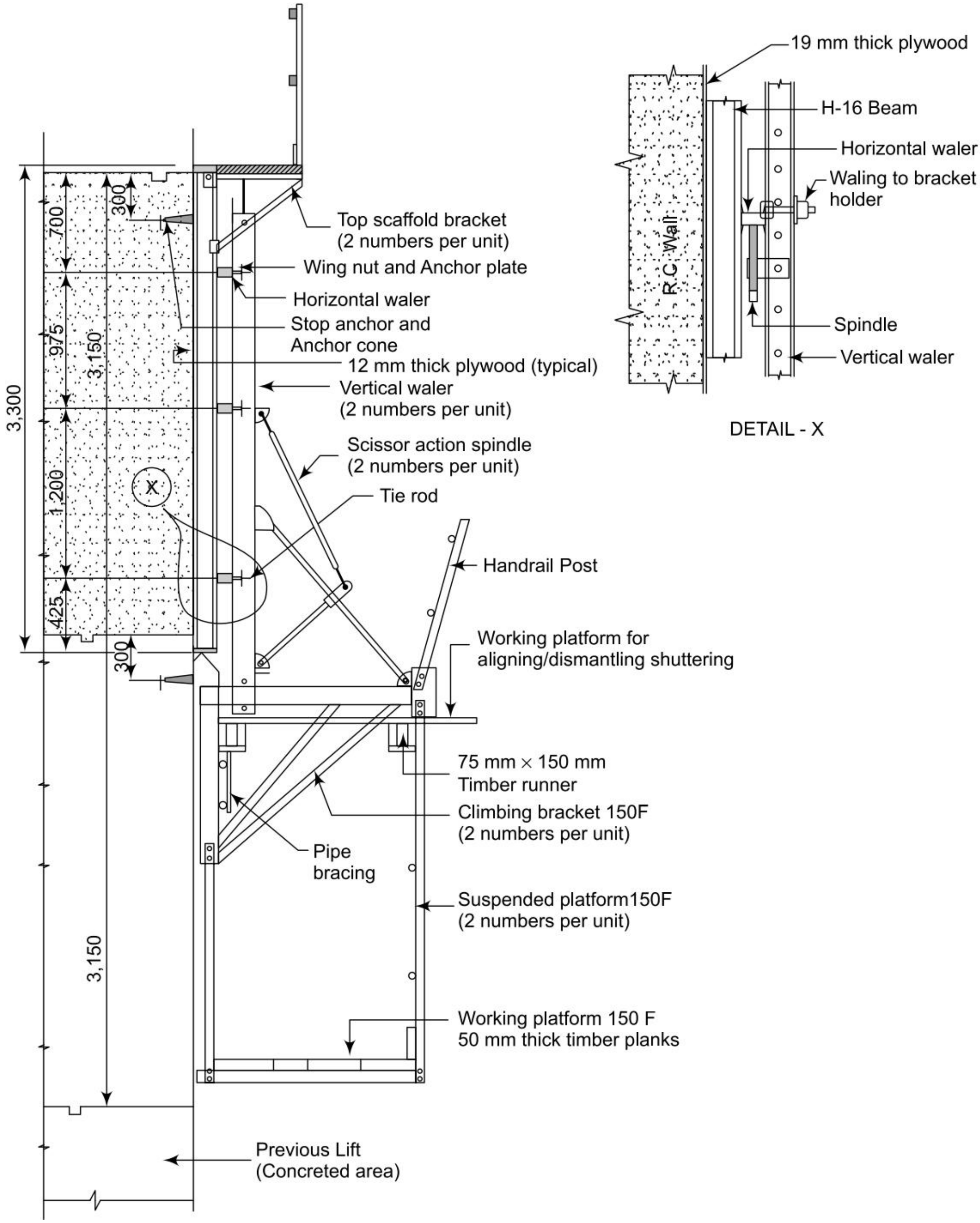


Figure 5.35 A Typical CB-150F Formwork System.

The application of traveling climbing formwork CB-150F is shown in Figs. 5.36 and 5.37. The contractor used CB-150F for the construction of an LNG tank of 80 m diameter and 40 m height. Shutter panels of size 4.0 m x 3.8 m were used for the wall formwork construction.

Figure 5.36 View of CB-150F in Use For Construction of an LNG Tank (Courtesy L&T Formwork).

Figure 5.37 Another View of CB-150F in Use For Construction of an LNG Tank (Courtesy L&T Formwork).

5.9.3 SKE System

Doka offers SKE system, which is an automatic climbing formwork. In this system, climbing machines (climbers) are combined with the standard wall form and working platforms (see Fig. 5.38). The system has provision for a number of working platforms at different levels to be used by different crew categories. The climbers automatically hoist up the wall formwork along the wall, step by step, using hydraulics. Only for the first few lifts, which are done in a conventional manner, a crane may be needed for formwork erection. Once the climber starts operating, the system does not require a crane for formwork operations, thereby cutting down crane cost on formwork operations. The system has appropriate safeguards against wind, and work can continue even in stiff weather conditions, which helps in adhering to tight control of schedules and costs. Different variants of SKE formwork system are available, such as SKE 50, SKE 50 Plus, and SK 175. The system can be very advantageously used for core walls in high rise structures, cable stayed bridge pylons, large tall structures like natural draught cooling towers, etc. SK 175 is an automatic climbing formwork solution for constructing natural draught cooling towers. The detailed discussion on this formwork is available in Chapter 8 of the book.

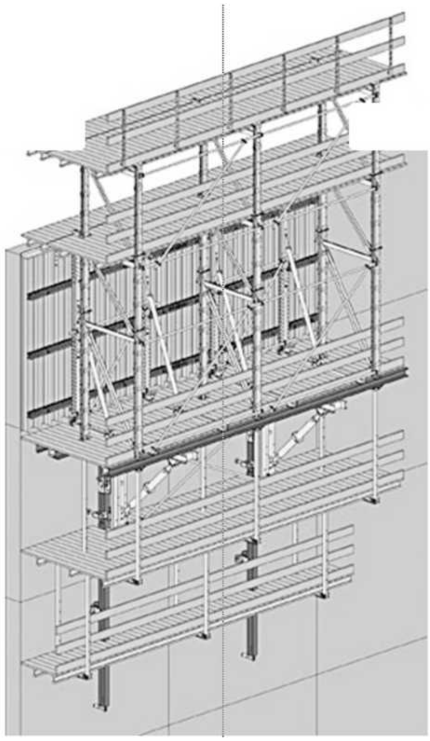


Figure 5.38 A Typical SKE System (Courtesy Doka).

5.10 WALL FORM DESIGN

The following steps are needed to perform the design of conventional wall formwork.

Determine the pressure on the form: The lateral pressure is determined from any of the methods listed in the previous chapter. As already explained, the lateral pressure would depend on the height of the wall, the rate of filling forms, temperature, etc.

Sheathing design: There could be two possibilities. In one case, the thickness of the sheathing is given and in another case, it may not be given. In the first case, we straight away move to the next step of designing the stud. For the second case, sheathing thickness is assumed (for example, 12 mm thick plywood, or 19 mm thick plywood) and the spacing of studs is determined as given in the next step.

Design of stud: For designing the stud, given the thickness of plywood, we perform the following steps:

For a given thickness of plywood, the permissible bending stress and shear stress are found out from the relevant Indian Standards or from the manufacturer's data and the span for the plywood (same as the spacing of the studs) is found out. In Table 5.1, approximate values for allowable bending moment, shear force, and deflection for 12 mm, and 19 mm thick plywood are provided.

Table 5.1 Allowable Bending Moment, Shear, and Deflection for 12 mm and 19 mm Thick Plywood

Criteria	Type of material			
	12 mm thick plywood	19 mm thick plywood	H-16	Steel waler
Allowable bending moment (kNm/m)	0.2	0.34	3	10.2
Allowable shear (kN/m)	6.16	9.75	6	103.4
Allowable deflection	L/360 or 1.6 mm	L/360 or 1.6 mm	L/360 or 6 mm	L/360 or 3 mm
Permissible EI value	1.07 kNm^2	2.73 kNm^2	145 kNm^2	784.14 kNm^2

Based on the above values, the span of the plywood (spacing of the studs) can be computed by referring to Table 5.2.

Table 5.2 Formula for Spacing Based on Bending Moment, Shear and Deflection for Continuous Ends Conditions

Material	Bending Moment	Shear	Deflection L/360
12 mm thick plywood	$\sqrt{\frac{2.0}{w}}$	$\frac{9.86}{w}$	$\left(\frac{145EI}{360w}\right)^{\frac{1}{3}}$
19 mm thick plywood	$\sqrt{\frac{3.4}{w}}$	$\frac{15.6}{w}$	$\left(\frac{145EI}{360w}\right)^{\frac{1}{3}}$
H-16	$\sqrt{\frac{3.0}{w}}$	$\frac{9.6}{w}$	$\left(\frac{145EI}{360w}\right)^{\frac{1}{3}}$

Table 5.3 Formula for Spacing Based on Bending Moment, Shear and Deflection for Simply Supported Ends

Material	Bending Moment	Shear	Deflection L/360
12mm thick plywood	$\sqrt{\frac{1.6}{w}}$	$\frac{12.32}{w}$	$\left(\frac{16EI}{75w}\right)^{\frac{1}{3}}$
19mm thick plywood	$\sqrt{\frac{2.72}{w}}$	$\frac{19.5}{w}$	$\left(\frac{16EI}{75w}\right)^{\frac{1}{3}}$
H-16	$\sqrt{\frac{24}{w}}$	$\frac{12}{w}$	$\left(\frac{16EI}{75w}\right)^{\frac{1}{3}}$

For example, for a 12 mm thick plywood, the spacing of the studs can be computed as below:

Based on bending moment for a load $w = 7$ kN/m^2,

$$\text{spacing} = \sqrt{\frac{1.6}{w}} = 478 \text{ mm}$$

Based on shear for a load $w = 7$ kN/m^2,

$$\text{spacing} = \frac{12.32}{w} = 1{,}760 \text{ mm}$$

Based on deflection limit of $L/360$, $w = 7$ kN/m^2, and $EI = 1.07$ kNm^2,

$$\text{spacing} = \left(\frac{16EI}{75w}\right)^{\frac{1}{3}} = 319 \text{ mm}$$

The minimum value of spacing obtained from above three conditions is chosen as the spacing of the studs.

At this point of time, there are two possibilities:

(i) We choose the cross section and material of the studs, and compute the permissible span of the studs (this is same as the spacing of the walers).

(ii) We decide on the spacing of the walers and thereby the span of the studs, and find the cross section and material of the studs.

The illustration for both the cases is given below:

(i) Let us choose timber studs of 50 mm × 100 mm cross section. The permissible stress values can be taken from IS: 883–1994 for exact calculation. However, for preliminary design, the permissible stress values can be taken from Table 5.4.

Table 5.4 Permissible Stresses in Indian Species of Timber

Classification	Trading Name of Timber	Basic Stress in N/mm^2					Modulus of elasticity N/mm^2
		Bending	Tension along grain	Compression		Shear parallel to grain	
				Parallel to grain	Perpendicular to grain		
Group A	Kongo/Sal	15.2	15.2	10.6	4.6	1.2	1.26
Group B	Casuarina/ Gurjan/ Benteak/ Sal/ Teak/ Kindal/ Laurel/Irul	10.2	10.2	6.3	1.8	0.9	1.12
Group C	Poon/ Deodar/ Mango/ Chir	7.0	7.0	5.6	1.7	0.6	1.77

(a) Bending Stress:

Extreme fibre stress, $$f = \frac{My}{I} \tag{5.1}$$

For rectangular beam, $$I = \frac{bd^3}{12} \tag{5.2}$$

Section modulus, $$z = \frac{I}{(d/2)} = \frac{bd^2}{6} \tag{5.3}$$

Therefore, $$f = \frac{M}{Z} = \frac{6M}{bd^2} \tag{5.4}$$

The allowable stress for timber can be obtained from IS:883–1994, and one can arrive at different combinations of b and d.

The value of M in the above expressions can be obtained from Table 3.11 of Chapter 3.

The permissible span of the studs based on bending would be computed by equating the permissible value of bending stress and the expression of f using relevant expression for M as given in Eq. (5.4).

(b) Horizontal shearing stress:

The maximum unit stress in horizontal shear in a rectangular beam is given by:

$$H = \frac{3V}{2bd} \tag{5.5}$$

The shear force is computed using the relevant expression of Table 3.11. The permissible span of the studs based on shear would be computed by equating the permissible value of shear stress and the expression of H using relevant expression for V as given in Eq. (5.5).

Deflection:

The expression for deflection ($5wl^4/384EI$ in case of U.D.L. simply supported condition) is equated with the permissible deflection to get the span of the studs.

The minimum value of spacing obtained from above three conditions is chosen as the span of the studs, which is the spacing for the wales.

Step 4 Design of the Wales

Design of the wales is performed for either of the two cases: (i) the tie rod spacing is known (ii) the cross section of the wales is known:

(i) ***the tie rod spacing is known***

In this case, the tie rod spacing is considered as the span of the wales. The load exerted from the studs, even though they are concentrated, is treated as uniformly distributed load. Depending on whether the walers are simply supported or continuous, the cross section of the walers is decided based on the permissible bending stresses, shear stresses, and deflection, whichever governs in a similar manner illustrated above for the design of the studs.

(ii) ***the cross section of the walers is known***

In this case, depending on the cross sectional property of the walers, the span of the walers is fixed based on bending moment, shear forces, and deflection considerations.

Step 5 Design of the Tie

There could be following possibilities:

The span of the waler is known, the load exerted on the tie rod can be computed by multiplying the concrete pressure with the area of load being taken care of by the tie rod. For this load, the tie rod can be designed using the following expression:

Capacity of tie rod required = *Area catered by one tie rod* × *Concrete pressure on formwork* × *Factor of safety* (5.6)

Step 6 Check for bearing

- Bearing of the studs on the wales
- Bearing between the tie washer or tie holders and the wales.

$$\textit{The actual bearing stress} = \frac{\textit{Maximum tie load}}{\textit{Bearing area}} \tag{5.7}$$

The various steps in the wall formwork design are given in the form of a flow chart in Fig. 5.39.

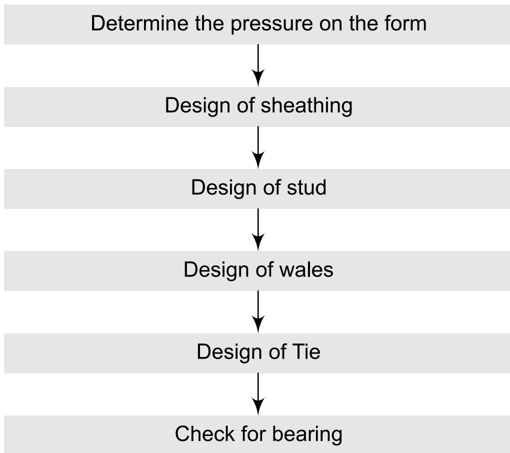


Figure 5.39 Steps in the Wall Formwork Design.

5.11 ILLUSTRATION OF WALL FORMWORK DESIGN USING PLYWOOD AND H-16 BEAMS

It is desired to check the adequacy of wall formwork shown in Fig. 5.40. As can be seen from the Figure, 12 mm thick plywood has been used as sheathing, H-16 @250 mm centre to centre distance has been used as studs. Walers have been used @1,200 mm centre to centre distance. Tie rods have been proposed @1,200 mm centre to centre.

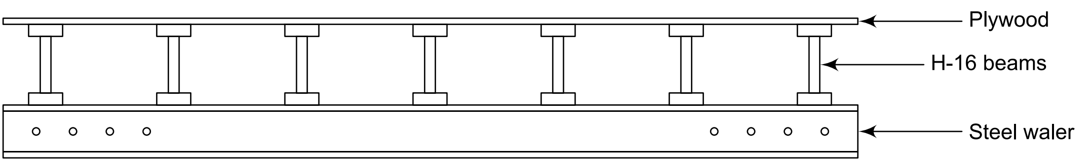


Figure 5.40 Proposed Wall Formwork.

The following design inputs are also available:

1.	*D*, weight density of concrete	25 kN/m^3
2.	*R*, rate of rise	1 m/h
3.	Temperature of concrete	25˚C

4. H, vertical form height	6.15 m
5. h, vertical pour height	6 m
6. Cantilever portion of H-16	0.4 m
7. C_1, shape constant	1
8. C_2, concrete constituent factor	0.3

For 12 mm plywood, consider the following:

Allowable moment carrying capacity	0.2 kNm/m
Allowable shear	6.16 kN/m
Permissible EI	1.1 kNm^2/m
Permissible deflection	0.730 mm

For H-16 beam, consider the following:

Depth of H-16 beam	160 mm
Flange of H-16 beam	65 mm
Allowable moment carrying capacity	3 kNm
Allowable shear	6 kN
Permissible EI	145 kNm^2
Permissible deflection = $L/360$	3.333 mm

The tie rod is rolled from 16 mm ST 58 (material specification) to form 18 × 5 pitch (thread detail). The yield stress the for tie rod is 360 N/mm^2.

5.11.1 Computation of Lateral Concrete Pressure on Formwork

Lateral concrete pressure (as per CIRIA formula) is given by:

$$P_{max} = D\left[C_1\sqrt{R} + C_2K\sqrt{H - C_1\sqrt{R}}\right]$$

Temperature co-efficient, $K = \left(\dfrac{36}{T+16}\right)^2 = \left(\dfrac{36}{25+16}\right)^2 = 0.77$

For the given values of D, C_1, R, C_2, H, and R

$$P_{max} = D\left[C_1\sqrt{R} + C_2K\sqrt{H - C_1\sqrt{R}}\right]$$

$$P_{max} = 25 \times \left[1 \times \sqrt{1} + 0.3 \times 0.77 \times \sqrt{6.15 - 1 \times \sqrt{1}}\right]$$

$$P_{max} = 38.12 \text{ kN/m}^2$$

Also from the expression: $P_{max} = Dh = 25 \times 6 = 150 \text{ kN/m}^2$

The P_{max} is considered as the minimum of the two values 38.12 kN/m^2 and 150 kN/m^2 as suggested by CIRIA.

Thus $$P_{max} = 38.12 \text{ kN/m}^2$$

$$\frac{P_{max}}{D} = 1.52 \text{ m}$$

The design pressure distribution corresponding to these values is shown in Fig. 5.41.

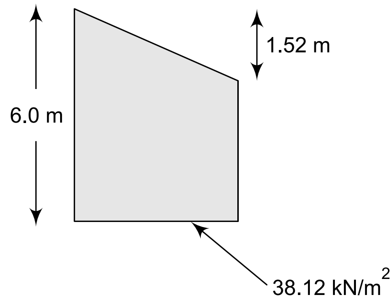


Figure 5.41 Design Pressure Distribution.

5.11.2 Check for Plywood

From Fig. 5.40, it can be seen that the plywood is supported by H-16 beams, which are spaced at 250 mm centre to centre.

Thus, the span of the plywood

l_p = clear span + thickness of plywood = (centre to centre distance of H-16 beams –2 × half of one H-16 flange width) + thickness of plywood = $\left(250 - 2 \times \frac{65}{2} + 12\right)$ = 197 mm

Check for bending moment

$$M = \frac{w \times l_p^2}{10} = \frac{38.12 \times 0.197^2}{10} = 0.148\ \frac{\text{kNm}}{\text{m}}$$ (less than 0.2 kNm/m, hence safe).

Check for shear

$$V = \frac{5 \times w \times l_p}{8} = \frac{5 \times 38.12 \times 0.197}{8} = 4.69 \text{ kN}$$ (less than 6.16 kN/m, hence safe).

Check for deflection

$$\delta = \frac{w \times l_p^4}{145 \times E \times I} = \frac{38.12 \times 0.197^4}{145 \times 1.1} = 0.00036 \text{ m} = 0.36 \text{ mm}$$ (less than 0.73 mm, hence safe).

Thus, the given spacing of the H-16, and hence the span of the plywood is safe in bending, shear, and deflection.

5.11.3 Check for H-16 Beams

The walers have been spaced at 1.2 m centre to centre, thus the span of the H-16 beams, l_h = 1.2 m. The span shall be checked for bending moment, shear, and deflection for its adequacy under simply supported condition.

Load on the H-16 beam = $w_h = w \times$ spacing of H-16 = 38.12 × 0.250 = 9.53 kN/m

Check for bending moment

$$M = \frac{w_h \times l_h^2}{8} = \frac{9.53 \times 1.2^2}{8} = 1.715 \text{ kNm (less than 3 kN-m, hence safe).}$$

Check for shear

$$V = \frac{w_h \times l_h}{2} = \frac{9.53 \times 1.2}{2} \ 5.72 \text{ kN (less than 6.16 kN/m, hence safe).}$$

Check for deflection

$$\delta = \frac{5 \times w_h \times l_h^4}{384 \times E \times I} = \frac{5 \times 9.53 \times 1.2^4}{384 \times 145} = 0.00177 \text{ m}$$

$$= 1.77 \text{ mm (less than 3.33 mm, hence safe).}$$

Thus, the given spacing of the walers, and hence the span of the H-16 is safe in bending, shear, and deflection.

The span shall also be checked for bending moment, shear, and deflection for its adequacy for the bottom cantilever portion.

The span in cantilever portion: $l_c = 0.4$ m

$$M = \frac{w_h \times l_c^2}{2} = \frac{9.53 \times 0.4^2}{2} = 0.76 \text{ kNm (less than 3 kN-m, hence safe).}$$

$$V = w_h \times l_c = 9.53 \times 0.4 = 3.81 \text{ kN (less than 6.16 kN/m, hence safe).}$$

$$\delta = \frac{w_h \times l_c^4}{8 \times E \times I} = 0.00021 \text{ m} = 0.21 \text{ mm (less than } \frac{l_c}{360} = 1.11 \text{ mm, hence safe).}$$

Thus, the span of the cantilever portion is also adequate.

5.11.4 Check for Waler (As per IS: 800–2007)

Geometrical properties of waler *(ISMC 100 back to back with 50 mm gap)*:

Cross section area $A = 2{,}340 \text{ mm}^2$

Waler span $l_w = 1{,}200$ mm

Effective length $l_{eff} = 1{,}200$ mm

$I_{xx} = 37{,}34{,}000 \text{ mm}^4$

$Z_{xx} = 74{,}600 \text{ mm}^3$

$r_y = 40$ mm

$D = 100$ mm

$d_1 = 64$ mm

$T = 7.5$ mm

$t = 4.7$ mm

Maximum slenderness ratio $(l_w/r_y) = 30.0$

$D/T = 13.33$

$d_1/t = 13.62$

$T/t = 1.60$

Permissible stresses in bending

The permissible stresses in bending σ_{bc} and shear for the waler have been calculated based on IS: 800–2007 provisions.

Permissible bending stress is given by the following expression:

$$\sigma_{bc} = \frac{0.66 \times f_{cb} \times f_y}{\left[f_{cb}^n + f_y^n\right]^{1/n}}$$

Where, f_{cb} = elastic critical stress in bending, f_y = yield stress of the steel in MPa, and n is a factor which is taken as 1.4.

The expression for computing f_{cb} is given below:

$$f_{cb} = \frac{k_1(X + k_2 Y) \times c_2}{c_1}$$

Where, k_1 = a coefficient to allow for reduction in thickness or breadth of flanges between points of effective lateral restraint.

k_2 - a coefficient to allow for the inequality of flanges.

The values of k_1 and k_2 can be read from IS: 800–2007. In the present case, k_1 and k_2 can be taken as 1 and 0 respectively.

c_1, c_2 = respectively the lesser and greater distances from the section neutral axis to the extreme fibers. Consider c_2/c_1 to be 1.0

$$Y = \frac{26.5 \times 1,00,000}{(l/r_y)^2} = 2{,}944.44 \text{ N/mm}^2$$

$$X = Y \times \sqrt{1 + \frac{1}{20} \times \left(\frac{lT}{r_y D}\right)^2} = 3{,}296.10 \text{ N/mm}^2$$

Thus,

$$f_{cb} = \frac{k_1(X + k_2 Y) \times c_2}{c_1} = 1 \times (3{,}296.10 + 0 \times 2{,}944.44) \times 1 = 3{,}296.10$$

For, f_y = 250 N/mm^2, and the computed values of f_{cb}

$$\sigma_{bc} = \frac{0.66 \times f_{cb} \times f_y}{\left[f_{cb}^n + f_y^n\right]^{1/n}} = \frac{0.66 \times 3,296.10 \times 250}{\left[3,296.10^{1.4} + 250^{1.4}\right]^{1/1.4}} = 161.89 \text{ N/mm}^2$$

The permissible stresses, deflection, and EI value for the waler are summarized below:

Permissible bending stress σ_{bc} = 161.89 N/mm^2

Permissible shear stress = $0.4 \times f_y$ = 100 N/mm^2 (IS: 800–2007)

Permissible deflection = $\frac{l_w}{325} = \frac{1,200}{325}$ = 3.69 mm (IS: 800–2007)

$$EI = 7.84 \times 10^{11} \text{ N/mm}^2$$

The tie rods have been spaced at 1.2 m centre to centre, thus the span of the waler, $l_w = 1.2$ m. The span shall be checked for bending moment, shear, and deflection for its adequacy under simply supported condition.

$$\text{Load on waler} = w_w = w \times \text{spacing of walers} = 38.12 \times 1.2 = 45.74 \text{ kN/m}$$

Check for bending stress

$$M = \frac{w_w \times l_w^2}{8} = \frac{45.74 \times 1.2^2}{8} = 8.233 \text{ kNm}$$

Bending stress(M/Z) = 110.38 N/mm^2 (less than 161.89 N/mm^2, hence safe).

Check for shear stress

$$V = \frac{w_w \times l_w}{2} = \frac{45.74 \times 1.2}{2} = 27.44 \text{ kN}$$

Shear stress (V/A_{web}) = 29.20 N/mm^2 (less than 100 N/mm^2, hence safe).

Check for deflection

$$\delta = \frac{5 \times w_w \times l_w^4}{384 \times E \times I} = 1.6 \text{ mm (less than 3.69 mm, hence safe).}$$

Thus the given spacing of the tie rods, and hence the span of the walers is safe in bending, shear, and deflection.

5.11.5 Check for Tie Rod

Minimum yield = 360 N/mm^2

Cross sectional area of 16 mm tie rod = 200 mm^2

Yield force = 72,000 N = 72.00 kN

Load on one tie rod = $38.12 \times 1.2 \times 1.2$ = 54.89 kN (less than 72.00 kN, hence safe).

5.12 ILLUSTRATION OF ALL STEEL WALL FORM DESIGN

Design a wall formwork (all steel) for casting walls of 5.5 m height and 0.8 m thickness. The maximum lateral concrete pressure on the formwork can be assumed as 50 kN/m^2. Following materials are available: Mild steel plates of 4 mm thickness and mild steel flats of 50 mm × 6 mm and 60 mm × 6 mm are available.

Solution:

5.12.1 Design of Skin Plate

The proposed arrangement of the skin plate, horizontal and vertical stiffeners are shown in Figure 5.42. The skin plate is of 4 mm thickness while the horizontal stiffeners are of 50 mm width and 6 mm flat. The vertical stiffeners are made up of 60 mm × 6 mm flats. The stiffeners are arranged in a grid of 300 mm × 300 mm.

Lateral pressure on the side shutter P_{max} = 50 kN/m^2 (given).

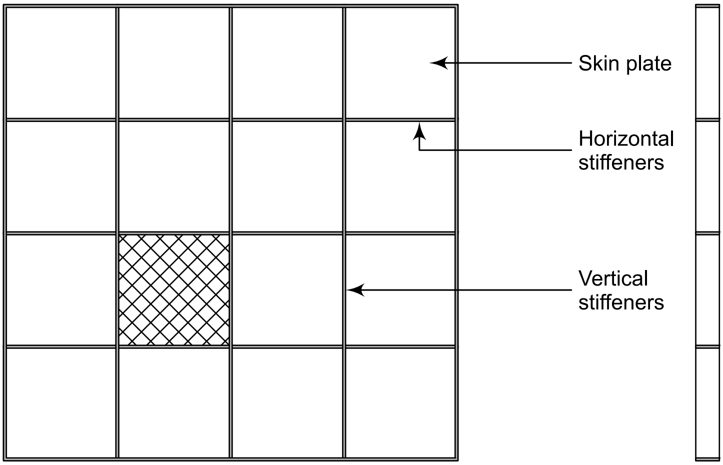


Figure 5.42 Front and Side Elevation of Proposed Wall Formwork (All Steel).

Considering a typical square element (shorter side a = longer side b = 300 mm) of the plate with all four edges inbuilt (see shaded portion in Fig. 5.42).

Thus, span ratio $r = b/a = 1$.

Maximum stress in bending tension $\sigma_{bt.cal} = \dfrac{b_1 \times P_{\max} \times a^2}{t_p^2}$

$$\sigma_{bt.cal} = 140.63 \text{ N/mm}^2 \text{ for } b_1 = 0.5,\ t_p = 4 \text{ mm}$$

Maximum permissible bending stress in tension $\sigma_{bt.all} = 165.0$ N/mm^2.

Bending moment at the centre of plate: $M_x = \alpha_x \times P_{\max} \times a^2$.

Taking $\alpha_x = 0.062$, $P_{\max} = 50$ kN/m^2, and $a = 0.3$ m,

$$M_x = 0.062 \times 50 \times 0.3^2 = 0.28 \text{ kNm/m}$$

Calculated tensile stress due to bending = $\dfrac{6M_x}{1{,}000 \times t_p^2} = \dfrac{6 \times 0.28 \times 10^6}{1{,}000 \times 4^2}$ = 105 N/mm^2 (less than 140.63 N/mm^2, hence safe).

Check for deflection

For computing maximum deflection at the centre of the plate, consider,

$$K = 0.02720 \text{ and } EI = 2{,}00{,}000 \times 4^3 = 1.28 \times 10^7 \text{ Nmm}^2$$

Thus,

$$\delta_{\max} = \frac{K \times P_{\max} \times a^4}{E \times I} = \frac{0.02720 \times 0.05 \times 300^4}{1.28 \times 10^7}$$

$$= 0.86 \text{ mm (less than span/325 = 0.92 mm, hence safe).}$$

5.12.2 Design of Horizontal Stiffeners

Load on one horizontal stiffener $w_h = P_{max} \times a = 50 \times 0.3 = 15 \dfrac{\text{kN}}{\text{m}} = 15$ N/mm.

Bending moment $BM_h = \dfrac{w_h \times l_h^2}{8} = 1{,}68{,}750$ Nmm: where $w_h = 15$ N/mm and $l_h = a = 300$ mm.

Section modulus required $Z_{rc} = \dfrac{BM_h}{\sigma_{bt.all}} = \dfrac{1{,}68{,}750}{165} = 1{,}023$ mm^3.

Section modulus provided $Z_p = \dfrac{t_f \times b_f^2}{6}$, where t_f = thickness of flat = 6 mm and b_f = width of flat = 50 mm.

Thus, $Z_p = \dfrac{6 \times 50^2}{6} = 2{,}500$ mm^3 (more than 1,023 mm^3, hence safe). For the calculation of Z_p, the flange plate has not been considered.

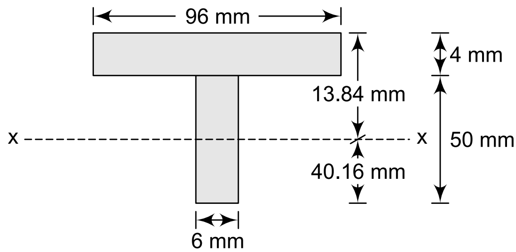


Figure 5.43 Cross Sectional Dimension of Horizontal Stiffeners (Not to Scale)

For the cross sectional dimensions shown in Fig 5.43, the centre of gravity CG and the distances of extreme bottom and top fibres y_b and y_t can be found out.

The CG is located at a distance of 40.16 mm from the bottom.

y_b and y_t are 40.16 mm and 13.84 mm respectively as marked in the figure.

Moment of Inertia

$$I_{XX} = \frac{6 \times 50^3}{12} + 6 \times 50 \times (40.16 - 25)^2 + \frac{4^3 \times 96}{12} + 4 \times 96 \times (13.84 - 2)^2$$
$$= 1{,}85{,}791 \text{ mm}^4$$

Section modulus $Z_b = \dfrac{I_{XX}}{y_b} = \dfrac{1{,}85{,}791}{40.16} = 4{,}626.27$ mm^3

Section modulus $Z_t = \dfrac{I_{XX}}{y_t} = \dfrac{1{,}85{,}791}{13.84} = 13{,}424.21$ mm^3

Calculated tensile stress due to bending $s_{bt,\, cal} = \dfrac{BM_h}{Z_b} = \dfrac{1{,}68{,}750}{4{,}626.27} = 36.48$ N/mm^2.

Calculated compressive stress due to bending $s_{bc,\, cal} = \dfrac{BM_h}{Z_t} = \dfrac{1{,}68{,}750}{13{,}424.21} = 12.57$ N/mm^2.

Shear force $V = \dfrac{w_h \times l_h}{2} = \dfrac{15 \times 0.3}{2} = 2.25 \text{ kN} = 2{,}250 \text{ N}$

Shear stress $\tau = \dfrac{V}{b_f \times t_f} = \dfrac{2{,}250}{50 \times 6} = 7.5 \text{ N/mm}^2$

$E \times I = \dfrac{2{,}00{,}000 \times 1{,}85{,}791}{1{,}000 \times 1{,}000^2} = 37.16 \text{ kNm}^2$

Deflection $\delta = \dfrac{5 \times w_h \times l_h^4}{384 \times E \times I} = \dfrac{5 \times 15 \times 0.3^4}{384 \times 37.16} = 0.00004 \text{ m} = 0.04 \text{ mm}$ (less than $\dfrac{\text{span}}{325} = \dfrac{300}{325} = 0.92$ mm, hence safe).

5.12.3 Design of Vertical Stiffeners

The span of vertical stiffeners l_v, is the spacing of the walers, which in this case is considered as 750 mm centre to centre. Simply supported end conditions are assumed for vertical stiffeners. The vertical stiffeners are to be fabricated using 60 mm × 6 mm flats.

Load on one vertical stiffener $w_v = P_{\max} \times a = 50 \times 0.3 = 15 \dfrac{\text{kN}}{\text{m}} = 15$ N/mm.

Bending moment $BM_v = \dfrac{w_v \times l_v^2}{8} = 10{,}54{,}688$ Nmm, where $w_v = 15$ N/mm and $l_v = 750$ mm.

Section modulus required $Z_{rc} = \dfrac{BM_v}{\sigma_{bt,all}} = \dfrac{10{,}54{,}688}{165} = 6{,}392 \text{ mm}^3$.

Section Modulus provided $Z_p = \dfrac{t_f \times b_f^2}{6} = 3{,}600 \text{ mm}^3$, where t_f = thickness of flat = 6 mm and b_f = width of flat = 60 mm.

For the calculation of Z_p, the flange plate has not been considered.

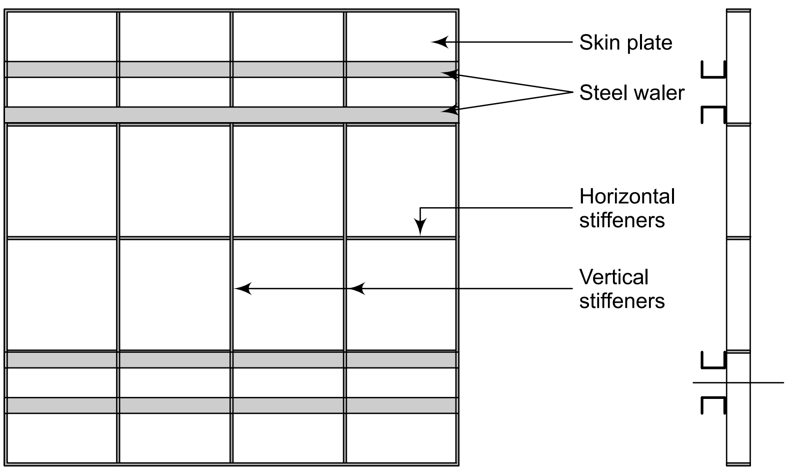


Figure 5.44 Front and Side Elevation of Proposed Wall Formwork Showing Waler and Tie Locations.

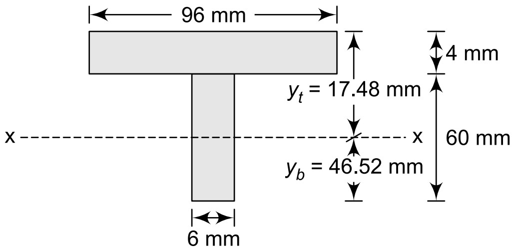


Figure 5.45 Cross Sectional Dimension of Vertical Stiffeners (Not to Scale).

For the cross sectional dimensions shown in Fig 5.45, the centre of gravity CG and the distances of extreme bottom and top fibres y_b and y_t can be found out.

The CG is located at a distance of 46.52 mm from the bottom.

y_b and y_t are 46.52 mm and 17.48 mm respectively as marked in the figure.

Moment of Inertia

$$I_{XX} = \frac{6 \times 60^3}{12} + 6 \times 60 \times (46.52 - 30)^2 + \frac{4^3 \times 96}{12} + 4 \times 96 \times (17.48 - 2)^2$$

$$= 2{,}98{,}778 \text{ mm}^4$$

$$\text{Section modulus } Z_b = \frac{I_{XX}}{y_b} = \frac{2{,}98{,}778}{46.52} = 6{,}422.57 \text{ mm}^3$$

$$\text{Section modulus } Z_t = \frac{I_{XX}}{y_t} = \frac{2{,}98{,}778}{17.48} = 17{,}092.56 \text{ mm}^3$$

$$\text{Calculated tensile stress due to bending } s_{bt,cal} = \frac{BM_v}{Z_b} = \frac{10{,}54{,}688}{6{,}422.57} = 164.22 \text{ N/mm}^2.$$

$$\text{Calculated compressive stress due to bending } s_{bc,cal} = \frac{BM_v}{Z_t} = \frac{10{,}54{,}688}{17{,}092.56} = 61.70 \text{ N/mm}^2.$$

$$\text{Shear force } V = \frac{w_v \times l_v}{2} = \frac{15 \times 0.75}{2} = 5.625 \text{ kN} = 5{,}625 \text{ N}$$

$$\text{Shear stress } \tau = \frac{V}{b_f \times t_f} = \frac{5{,}625}{60 \times 6} = 15.63 \text{ N/mm}^2$$

$$E \times I = \frac{2{,}00{,}000 \times 2{,}98{,}778}{1{,}000 \times 1{,}000^2} = 59.76 \text{ kNm}^2$$

5.12.4 Check for Waler

Two numbers ISMC 100 back to back have been provided as walers in the wall formwork. Let's take the spacing of the walers as 0.75 m.

Thus, load on one waler = $w_w = w \times$ sapacing of the walers = $50 \times 0.75 = 37.5$ kN/m

Check for bending stress

The span of the waler l_w, is taken as the spacing of the tie rod which is assumed as 1.2 m centre to centre.

$M = \frac{w_w \times l_w^2}{8} = \frac{37.5 \times 1.2^2}{8}$ = 6.75 kNm (less than the allowable bending moment of 10.2 kNm, hence safe).

Check for shear stress

$$V = \frac{w_w \times l_w}{2} = \frac{37.5 \times 1.2}{2} = 22.5 \text{ kN (less than 103 kN, hence safe).}$$

Check for deflection

$$EI = 784.00 \text{ kNm}^2$$

$$\delta = \frac{5 \times w_w \times l_w^4}{384 \times E \times I} = \frac{5 \times 37.5 \times 1.2^4}{384 \times 784} = 0.00129 \text{ m}$$

$$= 1.29 \text{ mm (less than } \frac{\text{span}}{325} = 3.69 \text{ mm, hence safe).}$$

Thus, the assumed spacing of the tie rods, and hence the span of the walers is safe in bending, shear, and deflection.

5.12.5 Check for Tie Rod

Tie rods of 18mm diameter HT bar are proposed.

Maximum pull in tie rod $P_t = 2 \times V = 45$ kN

Allowable pull in tie rod $P_{ta} = 50$ kN, O.K.

REVIEW QUESTIONS

Q1. True or False

(a) Various types of proprietary wall formworks are: climbing scaffold, traveling climbing scaffold, and automatic climbing scaffold.

(b) Various types of PERI climbing formwork are :-CB240,CB160, RCS,ACS.

(c) The three variants of ACS are: ACS-R, ACS-P, and ACS-G.

(d) Various types of Doka climbing formwork are: CB-150A, CB-150F, MF-240,and SKE-50.

Q2. Match the following

(i) Large area wall form	(a) Used for high rise structures: jump form.
(ii) Climbing formwork	(b) Gang form, prefabricated panels joined together to form a large shutter panel.

Q3. Match the following

(i) Maximo panel wall formwork	(a) Used for semi circular arch, segmental arch.
(ii) Trio panel wall formwork	(b) Used for structures having curvature and circular structures.
(iii) Handset panel wall formwork	(c) Continuously adjustable elements, wide range of applicability.
(iv) Vario GT 24 girder wall formwork	(d) Light, withstand concrete pressure up to 40 kN/m^2.

(v) Rundflex circular wall formwork
(vi) GRV circular wall formwork

(e) Trio housing, Aluminum, 330 panel.
(f) Used for speedy construction, conical tie system.

Q4. Sequence the following steps in wall formwork design; (a) design of sheathing ; (b) design of wales; (c) determine the pressure on form ; (d) check for bearing ; (e) design of tie ; (f) design of stud.

Q5. List out the salient features of each of the following:
(i) Climbing formwork;
(ii) Traveling formwork;
(iii) Automatic climbing scaffold.

Q6. List out the various design inputs and steps for wall formwork design.

Q7. List out the salient features of L&T wall formwork.

Q8. List out the various features of PERI wall formwork.

Q9. List out the salient features of conventional wall formwork.

Q10. Design the formwork for a 3.0 m high wall, where ordinary concrete is to be placed at 10°C progressively over a 1 hour period. The following inputs may be taken for the design:

C1	Shape co-efficient	1.0
C2	Material co-efficient	0.3
D	Concrete density	26 kN/m^3
H	Vertical height of form	3.0 m

Q11. Design the forms for a concrete wall for the following data.
(a) Height of wall = 4.0 m; (b) Rate of filling forms = 1.33 m per hour; (c) Temperature = 25°C.
It is proposed to limit the deflection to l/270. Use timber sheathing and studs.

Q12. Thickness of plywood is 20 mm, concrete wall = 150 mm, and allowable stress in the deck = 12 MPa, E = 10GPa; it is decided to use 75 mm × 150 mm batten as stud. What is the spacing of the joist and what shall be the spacing of the wales, height of the wall is 4.5 m. Maximum permissible stress on the timber is 8 MPa. Tie and braces shall be designed suitably.

Chapter

6 Column Formwork

Contents: Introduction; Conventional Column Formwork; Proprietary Column Formwork; L&T Column Formwork; Doka Column Formwork System; PERI Column Formwork; Disposable Column Formwork; All Metal Column Formwork; Achieving Formwork Economy in Column Construction; Design for Column Form; Illustration of Column Formwork Design; Example for Computation of Force in Diagonal Tie Rod of Column

6.1 INTRODUCTION

In this chapter, we discuss the conventional column formwork and some proprietary column formwork. In conventional column formwork, the emphasis is on all timber and all steel formwork. Under proprietary column formwork, we discuss L&T, Doka column formwork, and PERI formwork, and some disposable column formworks. The design issues of both the types of columns are discussed. Some field applications of column formworks are also illustrated.

6.2 CONVENTIONAL COLUMN FORMWORK

The main components of a column formwork are sheathings, studs, yokes, a tie arrangement, and alignment arrangement. The sheathing materials typically would consist of timber planks or plywood sheets, or steel plates. The studs would be typically of timber while yokes could be of timber or structural steel sections. The studs are kept at closer interval compared to wall formwork since the pressure exerted in case of column formwork is usually higher than that in case of wall formwork. This is because a column requires small quantity of concrete and thus, higher rate of rise is usually the case which results in higher concrete pressure on formwork. In handset wood formworks, studs are usually pieces of timber 100 mm × 100 mm. A typical timber yoke would consist of two timber members of cross section 100 mm × 150 mm, while the typical steel yoke would have two channel sections (ISMC 100) in back to back position. The yokes help maintain the sides of column formwork in position and also prevent the buckling of sheathing due to the pressure exerted by the concrete. The yokes at the bottom may be closely spaced when compared to the yokes at the top of the column form because the pressure is high at the bottom and less at the top.

There may be chamfer strips at the corners of the columns if the drawing shows the chamfer. When the size of the column is small, the adjustable column clamps have been provided to take care of lateral pressure exerted on the columns (see Fig. 6.1). However, for large column cross section, this arrangement may not work and one has to think of providing the through ties at suitable spacing.

A column formwork typically consists of four sides even though different shapes of column formwork are possible. Some commonly used column shapes are given from Figs. 6.2–6.4.

Inclined struts or shores are used in tall column formwork in order to ensure the stability of the formwork, and also verticality. For columns, shores on all sides are preferred. Shores in conventional column form arrangement are usually of timber; however, proprietary metal shores are also very common these days. The shores are connected to the bedplate to distribute the pressure uniformly on the soil.

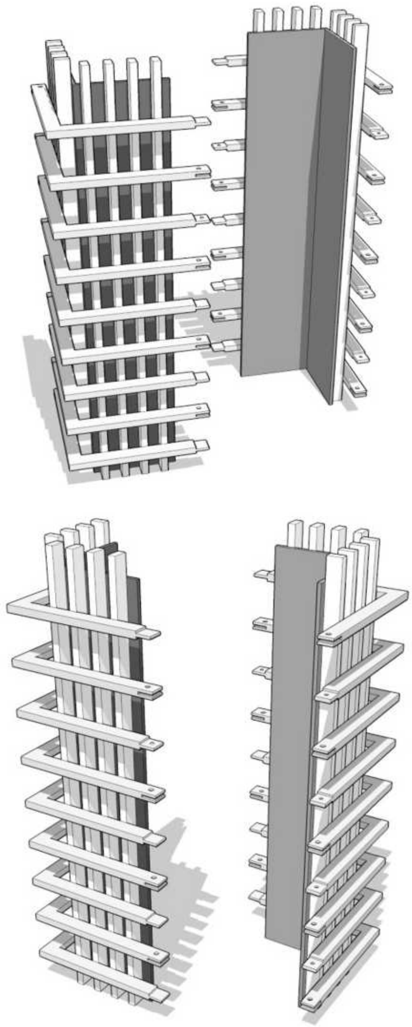


Figure 6.1 Typical Construction of Heavier Column.

In all steel column formwork, sheathing would consist of steel plates of thickness usually varying from 3.15 to 5 mm and the studs (supporting members for sheathing), could be made with flats (50 mm × 6 mm is a very common section) or angle sections (ISA 50 mm × 50 mm × 6 mm is a very common section). The yokes would be made out of two numbers ISMC 100 back to back. The size of a shutter panel is chosen in such a manner that it is easy to lift them manually. Individual side shutters are brought at column location and they are connected together with the help of bolts. The joints formed when adding the two shutters need to be made water tight so that the slurry does not leak. For this, foam or similar substance is glued at the joint location.

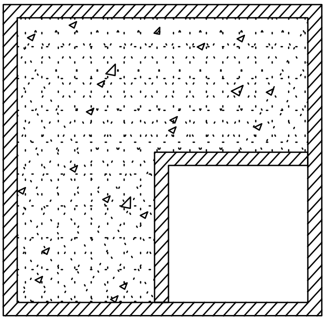


Figure 6.2 L-shape Column.

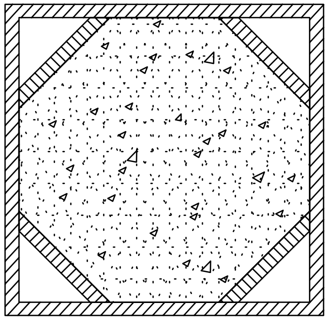


Figure 6.3 Octagonal Column.

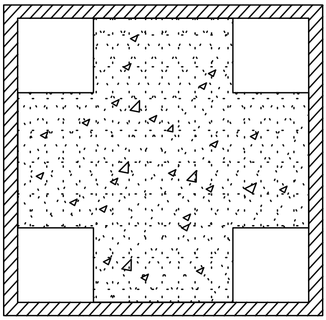


Figure 6.4 Plus Shape Column.

6.3 PROPRIETARY COLUMN FORMWORK

A large number of proprietary column formwork systems are available these days from different manufacturers. The column formwork solution offered by these manufacturers can suit varied column form needs. Most of the manufacturers have solutions for varied column cross-section, usually starting from 150 mm × 150 mm and going even beyond 1,000 mm × 1,000 mm. The solutions are available for varied shapes such as rectangular, square, and circular columns. The solutions are available even on the desired requirement of the kind of concrete finish. For example, some manufacturers have special formwork solutions for fair-faced concrete.

For most proprietary column formworks, the components used are the same as that used for wall formwork. An additional component specific to column formwork is a set of column clamps or yokes which encircle the column forms and hold them securely to withstand the concrete pressure. The clamps are generally adjustable and can suit various column dimensions. Some manufactures also offer special shaped form panels to fit in the corners or splays or construction of curved structures. In some cases, the column formwork components with some additional accessories, can be converted and used as a climbing formwork system for a column.

The manufacturers specify the permissible concrete pressure on the formwork. The proprietary formworks have flexibility to be used both for manual applications and crane assisted applications. Specially designed lifting hooks are in-built in the system for ease in lifting the form panels by cranes. Most of the proprietary column formworks have the provision for fixing an access ladder for climbing up and down besides the provision to fix the working platform at the top of the column form. The proprietary column formwork can be reused large number of times.

Some commonly visible proprietary column formworks are briefly explained in the following sections.

6.4 L&T COLUMN FORMWORK

L&T column formwork system is suitable for casting columns of minimum 150 mm × 150 mm. The system uses plywood sheathing which is supported by H-16 beams which in turn are supported by steel walers. With the mentioned components, it is possible to form column boxes of widely varying dimensions by suitable rearrangement.

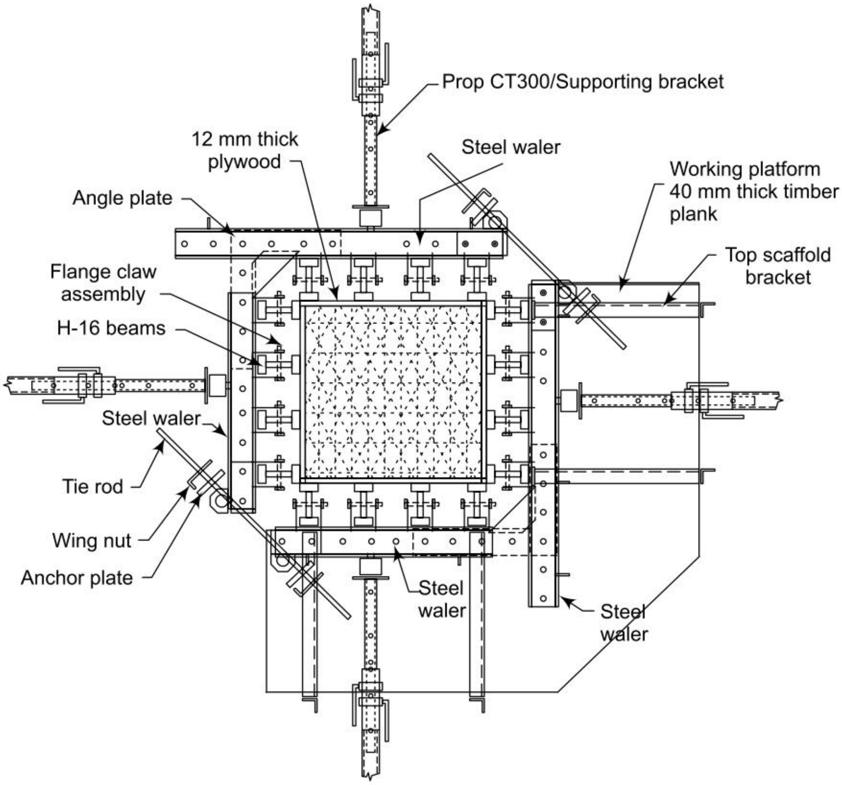
The system facilitates fixing working platforms for access, checking of reinforcement, concreting, etc. The high strength tie system in the L&T column form enables it to sustain even high concrete pressure. The high strength tie system can either be through tie system, or lost anchor system depending on the structure. The H-16 beams can be butt and splice jointed to form larger size of panels. The H-16 beams and walers are available in various sizes for making panels depending on the requirement of column formwork. The entire panel, along with the platform and alignment system can be moved using cranes as a single unit.

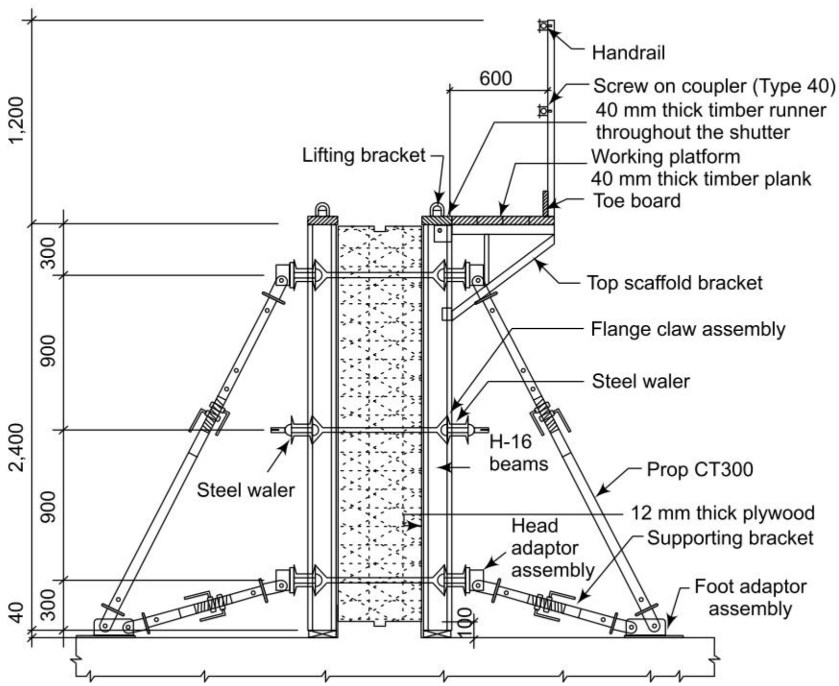
The column form panels are usually fabricated in workshops at the site which reduces assembly at the work location. This also ensures accuracy besides increasing the productivity. Since the entire unit is handled as a whole, loss of smaller components is reduced, and contributes to material productivity. In the absence of a crane the panels can be dismantled and handled manually. The column formwork panels can be used along with climbing formwork system.

In Fig. 6.5, a typical column formwork for two sizes is shown. The components are clearly shown in the figure.

For the construction of factory sheds (see Fig. 6.6) at Tirupur, Tamilnadu, the contractor used L&T column formwork. The application of a heavy duty tower for erecting the working platform can also be seen in the figure.

For the construction of RC columns of the TG deck building for the combined power plant at Kuthalam, the contractor used L&T column formwork system. The six RC columns of varying sizes (1.2 m × 1.5 m, 1.0 m × 1.70 m, and 1.0 m × 1.2 m) were supporting a heavy slab. The forms for columns were fabricated in L-shape of 9.6 m height. Figure 6.7 shows the lifting of an L-shape form. These forms were transported and erected with the help of a mobile crane. Figure 6.8 shows the application of a crane in erecting the column form. Window openings were provided on all four sides of the forms for pouring and compacting of concrete.


Prop CT300/Supporting bracket
12 mm thick plywood
Steel waler
Angle plate
Working platform
40 mm thick timber plank
Top scaffold bracket
Flange claw assembly
H-16 beams
Steel waler
Tie rod
Wing nut
Anchor plate
Steel waler
Steel waler


Handrail
600
Screw on coupler (Type 40)
40 mm thick timber runner throughout the shutter
Lifting bracket
Working platform
40 mm thick timber plank
Toe board
Top scaffold bracket
Flange claw assembly
Steel waler
H-16 beams
Steel waler
Prop CT300
12 mm thick plywood
Supporting bracket
Head adaptor assembly
Foot adaptor assembly
1,200
2,400
300
900
900
300
40
100

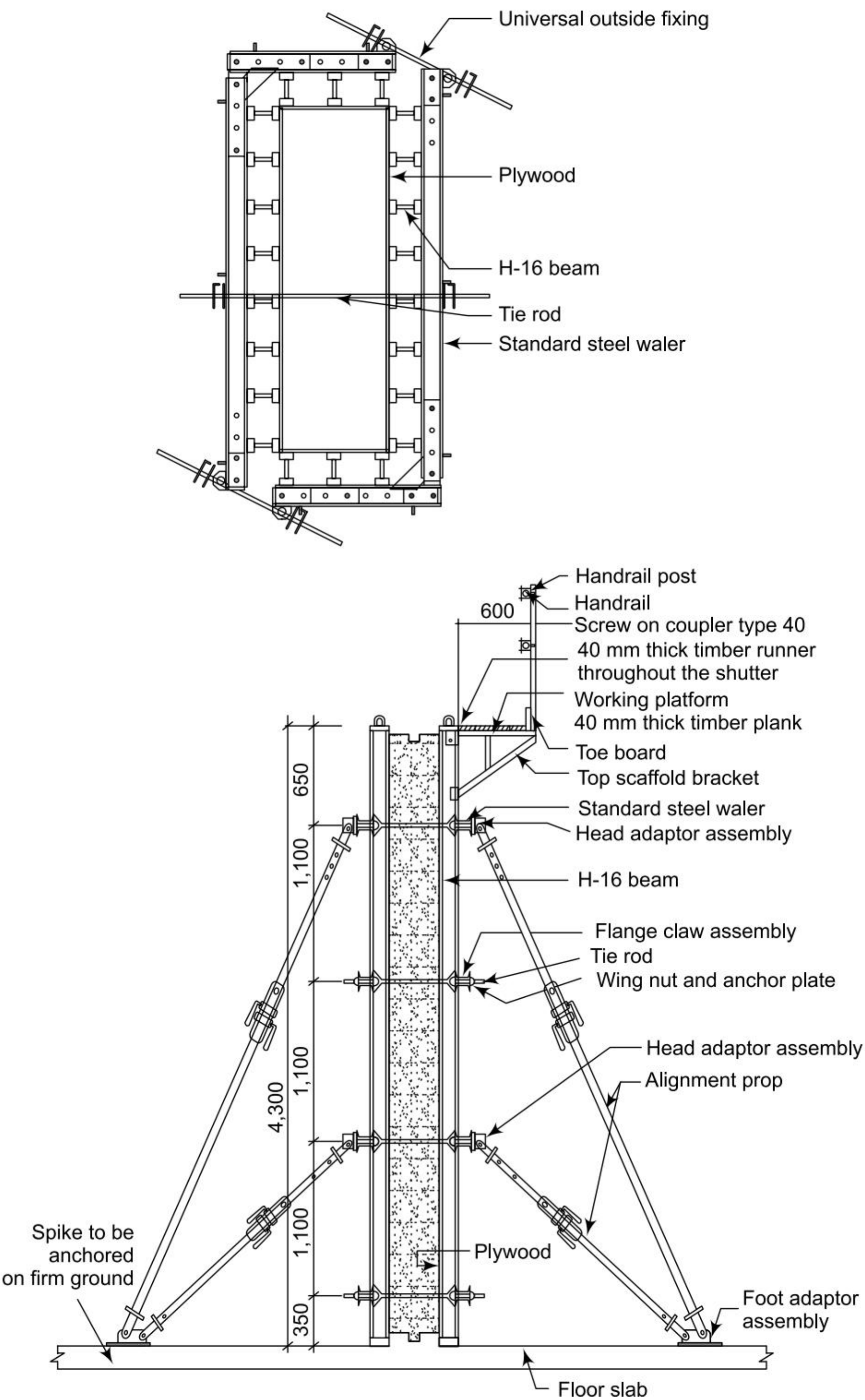


Figure 6.5 L&T Column Formwork System.

Figure 6.6 Application of L&T Column Formwork for the Construction of Factory.

Figure 6.7 Column Formwork Being Lifted With the Help of a Crane.

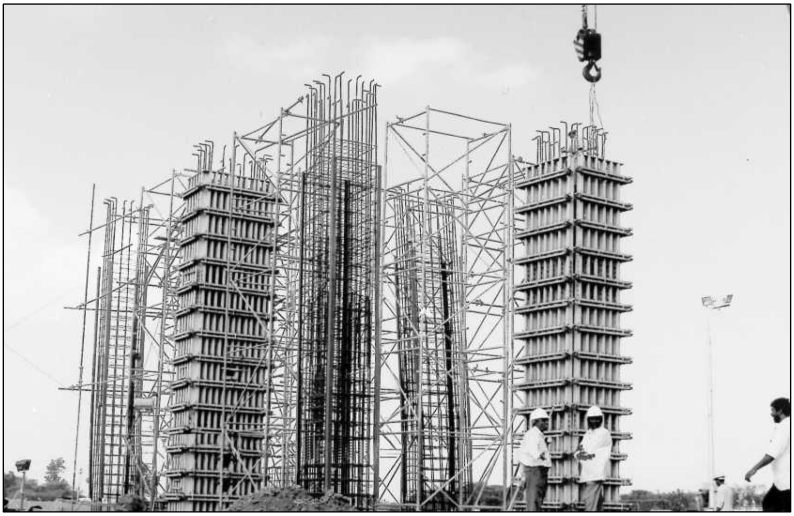


Figure 6.8 Column Formwork in Position (Height 9.6m).

Sometimes, a combination of conventional formwork and some components of a system formwork can also be used effectively for column forming. For example, a combination of conventional steel form panels and components of L&T column formwork consisting of walers and tie assembly was used to cast Y-shaped columns for the construction of National Games Stadium, Guwahati. The collapsible tube props resting on heavy duty towers were also used in Y-shaped column formwork. The heavy duty towers were erected on the ground as shown in Fig. 6.9. The Y-shaped columns are 16.5 m high from the finished ground level. Each column comprises of shuttering area of about

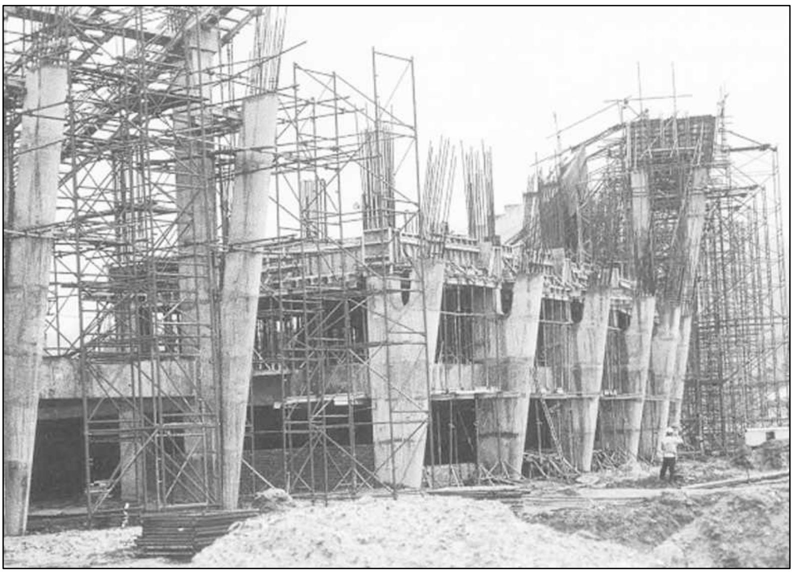


Figure 6.9 Y-shaped Columns Under Construction.

100 m^2, concrete quantity of 27 m^3. The size of the column at the pile cap is 600 mm wide × 1,100 mm long, with outside face semi circular at 300 mm radius. The angle of inclination of the inclined part is 80°. Pile cap top is at –1.15 and the inclination starts from –0.3. The column splits into two parts as shown in Fig. 6.9 at +5.455, with vertical part of dimension 600 mm × 800 mm and the inclined part of 600 mm × 808 mm, with outside face semi circular at 300 mm radius. From +10.45 to +16.5, the vertical and the inclined parts flare in width from 600 mm to 1600 mm, resulting in top plan area of dimensions 4,210 mm long × 1,600 mm wide.

The sheathing material adopted for the column is steel, for the purpose of huge concrete volumes in each lift and to maintain the semicircular shape, quality, and for more number of repetitions. The form heights were restricted to 2.5 m for ease in manual handling. The formwork for the full height of columns was done in stages. The completed Y-shaped columns are shown in Fig. 6.10.

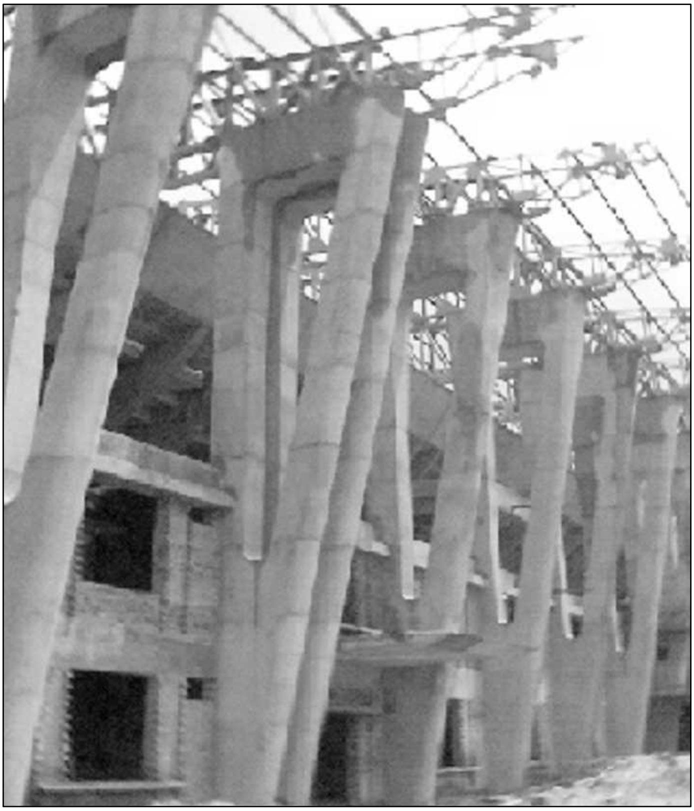


Figure 6.10 Completed Y-shaped Columns.

6.5 DOKA COLUMN FORMWORK SYSTEM

Doka offers a number of solutions for column formwork, such as (1) Column formwork KS Xlife, (2) Column formwork Top 50, (3) Column formwork RS, (4) Column formwork Framax Xlife, and (5) Column formwork Frami Xlife. Brief descriptions of each of the above systems are given below:

6.5.1 Column Formwork KS Xlife

The KS Xlife column formwork system offers a quick way to form quadratic and rectangular columns. These are suitable for projects having large number of columns with varying cross-sections. The change in cross section is possible without changing the form facing, anywhere between 200 and 600 mm, in an increment of 50 mm. The system has a folding mechanism which minimizes set-up and removal times and thus, allows rapid forming. The system uses plastic coated, multi-layer panels as sheathing member, which can be repeated large number of times and requires less time for cleaning. The plastic layer on both sides enables easy nailing and prevents flaking which is one of the main reasons for early damages and ruin of panels. Up to a column formwork height of 3,600 mm, the formwork system can be shifted by hand from one location to the other using wheels. For heights in excess of 3,600 mm, crane assistance is required to shift the form. The system has provision for an access ladder, and a working platform for safe working at all heights. Figure 6.11 shows a typical column formwork KS Xlife.

Figure 6.11 A Typical Column Formwork KS Xlife (Courtesy Doka).

6.5.2 Column Formwork Top 50

The column formwork Top 50 system is suitable for complicated column cross-sections, large column heights, many formwork re-use cycles, and tough specifications regarding the concrete finish. The system offers an economical way of forming columns of any shape and size. Column formwork Top 50 has provisions for vertical access and a working platform for safe working. A typical column formwork Top 50 is shown in Fig. 6.12.

Figure 6.12 A Typical Column Formwork Top 50 (Courtesy Doka).

6.5.3 Column Formwork RS

The column formwork RS system is suitable for circular columns. The sheathing component is made up of steel. The factory made components can be assembled faster and also ensure good concrete finish. The system is provided with lifting hooks (crane hoisting points). Thus, the formwork can be transported easily from one location to the other. The formwork panels are available in heights of 250, 500, 1,000, and 3,000 mm, and in diameters of 300 mm–600 mm. The formwork system has provisions for vertical access and a working platform for safe working. Figure 6.13 shows the various heights of available form panels, besides an erected column formwork RS system.

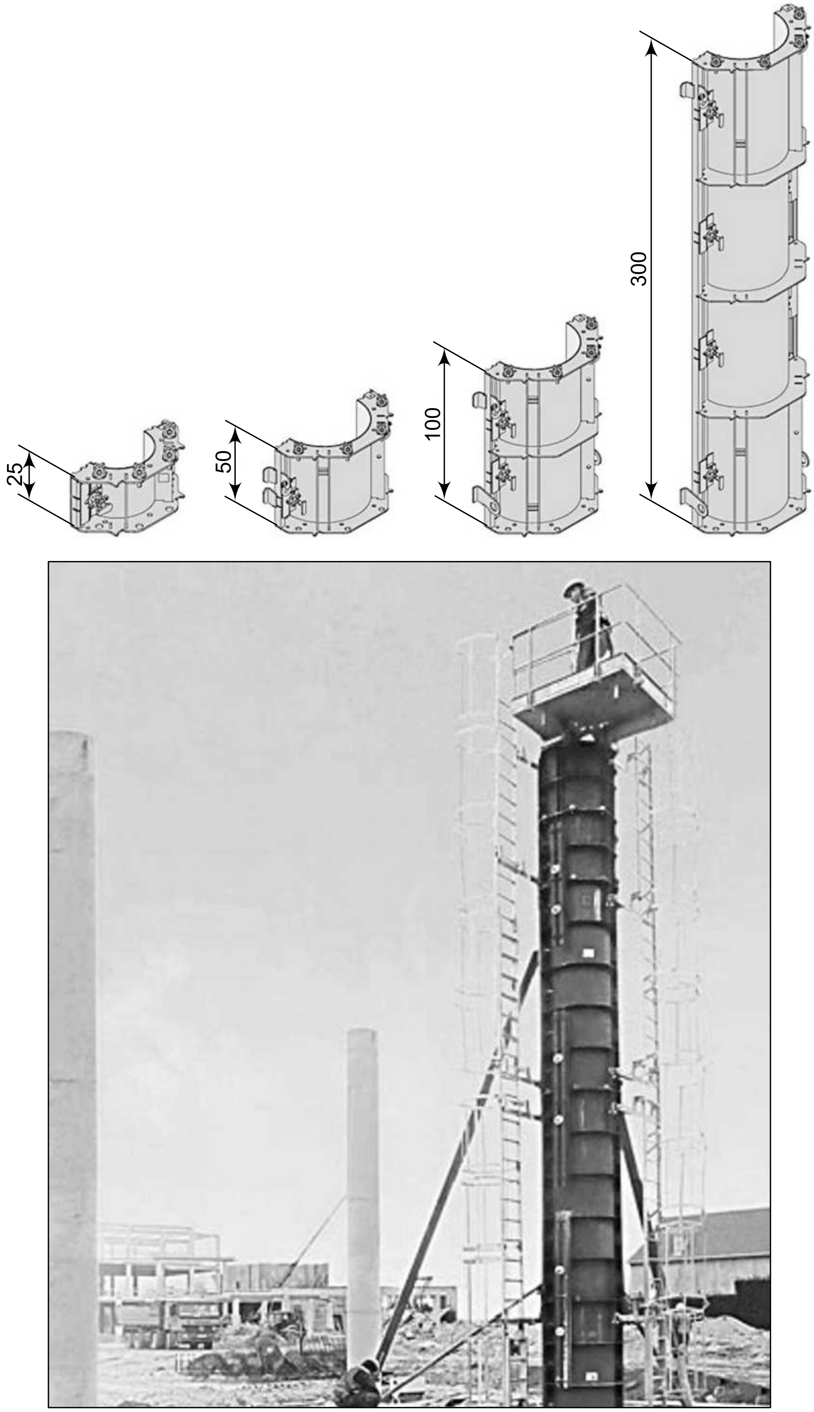


Figure 6.13 Column Formwork RS (Courtesy Doka).

6.5.4 Column Formwork Framax Xlife

The Framax Xlife column formwork system uses universal panels to form columns of up to 1,050 mm × 1,050 mm, in an increment of 50 mm. The universal panels are available in widths of 900 mm and 1,200 mm and in panel heights of 900 mm, 1,350 mm, 2,700 mm, and 3,300 mm. These panels can be used for forming columns, corners, stop-ends and wall junctions, and wall elements. Form ties through the concrete are not needed in this system, making it faster to assemble and dismantle. The system has provisions for access and a working platform. Figure 6.14 shows a typical column formwork Framax Xlife.

Figure 6.14 A Typical Column Formwork Framax Xlife (Courtesy Doka).

6.5.5 Column Formwork Frami Xlife

The system uses universal panels for forming varied cross-sections of columns up to 650 mm × 650 mm, in an increment of 50 mm. The forms can be quickly erected and dismantled. The Frami Xlife universal panels can be used for forming columns, corners, stop-ends, wall junctions, and wall elements. The panels can withstand a concrete pressure of 80 kN/m². The system can be equipped with access routes for climbing up and down and a working platform. Figure 6.15 shows a typical column formwork Frami Xlife.

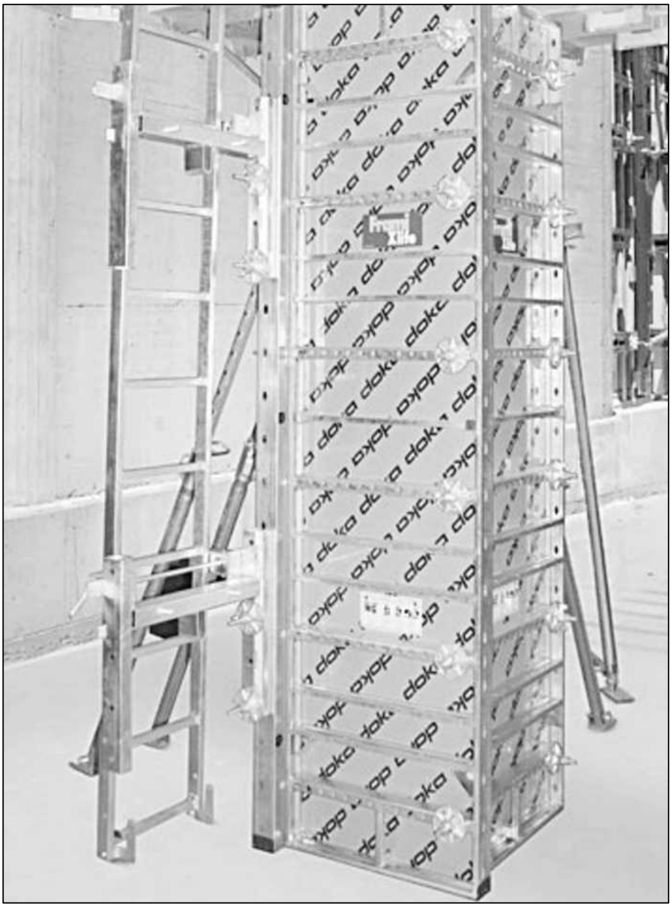


Figure 6.15 A Typical Column Formwork Frami Xlife (Courtesy Doka).

6.6 PERI COLUMN FORMWORK

PERI offers a number of column formwork solutions such as (1) Rapid column formwork, (2) Vario GT 24 column formwork, (3) Trio column formwork, (4) Quattro column formwork, (5) SRS steel circular column form, (6) LICO lightweight column formwork, and (7) Vario Quattro column formwork. The features of each of these column formworks are given in the following sections:

6.6.1 Rapid Column Formwork

PERI has introduced Rapid column formwork (see Fig. 6.16), in which the plywood is clamped (instead of nailed or screwed) to the lightweight aluminum frame. This results in a high quality concrete surface without any indentations resulting from either screws or nails. The system can be used for forming rectangular and square columns having cross sections up to 600 mm × 600 mm. Sharp edged columns are also possible to be formed using this system. It has frames available in three heights: 3,000 mm, 2,100 mm, and 600 mm. This allows various height adjustments besides allowing faster construction. The system being light weight does not require crane involvement. It can resist concrete pressure up to 120 kN/m^2. The provision of access ladders and a concreting platform allows safe working at all heights.

Figure 6.16 Rapid Column Formwork (Courtesy PERI). Alignment Props Not Shown for Clarity.

6.6.2 Vario GT 24 Column Formwork

The basic components used in Vario GT 24 column formwork (see Fig. 6.17), are plywood, GT 24 wooden beams, waler, wedges, and tie yokes. The column is formed in two halves (L-shaped), which are tied together using wedges and tie yokes. The system can be used for forming rectangular and square cross-sections up to 800 mm × 1,200 mm. Internal ties are not required and two numbers external ties at each waler location are sufficient to withstand concrete pressure up to 100 kN/m^2. The system can be designed for higher concrete pressure as well. The provision of access ladders and a concreting platform allows safe working at all heights.

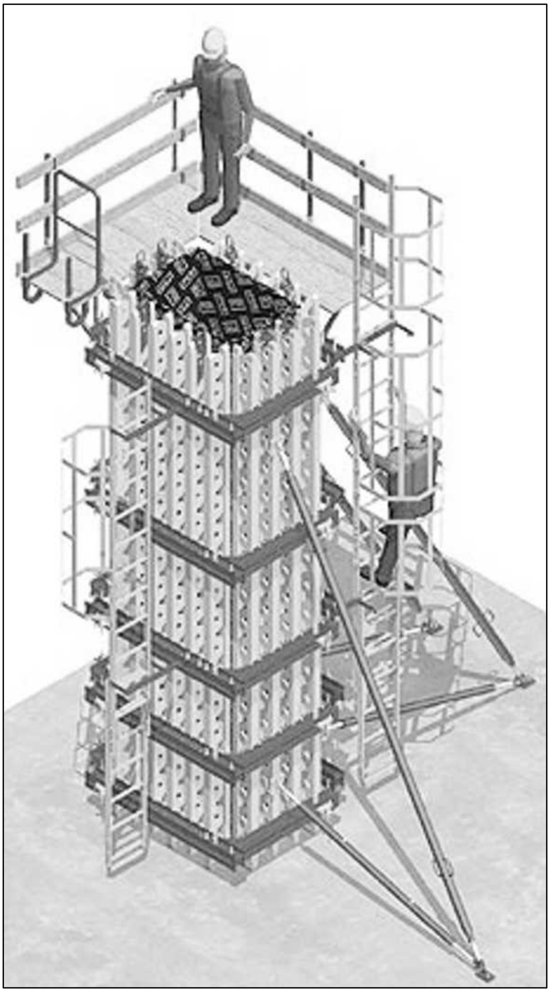


Figure 6.17 Vario GT 24 Column Formwork (Courtesy PERI).

6.6.3 Trio Column Formwork

Trio column formwork system can be used for column cross sections up to 750 mm × 750 mm, in an increment of 50 mm. It is possible to form both, a rectangular as well as a square section with sharp edges or with chamfers. The system has provisions of a concreting platform and access ladders for safe working. The system is designed to withstand concrete pressure up to 100 kN/m^2. The view of a typical Trio column formwork is shown in Fig. 6.18. The arrangement of formwork in plan is shown in Fig. 6.19.

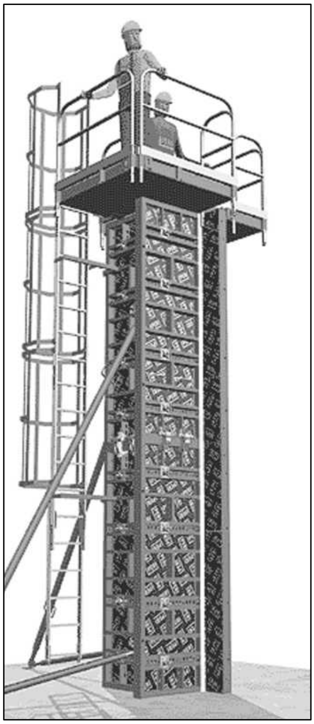


Figure 6.18 Typical Trio Column Formwork (Courtesy PERI).

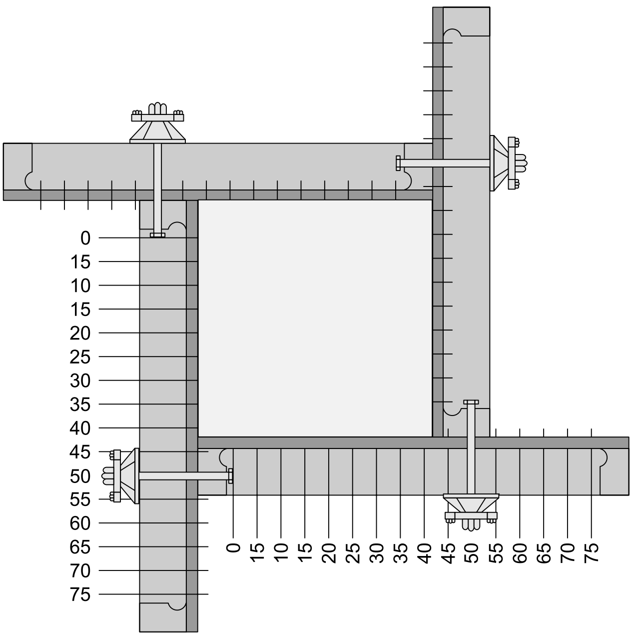


Figure 6.19 Plan of Trio Column Formwork (Courtesy PERI).

6.6.4 Quattro Column Formwork

The system can be used for forming rectangular and square columns having cross sections up to 600 mm × 600 mm, in an increment of 50 mm. It has frames available in four heights: 3,500 mm, 2,750 mm, 1,250 mm, and 500 mm. This allows various height adjustments besides allowing faster construction. It can resist concrete pressure up to 80 kN/m^2. All four sides of the column formwork can be moved together with the help of a crane or manually through transportation wheels. The system has foldable column frames and thus requires less space for stacking. Figure 6.20 shows a typical Quattro column formwork.

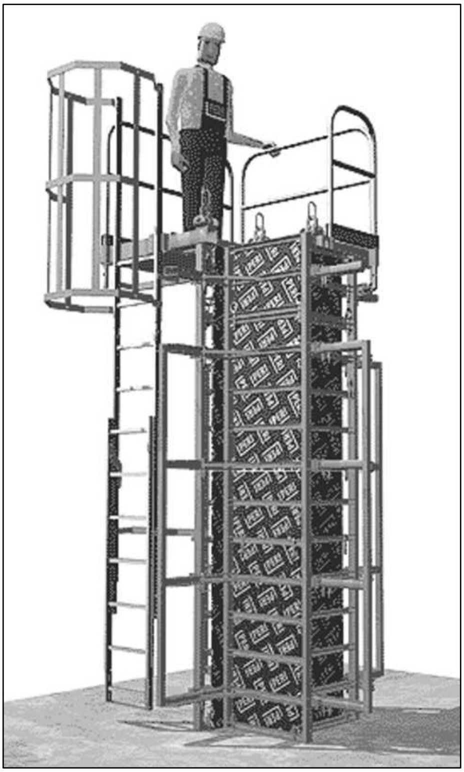


Figure 6.20 A typical Quattro Column Formwork (Courtesy PERI).

6.6.5 SRS Steel Circular Column Form

For circular column, PERI provides SRS steel form. The system can form column formwork up to 700 mm diameter, starting from 250 mm, in an increment of 50 mm. The system can resist concrete pressure up to 150 kN/m^2. The column form is supplied in two halves, with a bolt arrangement to join the two halves. The column shutters are available in four heights: 3,000 mm, 2,400 mm, 1,200 mm and 300 mm, which allow shuttering in 300 mm increments. The quality of concrete surface obtained is extremely good. The provision of access ladders and a concreting platform allows safe working at all heights. The view of a typical SRS steel column formwork is shown in Fig. 6.21.

Figure 6.21 A Typical SRS Steel Circular Column Form (Courtesy PERI).

6.6.6 LICO Lightweight Column Formwork

For columns of small cross sections starting from 200 mm × 200 mm to 600 mm × 600 mm, PERI offers LICO lightweight column formwork. The system is suitable where the forms are to be set without crane assistance. The connecting parts of the formwork are permanently attached to the form panels and thus, there are fewer chances of them getting lost. The formwork panels are available in three heights of 3,000 mm, 1,000 mm and 500 mm. The system can withstand a concrete pressure of 80 kN/m^2. Figure 6.22 shows a typical LICO lightweight column formwork.

6.6.7 Vario Quattro Column Formwork

The Vario Quattro column formwork system is suitable for large column cross sections. The column formwork assembly consists of four column sides, the push-pull props, a concreting platform and an access ladder. The entire assembly can be lifted with the help of a crane. The rectangular and square column cross sections starting from 200 mm × 200 mm to 1,200 mm × 1,200 mm, can be formed in an increment of 50 mm, both for sharp edged and chamfered columns. The formwork system can withstand a concrete pressure of 100 kN/m^2. The provision of a concreting platform and access ladders makes the system safer to work at all heights. A typical vario Quattro column formwork is shown in Fig. 6.23.

Figure 6.22 LICO Lightweight Column Formwork (Courtesy PERI).

Figure 6.23 Application of Vario Quattro Column Formwork (Courtesy PERI).

6.7 DISPOSABLE COLUMN FORMWORK

6.7.1 Column Formers

This is a paper-based disposable formwork system and is being manufactured by SDG, and is claimed to be a cost effective solution to problems of forming round columns (see Fig. 6.24). It gives great versatility to the architectural design and also provides the contractors an easy means of achieving the desired quality standard. The erection and stripping time are extremely less. Some of the advantages offered by this form are given below:

- The ideal fair face finish concrete column;
- Fast and cost effective;
- Strong yet light and easy to handle;
- One-use disposable design;
- Supplied to site ready prepared to length;
- Base and top are easily secured;
- No mould oil or release agent required;
- Stripped in seconds;
- Protects finished column from site damage;
- Available in a wide range of sizes;
- It can be made available within short span of time;
- Suitable for horizontal void cutting.

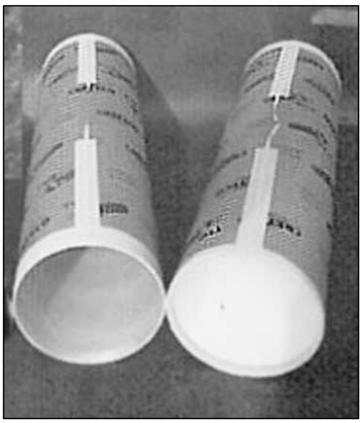


Figure 6.24 Column Formers.

6.7.2 Rapidobat Formwork Columns

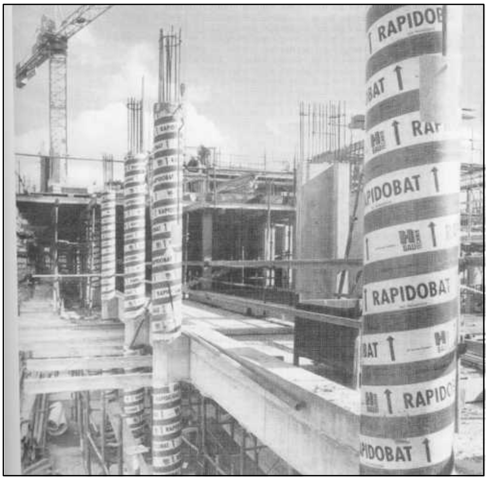
This is another disposable formwork for rapid construction of circular columns (see Fig. 6.25). The system has the following features.

- Provides the optimum fair face finish concrete column;
- Fast and economical to use;

- Very strong;
- Light and easy to handle;
- Quick and easy to erect;
- Requires no mould oil or release agent;
- Stripped literally in seconds;
- Delivered at a short notice;
- Manufactured in a large range of sizes;
- Can be used for horizontal void forming.

Figure 6.25 Rapidobat Column Formwork.

Rapidobat is a disposable formwork made up of spirally wound rigid paper, suitable for forming round column. The internal side of the formwork is lined with a smooth faced plastic release sheet. The release sheet helps attaining a good quality surface finish. The formwork system is claimed to be fast and an economical solution for forming round columns, compared to timber and steel column formwork. The number of columns cast in a day is limited only by the number of column formworks purchased. Figure 6.26(a) shows the Rapidobat form in position, while Fig. 6.26(b) shows the finish of the column after the form has been removed.



(a)

(b)

Figure 6.26 (a) Rapidobat Column in Installed Position (b) The Column After Removal of Forms.

6.8 ALL METAL COLUMN FORMWORK

Custom-made all metal column formworks are also commonly used in the industry. This is adopted where it is economical to form a large number of rectangular, square, or circular columns, and the contractor does not have proprietary formwork available for column forming.

The number of all metal column form sets is decided depending on the number of columns to be cast, available duration, and the cycle time. Steel is commonly used as the material for all metal column formwork. The sheathing consists of steel sheets in thicknesses varying from 3.15 mm to 5.0 mm. The supporting members are either flat or angle sections.

The column is formed by joining form panels of varying heights. The height of each panel is decided based on the method statement. For example, if the forms are to be assembled without crane assistance, the weight should be such that it can be easily handled by crew members. The formwork specification on joints would govern the number of pieces in which one form panel is to be manufactured.

In case of circular columns of diameters lesser than about 1,200 mm, the form panels are constructed in two half circles, while for column diameters larger than 1,200 mm, the column form panels can be constructed in four quarter circles. The form panels are connected to each other with bolts. Except for joints (where form panels meet), the concrete finish obtained with all metal circular forms is reasonably good. The horizontal and vertical joints are visible on the concrete surface after the forms have been dismantled. The joints usually require minor touch up works after the form removal. In order to improve the appearance of the joints after the form removal, sometimes a layer of foam is pasted on the angle sections at the joint locations both horizontally and vertically. The foam layer prevents leakage of slurry from the joints by making them tight fit.

In some cases, the circular column form panels are fabricated in single piece for the full height of the column. This avoids the joints and the whole column surface after the form removal looks uniform without any joint marks. However, in this case the form is to be slipped into the reinforcement cage already tied for the column formwork. This is slightly tedious and requires crane assistance.

The alignment of the circular column form can be done by using the props used for rectangular and square column formwork.

Figure 6.27 shows the details of a customized column formwork for casting concrete columns of 600 mm diameter and 6,150 mm height. The form panels have been fabricated using 4 mm steel sheets and flats of various sizes such as 50 mm × 6 mm, 60 mm × 6 mm, and 120 mm × 8 mm. Holes of 17.5 mm diameters have been left for fixing bolts of 16 mm diameter.

The details of the form panel for casting a starter of 200 mm height is also shown in the Fig. 6.27. The sheathing member is joined to the flats using 4 mm fillet welds as per Indian Standards (IS: 816–1969). The rate of pour for the given design is specified as 2 m/h. The form panels are constructed in two half-circles. The location of alignment props is also shown in the figure.

For the Delhi International Airport Project, the application of steel column formwork for casting rectangular columns is shown in Fig. 6.28. Some of the columns for this project were formed using steel formwork. Heavy duty towers were erected to support the working platform through which concrete was poured.

Figure 6.28 Column Construction Using Steel Formwork for Delhi International Airport Project.

6.9 ACHIEVING FORMWORK ECONOMY IN COLUMN CONSTRUCTION

Some of the ways in which economy in column construction can be achieved are described below:

6.9.1 Location

For maximum economy, as far as possible, standardized column location in a uniform pattern in both directions is recommended.

6.9.2 Orientation

The same orientation for as many rectangular columns as possible, should be used.

6.9.3 Shape

The use of same shapes as often as possible throughout any given floor, and vertically from floor to floor, is advisable for economy. Square or round columns are the most economical. The rectangular shapes should be used only when architectural, structural or flying form[1] (discussed in detail in Chapter 10) requirements so dictate.

6.9.4 Size

The column should be kept the same size throughout the building. If size changes are necessary, they should occur in 50 mm increments, one side at a time (Example: 600 mm × 600 mm column should go to a 600 mm × 550 mm, then 550 mm × 550 mm etc). This approach to changing column sizes results in material economies permitting gang forming[2] possibilities. For use with a flying form system, the distance between column faces and the flying form must be held constant. Column size changes should be made parallel to the flying form.

6.9.5 Varying Percentage of Steel

Based on formwork costs, it is more economical not to change column sizes from floor to floor, but rather to alter only the material strength and the percentage of reinforcing steel. In multi-story buildings, the largest column size is determined at the base of the building for a high strength concrete of, for instance, M50 and a reasonable maximum percentage of Fe415 steel. As the column progresses upward, the percentage of reinforcing bars decreases to a minimum of one percent. Then the M50 column is replaced by a lower strength concrete of, for example, M40. This process is continued until, after several floors, the column size can be substantially reduced. However, consideration should then be given to changing only one dimension of the column. It is often economical to change column sizes in increments of at least 30% to 50%, or more.

6.9.6 Avoiding Projections

Consider using steel shear heads: to avoid column capitals and drop panels in flat plate construction if the slab is at least 200 mm thick and if shear head will not interfere with either slab or column bars. Use small drop panels around columns in flat plates rather than tapered column capitals if shear strength must be increased without using steel (see Fig. 6.29).

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[1] Flying forms are large prefabricated units of formwork incorporating support and designed to be moved from place to place. These are explained in detail elsewhere in the book.

[2] Ganged forms are prefabricated panels joined to make a much larger unit for convenience in erecting, stripping and reusing; usually braced with wales, strongbacks, or special lifting hardware.

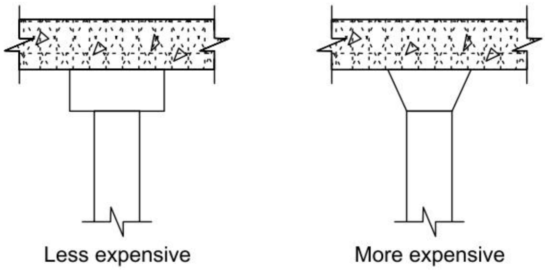


Figure 6.29 Two Alternatives of Column Capital.

6.10 DESIGN FOR COLUMN FORM

The different steps to design a column formwork are shown in Fig. 6.30.

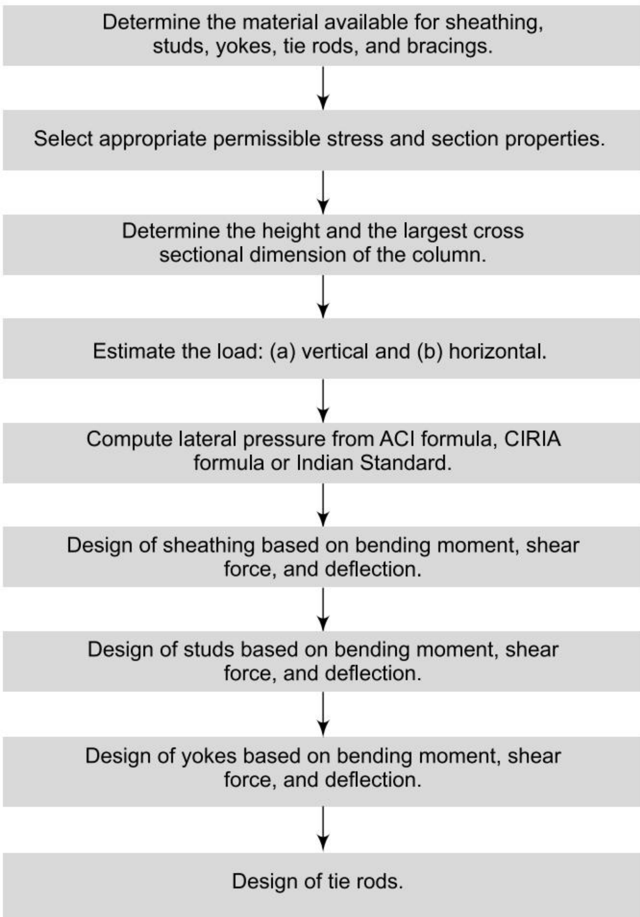


Figure 6.30 Steps in the Column Formwork Design.

6.11 ILLUSTRATION OF COLUMN FORMWORK DESIGN

Design the formwork for a column of cross section 350 mm × 350 mm, and a height of 3 m. A plywood of 12 mm thickness is available. Permissible bending stress on 12 mm plywood = 14 N/mm^2. Permissible bending moment = 0.2 kNm/m. Permissible shear force = 6.16 kN. Permissible deflection = span/360.

Timber of cross sections 50 mm × 100 mm, 100 mm × 100 mm, 100 mm × 150 mm, and 150 mm × 150 mm is available. Permissible bending stress for timber = 7 N/mm^2, E = 7,700 N/mm^2. Mild steel tie rod of 16 mm diameter is available. Dead load of concrete = 26 kN/m^3.

6.11.1 Computation of Lateral Concrete Pressure on Formwork

Maximum lateral pressure for columns,

$$P = C_w \times C_c \left[7.2 + \frac{785R}{T + 17.8}\right]$$

With a maximum of 150 $C_w C_c$ kN/m^2, a minimum of 30 C_w kN/m^2, but in no case greater than wh; where,

P = lateral pressure (kN/m^2)

R = rate of pouring concrete (m/h)

T = temperature in °C

Let's assume:

$$R = 2 \text{ m/h} ;\ T = 15°\text{C} ;\ \text{and } C_w \times C_c = 1$$

Permissible bending stress for timber = 7 N/mm^2

Permissible bending stress for plywood = 14 N/mm^2

Therefore, maximum lateral pressure,

$$P = 1 \times \left[7.2 + \frac{785 \times 2}{15 + 17.8}\right] = 55.066\ \frac{\text{kN}}{\text{m}^2}$$

6.11.2 Design of Sheathing

Based on bending moment

Maximum bending moment (assuming two spans), $M = \dfrac{w \times l_p^2}{8}$

To find l_p, assume that 12 mm thick plywood is being used,

Maximum permissible bending moment = 0.2 kNm/m

On substituting, we get, $\dfrac{55.066 \times l_p^2}{8} \leq 0.2$

$\Rightarrow \qquad l_p = 0.170 \text{ m} = 170 \text{ mm}$

Based on shear force

Assuming two spans

$$Q = \frac{5 \times w \times l_p}{8} \le 6.16$$

$$Q = \frac{5 \times 55.066 \times l_p}{8} \le 6.16$$

$$\Rightarrow \quad l_p \le \frac{8 \times 6.16}{5 \times 55.066}$$

$$\Rightarrow \quad l_p \le 0.178 \text{ m} = 178 \text{ mm}$$

Based on deflection

Assuming two spans,

$$\delta = \frac{w \times l_p^4}{185 \times E \times I} \le \frac{l_p}{360}$$

$$\Rightarrow \quad l_p \le \sqrt[3]{\frac{185 \times E \times I}{360 \times w}} \quad \Rightarrow \quad l_p \le \sqrt[3]{\frac{185 \times 1.07}{360 \times 55.066}}$$

$$\Rightarrow \quad l_p \le 0.215 \text{ m} = 215 \text{ mm}$$

Minimum of the above three values = 170 mm. Assuming 50 mm × 100 mm joist, keep the centre to centre distance of the studs as 150 mm only, since the side of the column is of 350 mm width. The arrangement of plywood and joists would be as shown below:

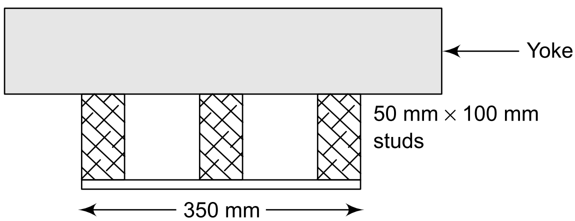


Figure 6.31 Column Formwork Side Arrangement.

6.11.3 Design of Joist

As mentioned earlier, the joist section proposed is 50 mm × 100 mm.

Area of cross section $A = 50 \times 100 = 5{,}000 \text{ mm}^2$

$$I = \frac{b \times d^3}{12} = \frac{50 \times 100^3}{12} = 41{,}66{,}666.67 \text{ mm}^4$$

$$E = 7{,}700 \text{ N/mm}^2$$

$$EI = 32.08 \text{ kNm}^2$$

$$Z = \frac{b \times d^2}{6} = \frac{50 \times 100^2}{6} = 83{,}333.33 \text{ mm}^3$$

Allowable bending stress, $f_b = 7 \text{ N/mm}^2$

Thus, allowable bending moment = $f_b \times Z_{xx} = 7 \times 83{,}333.33 = 5{,}83{,}333.3$ Nmm

= 0.5833 kNm

Allowable shear stress $f_q = 0.6$ N/mm^2

Thus, allowable shear force $= f_q \times A = 0.6 \times 5{,}000$ N $= 3{,}000$ N $= 3$ kN

Based on bending moment

Maximum bending moment (assuming continuous support conditions) $M = \dfrac{w_s \times l_s^2}{10}$

Maximum permissible bending moment = 0.5833 kNm

Load $w_s = 55.066 \times 0.150 = 8.26$ kN/m

On substituting, we get,

$$M = \frac{w_s \times l_s^2}{10} \leq 0.5833$$

$$\Rightarrow \qquad \frac{8.26 \times l_s^2}{10} \leq 0.5833$$

$$\Rightarrow \qquad l_s = 0.840 \text{ m} = 840 \text{ mm}$$

Based on shear force

Assuming continuous support conditions

$$Q = \frac{3w_s \times l_s}{5} \leq 3$$

$\Rightarrow \; l_s \leq \; 0.605$ m = 605 mm

Based on deflection

Assuming continuous support conditions

$$\delta = \frac{1 \times w_s \times l_s^4}{145 \times E \times I} \leq \frac{l_s}{360}$$

$$\Rightarrow \qquad l_s \leq \sqrt[3]{\frac{145 \times E \times I}{1 \times w_s \times 360}}$$

$$\Rightarrow \qquad l_s \leq \sqrt[3]{\frac{145 \times 32.08}{1 \times 8.26 \times 360}}$$

$$\Rightarrow \qquad l_s \leq 1.16 \text{ m} = \text{say } 1{,}160 \text{ mm}$$

Minimum of the above values is = 605 mm

Therefore, maximum span of the studs is = 605 mm, say 600 mm.

6.11.4 Design of Yoke

Assume a cross section of 100 mm × 100 mm.

$$b = 100 \text{ mm}$$

$$d = 100 \text{ mm}$$

$$A = b \times d = \; 10{,}000 \text{ mm}^2$$

$$I = \frac{100 \times 100^3}{12} \; = 83{,}33{,}333.33 \text{ mm}^4$$

$$Z_{xx} = \frac{I}{d/2} = 166{,}666.67 \text{ mm}^3$$

$$E = 7{,}700 \text{ N/mm}^2$$

Allowable bending stress, $f_b = 7$ N/mm^2

Thus, allowable bending moment $= f_b \times Z_{xx} = 1.167$ kNm

Allowable shear stress $f_q = 0.6$ N/mm^2

Thus, allowable shear force $= f_q \times A = 0.6 \times 10{,}000 = 6{,}000$ N $= 6$ kN.

Based on bending moment

Maximum bending moment (assuming simply supported conditions), $M = \dfrac{w_y \times l_y^2}{8}$

Maximum permissible bending moment = 1.167 kNm

Load $w_y = 55.066 \times 0.60 = 33.04$ kN/m

On substituting, we get,

$$M = \frac{w_y \times l_y^2}{8} \leq 1.167$$

$$\Rightarrow \quad \frac{33.04 \times l_y^2}{8} \leq 1.167$$

$$\Rightarrow \quad l_y = 0.531 \text{ m} = 531 \text{ mm}$$

Based on shear force

Assuming simply supported conditions,

$$Q = \frac{w_y \times l_y}{2} \leq 6$$

$$\Rightarrow \quad l_y \leq \frac{2 \times 6}{33.04} = 0.363 \text{ m} = 363 \text{ mm}$$

Based on deflection

$$\delta = \frac{5 \times w_y \times l_y^4}{384 \times E \times I} \leq \frac{l_y}{360}$$

$$\Rightarrow \quad l_y \leq \sqrt[3]{\frac{384 \times E \times I}{5 \times w_y \times 360}} \quad \Rightarrow \quad l_y \leq \sqrt[3]{\frac{384 \times 64.17}{5 \times 33.04 \times 360}}$$

$$\Rightarrow \quad l_y \leq 0.745 \text{ m} = \text{say } 745 \text{ mm}$$

Minimum of the above values is = 363 mm.

Therefore maximum span of the yoke = 363 mm say 360 mm.

However, the minimum span of the yoke obtained from the formwork scheme is = 350 + 2 × 12 + 100 + 100 = 574 mm. Thus we have to revise the yoke section or reduce the load intensity by reducing the spacing of the joist. Let's revise the section to 100 mm × 150 mm. Check for bending moment, shear force, and deflection as explained previously and thus check for adequacy of span of the yoke.

Alternatively, let's assume the span of the yoke approximately = 575 mm (minimum required = 574 mm), and find the section modulus required for the given permissible stresses.

Maximum bending moment = $\dfrac{w_y \times l_y^2}{8}$

$$= \frac{55.066 \times 0.575 \times 0.575}{8} = 2.275 \text{ kNm}$$

Permissible stress in timber = 7 N/mm^2 (given)

Section modulus required = $\dfrac{M}{f_b} = \dfrac{2.275 \times 10^6}{7} = 3{,}25{,}000 \text{ mm}^3$

Taking b = 100mm,

Depth of section required = $\sqrt{\dfrac{3{,}25{,}000 \times 6}{100}} = 139 \text{ mm}$

Thus, provide the yokes with a cross section of 100 mm × 150 mm.

Check the yoke section for shear and deflection.

Formwork scheme for the column is shown in the following figure:

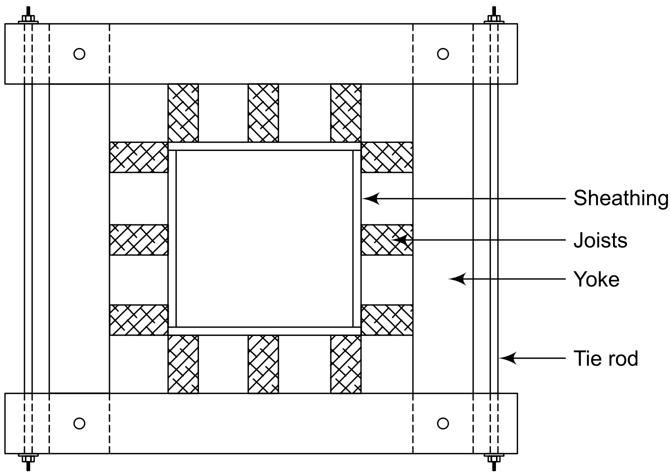


Figure 6.32 Arrangement of Different Components in the Column Formwork.

6.11.5 Design of Tie Rod

Alternative 1

Tie rods at the ends are provided as shown in Fig. 6.32. It may be pointed out that the tie rods in both the directions have been provided alternately, i.e at every 1,200 mm centre to centre.

Thus, load on each tie rod in each direction = 55.066 × 1.200 × 0.350/2 = 11.56 kN

The capacity of 16 mm diameter mild steel tie rod = (3.14 × 16^2) × 100/4 = 20,106 N = 20.10 kN

Thus, tie rods of 16 mm diameter are safe.

Alternative 2

Tie rods are placed diagonally.

For a 3.0 m height column box of sides 350 mm,

Column side = 350 mm

Yoke spacing = 600 mm

Design pressure of concrete for columns is = 50.066 kN/m^2

Width of loading on yoke = 600 mm

Horizontal force due to the pressure on each of the four yokes

$$= \frac{350}{1{,}000} \times \frac{600}{1{,}000} \times 55.066 = 11.56 \text{ kN}$$

At each of the two open corners of the column box, the yoke end will be subjected to the following forces:

where,

$$F_x = 11.56/2 = 5.78 \text{ kN}$$
$$F_y = 11.56/2 = 5.78 \text{ kN}$$

Resultant diagonal force, $R = 1.414 \times F_x$ (or F_y) $= 1.414 \times 5.78 = 8.17$ kN.

This force will be safely resisted by the tie rod which has a safe capacity of approximately 20 kN

6.12 EXAMPLE FOR COMPUTATION OF FORCE IN DIAGONAL TIE ROD OF COLUMN

Column height = 4.8 m

Column side = 600 mm

Waler spacing = 1,300 mm

Design pressure of concrete for columns is = 90 kN/m^2

Width of loading on waler = 1,300 mm

Horizontal force due to the pressure on each of the four walers

$$= \frac{600}{1{,}000} \times \frac{1{,}300}{1{,}000} \times 90 = 70.2 \text{ kN}$$

At each of the two open corners of the column box, the waler end will be subjected to the following forces:

where,

$$F_x = 70.2/2 = 35.1 \text{ kN}$$
$$F_y = 70.2/2 = 35.1 \text{ kN}$$

Resultant diagonal force, $R = 1.414 \times F_x$ (or F_y) $= 1.414 \times 35.1 = 49.6$ kN

This force will be resisted by the tie rod which has a safe capacity of 50 kN.

REVIEW QUESTIONS

Q1. True or False

(a) Main components of column formwork are: sheathings, studs, yokes, tie arrangement, and alignment arrangement.

(b) Various factors in achieving economy in column formwork are-location, orientation, shape, size, varying percentage of steel, avoiding projections.

Q2. Match the following:

(i) Column formwork KS × life	(a) Quickly erected and dismantled; 650 mm × 650 mm cross section.
(ii) Column formwork Top 50	(b) Used for forming columns, corners, stop ends and wall junctions; wall 1,050 mm × 1,050 mm cross sections.
(iii) Column formwork RS	(c) Circular columns.
(iv) Column formwork Framax × life	(d) Suitable for reusable complicated, large column heights, cross sections.
(v) Column formwork Frami × life	(e) Suitable for projects having large number of columns with varying cross sections; quick way to form quadratic and rectangular columns.

Q3. Sequence the following -(a) determine the height of the column (b) select appropriate permissible stress and section properties (c) determine the material available for sheathing, yokes, and batten (d) estimate the load (e) compute the lateral pressure (f) determine the largest cross sectional dimension of the column (g) select the sheathing material (h) determine the studs spacing.

Q4. Discuss the suitability and salient features of:

(i) Traditional formwork;

(ii) Proprietary formwork.

Q5. List out the various advantages and types of disposable column formwork.

Q6. List out the various features of Rapidobat formwork columns.

Q7. List out the salient features of the following PERI column formworks:

(a) Rapid column formwork;

(b) Vario GT 24 column formwork;

(c) Trio column formwork;

(d) Quattro column formwork;

(e) SRS steel column formwork;

(f) LICO light weight column formwork;

(g) Vario Quattro column formwork.

Q8. Compare and contrast the Doka and PERI column formwork systems.

Q9. What measures should be adopted to achieve economy in column formwork construction?

Chapter

7 Slab and Beam Formwork

Contents: Introduction; Traditional Slab and Beam Formwork; Slab and Beam Formwork Solutions Offered by L&T; Beam and Slab Formwork Solutions by PERI; Beam and Slab Formwork Solutions by Mivan; Achieving Economy in Slab Construction; Design of Slab and Beam Formwork; Illustration of Slab and Beam Formwork Design; Illustration of Proprietary Slab Formwork Design; Another Illustration of Slab Formwork Design

7.1 INTRODUCTION

In this chapter we discuss the conventional slab and beam formwork, and some proprietary slab and beam formworks. The main components of a slab and beam formwork are sheathing, joists, stringers, shores, and bracing. Diagonal braces in both the directions, though very important members in slab and beam formwork, are often found missing in the formwork scheme being implemented at project sites. Many formwork collapses in the past are attributed to the absence of the diagonal bracings. These bracings, in addition to ensuring stability of the formwork in the event of high winds and differential settlement of shores, also act as a framework on which the working platforms can be erected. These platforms are useful during form stripping operations. The diagonal braces between the top of the shore and the mid-height need to be provided at least in one bay along a row of shores of several bays.

In conventional slab and beam formwork, the emphasis is on all timber and all steel formwork. Under proprietary slab and beam formwork, we discuss L&T slab and beam formwork, PERI slab and beam formwork, and Mivan formwork. The design issues of both the types of slab and beam formwork are discussed.

7.2 TRADITIONAL SLAB AND BEAM FORMWORK

In traditional slab and beam formwork (see Figs. 7.1 and 7.2), the sheathing could be formed with either timber planks or plywood. In case of handset formwork, the sheathing is usually 1.2 m × 2.4 m sheets of plywood between 12 mm to 19 mm thicknesses. The joists are normally 100 mm × 100 mm lumber spaced about 300 mm to 400 mm on the center with spans mostly in the range of 1.2 to 1.8 m. Stringers in most cases are of the cross-section 100 mm × 150 mm timber with spans from less than 1 m to about 1.5 m.

The shores are usually two pieces of 100 mm × 100 mm (4″ × 4″) lumber clamped together in the middle (see Figs. 7.3 and 7.4). The shores are connected to a bedplate so as to distribute their pressures in a uniform manner on the soil in which they are embedded. The shores are connected to the adjacent ones by a system of horizontal braces. The braces are usually fixed at about 1.30 to 1.50 m below the sheathing. For a shore of height in the range of 3.0 – 3.5 m, that is somewhere slightly above the mid-height of the shores. This leaves a clear height of about 1.80 m – 2.0 m from the ground which can allow workmen to freely move about below the formwork.

The use of a traditional formwork system may be characterized as a labor-intensive and time-consuming operation. Not only the forms are tailor-made manually on the site, but the concrete surface also frequently requires man-hours for further finishing after stripping. In addition, the falsework involving formwork is one of the activities causing major casualties at construction job sites, and hence, hampers the worker's safety. Consequently, the activity of formwork construction is often critical in a project, affecting it in terms of time, cost, quality, and safety. Joints in traditional slab and beam formwork require special considerations and are dealt with, in the following section.

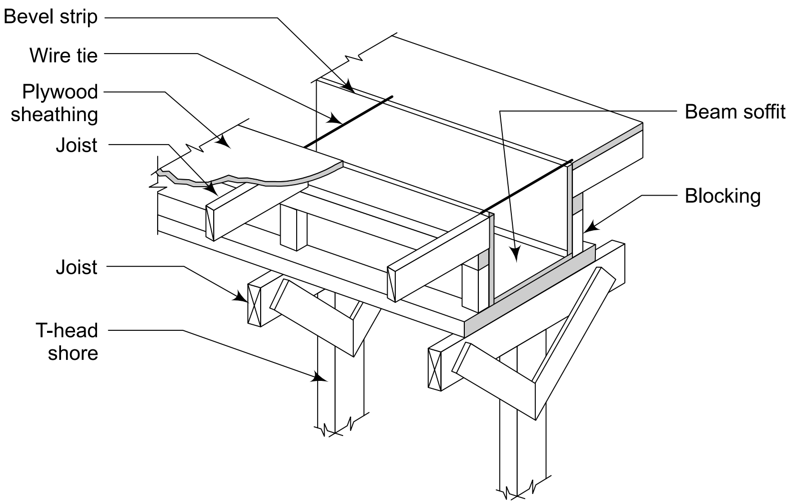


Figure 7.1 Typical Components of Beam Formwork with Slab Forming in.

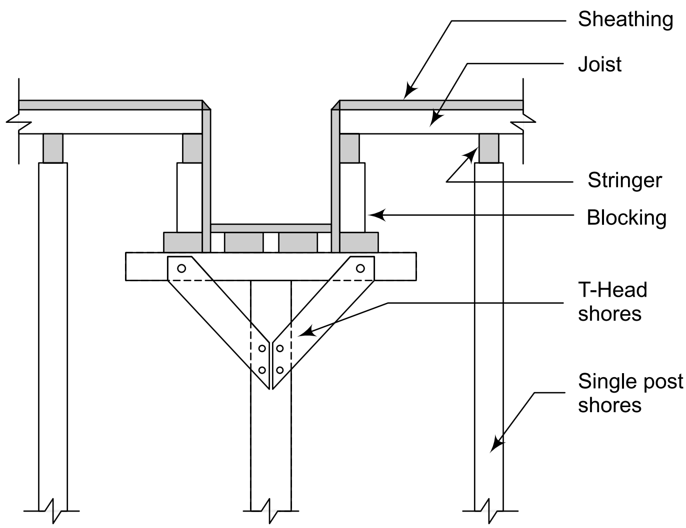


Figure 7.2 Typical Slab Form Resting on Beam Ledger and Stringers.

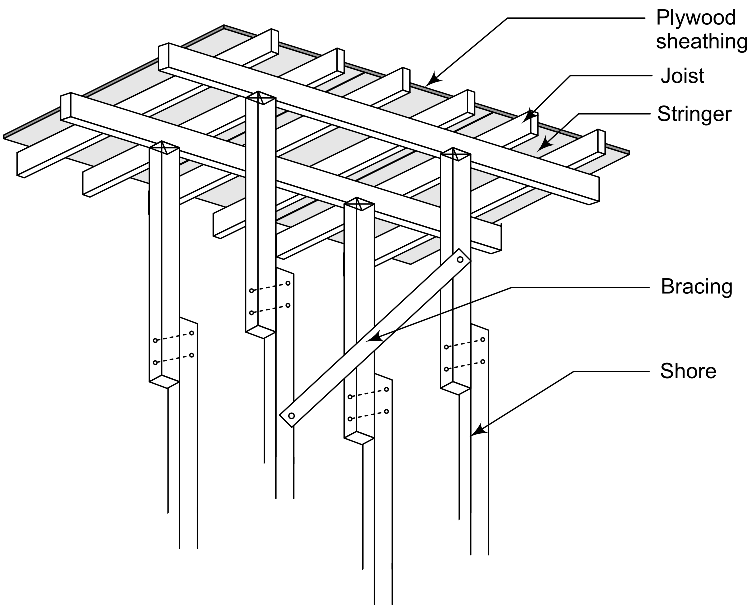


Figure 7.3 View of Traditional Slab Formwork.

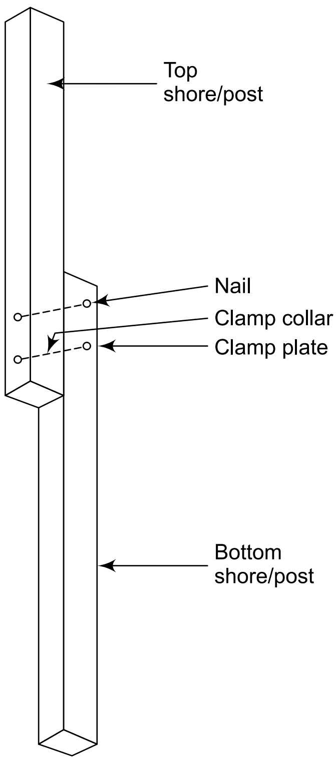


Figure 7.4 Clamping of Shores.

7.2.1 Joints in Traditional Slab and Beam Formwork

Joints in a formwork are temporary in nature. This is understandable as most of the formworks are assembled at the site with a view of dissembling them later. These joints are thus not at all rigid when either the lateral or unbalanced gravity loads act on them. One of the requirements for formwork design is thus to ensure the stability of formwork systems under the application of loads. In order to ensure the stability, specific attention needs to be given to the bracings and the connection details of various formwork elements.

The factors to be specifically given attention to, are:

(i) the joints between the posts and joists, and

(ii) the framework of the posts through a system of horizontal braces and inclined shores.

Joints between posts and joists

The joints between the posts and joists are often made only by nailing in case of traditional slab and beam formwork (see Fig. 7.5). The number of nails range from 2 to 4 depending on the carpenter's preference. The nails are usually 100 mm wire nails with plain flat heads. At such a joint, the post is neither restrained from rotation, nor from lateral movement when lateral forces act. This is more true when the posts are not truly vertical, which often is the case when slightly longer posts are tilted for the purpose of leveling the sheeting. Two small cleats, of size 50 mm × 50 mm × 28 mm thick, nailed to the bearer/joists on either side of the post will ensure restraint from lateral movement. This will also cause the effective length of the post to be smaller, thus contributing to adequate structural stability. Gross movements will also not occur even when, accidental knocking of the posts may take place. The extra labor required is small compared to the safety advantage that it offers.

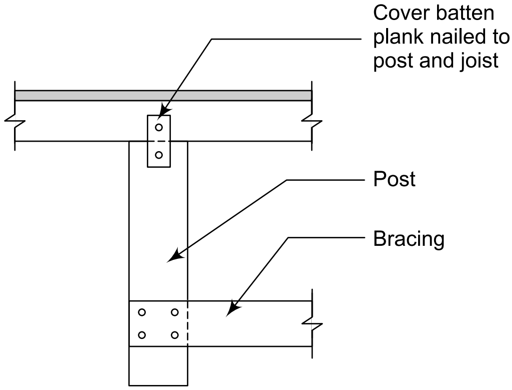


Figure 7.5 Joining Details of Posts and Joists, and Posts and Bracings.

Joints between beam bottoms and posts

Joints between the beam bottoms and posts also need attention. Figure 7.6 illustrates a typical joint. In this case, the beam bottom does not rest directly on the post, but rests on a cross member which is first nailed to the post, and then the joint is stiffened by raker members nailed both to the post and the cross bearer one on either side of the post, so as to reduce the effects of constructional eccentricity.

The steel form panels used for walls explained in earlier chapters can also be used for forming slabs. However, lighter panels, called slab panels or floor forms, are also available. These are usually

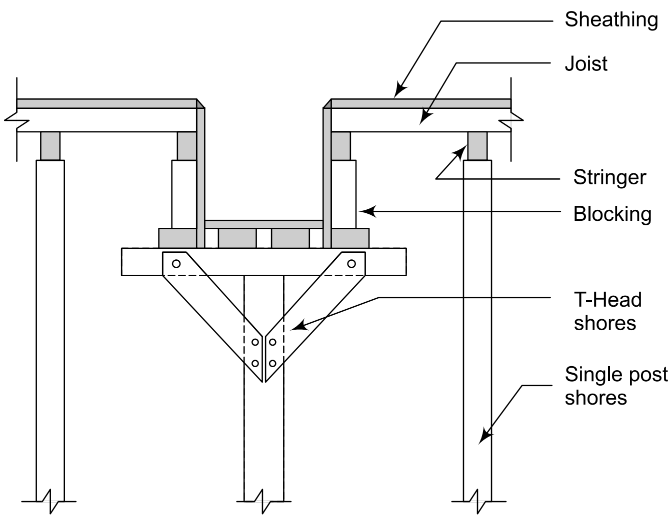


Figure 7.6 Joining Details of Posts and Beam Bottoms.

light gauge M. S. sheets with pressed flanges and stiffeners for rigidity and strength. They are also made in standard modular sizes. To fill up the odd gaps, adjuster panels are also available.

7.3 SLAB AND BEAM FORMWORK SOLUTIONS OFFERED BY L&T

For slab and beam formwork L&T offers-(1) L&T flex, (2) Beam forming support system, and (3) Heavy duty tower. These are discussed briefly in the following sections.

7.3.1 L&T Flex

L&T props are called L&T flex. These are adjustable props and are available for various heights ranging from a minimum of 1.41 m to a maximum of 4.1 m. The maximum load that can be taken by an L&T Flex is 34 kN. L&T has four types of props, namely CT 410 (Range 2.31 – 4.1 m), CT 340 (Range 1.91 – 3.4 m), CT 300 (Range 1.71 – 3.0 m) and CT 250 (Range 1.41 – 2.5 m).

This system has unique components of a tripod and a four-way head. Unlike other systems, no bracings are required between the props. The tripod takes care of stability. Four-way heads have sufficient space to hold two H-16 beams. This facilitates lapping of H-16 beams without cutting. L&T flex, folding tripods, four-way head, and H-16 beams combine to give an unusually versatile formwork system to tackle diverse types of floors with varying room dimensions. The arrangements of L&T flex for normal beam and slab construction, and inclined floor construction are shown in Figs. 7.7 and 7.8, respectively. The sheathing member used in both cases of formwork is 12 mm plywood.

A special component, called the beam forming head, is available in L&T formwork system (see Fig. 7.7). Generally for beams having, a very small section, the shuttering becomes uneconomical because the capacities of the various props used are not fully utilized. To counter this problem, a beam forming head is used because of which, a single prop can be used below the beam rather than using two props on either side of it. The beam forming heads can be used for beams having depths up to 450 – 500 mm.

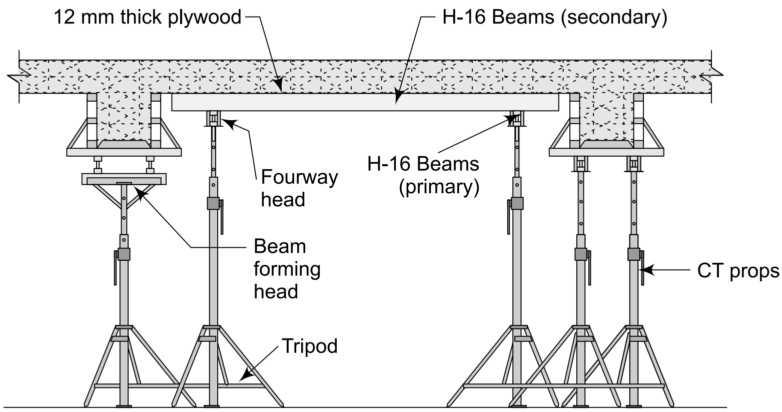


Figure 7.7 Typical Arrangement of L&T Flex System.

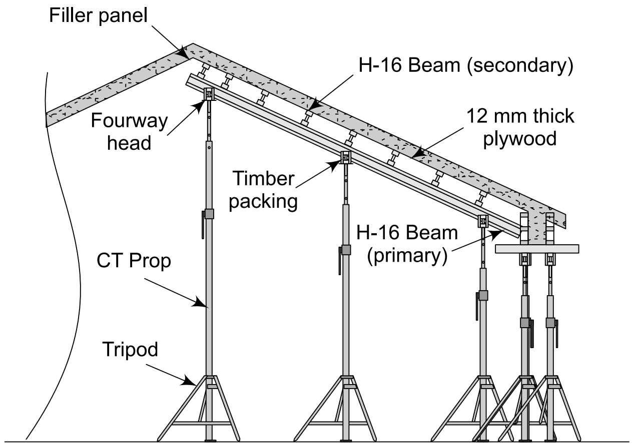


Figure 7.8 Typical Arrangement of L&T Flex System for Inclined RC Floors.

7.3.2 L&T Beam Forming Support System

Figure 7.9 shows a typical beam forming support system. This system eliminates the use of timber for the beam sides and the beam bottom. Because of this, there is no making involved for the beam sides and the beam bottom. Only the plywood is cut to size and placed. The system is suitable for beam depths from 300 mm onwards and an increment of 10 mm in depth of the beam is possible. The system is easy to assemble, align, fix, and deshutter. The system ensures the right level, accurate dimensions, and the right angle of side formwork with respect to the beam bottom.

7.3.3 L&T Heavy Duty Tower

One limitation of the L&T flex is that it can reach a maximum height of 4.1 m. Beyond that, L&T heavy duty towers are to be used. A typical L&T heavy duty tower application is shown in Fig.7.10. These towers have a safe capacity of 62.5 kN per leg for a height of 5.5 m. The capacity of each leg is almost double of the per leg capacity of other system towers. Because of this, the L&T towers can be placed at much larger spacing, for example, at 3 m c/c. But this will necessitate using structural steel channels over the towers as runners especially when the towers are provided below heavy beams.

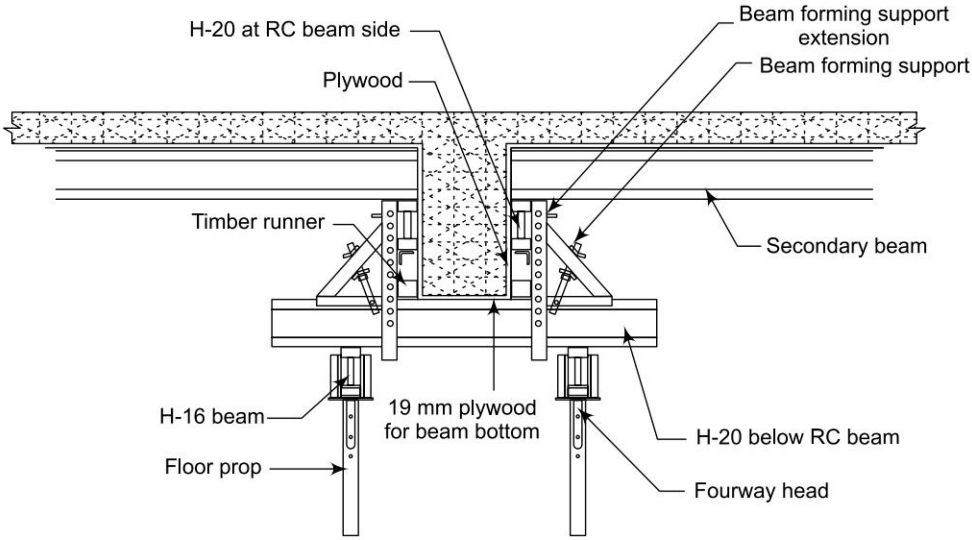


Figure 7.9 Beam Forming Support System.

Also no bracings are required between the towers, and each tower is self-supportive. The towers are provided with a two-way head and spindles both at the top and the bottom. The spindles have an adjustment range of 450 mm. Large and heavy table forms can be made out of the towers by fitting wheels to them for easy transporting and resetting. The units can also be lowered far enough to enable them to be rolled out from under the floor with the deep beam without dismantling.

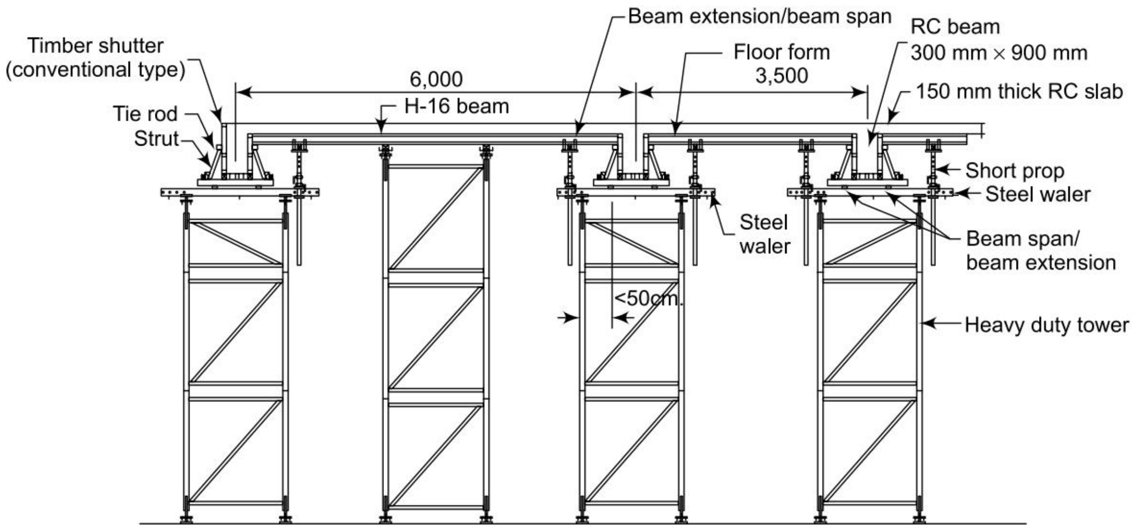


Figure 7.10 L&T Heavy Duty Tower.

7.3.4 Illustration of L&T Slab and Beam Formwork System

The L&T slab and beam formwork system can be applied for numerous applications. The application of the L&T flex system used in flat slab construction is shown in Figs. 7.11 and 7.12. The neat arrangement of the flex and the wide space available below the decking can easily be observed.

Figure 7.11 Construction of an RCC Floor for a Hotel Project Executed by Banzai Estate Ltd., Chennai.

Figure 7.12 Re-propping Strip Along with the Props at the Construction Site.

The application of the L&T heavy duty tower system is shown in Fig. 7.13 for casting waffle slabs for a commercial complex. The figure shows the laying of the waffles on the decking prepared using H-16 beams. It is also possible to use the L&T flex for casting the waffle slab if the height of the floor permits its use.

Figure 7.13 Commercial Complex for Mantri Group at Bangalore.

In Fig. 7.14, the heavy duty tower system has been used to cast beams and slabs for one of the buildings at Delhi International Airport. In the figure, the beam bottom is under preparation. H-16 beams have been used to support the beam bottom. The H-16 beams are resting on the walers, which in turn are supported on the U-head. The heavy duty towers need to be braced to each other when the height exceeds 6.0 m. The bracing is to be done as per the guidelines of the manufacturer. The tower spindles and the foot plates need to be provided in each leg of the heavy duty tower.

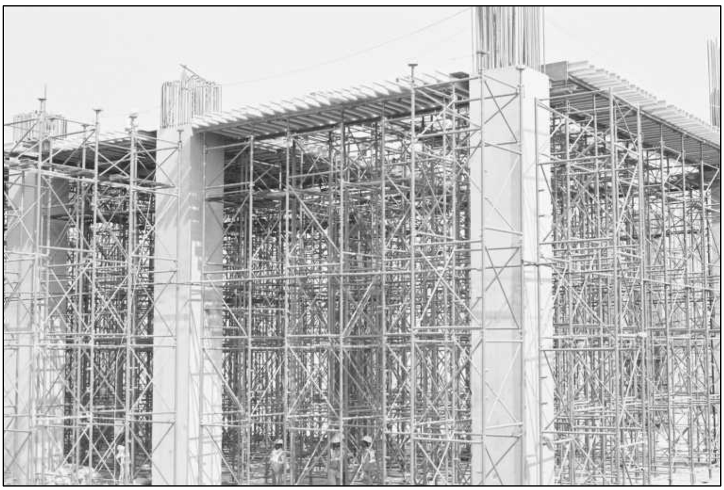


Figure 7.14 Beam and Slab Formwork Under Preparation at Delhi International Airport.

In Fig. 7.15, the beam formwork is implemented using a heavy duty tower for construction of a factory building for Rallis India Ltd. The height of the towers used here is comparatively less than the one shown in Fig. 7.14. The footplate and the tower spindle in each leg of the tower can be clearly seen. The H-16 in two layers can also be noticed. The side shutters for the beam are shown to be removed in the figure.

Figure 7.15 A Heavy Duty Tower in Position for Beam Formwork.

The application of a heavy duty tower formwork for constructing curve shaped gallery beams is shown in Figs. 7.16 and 7.17. These beams have a typical curve profile as shown in the Fig. 7.16 and are 1,250 mm (maximum) deep and 400 mm wide. At the peak, the contractor mobilized 12 sets of beam bottom formwork and 4 sets of side formwork which resulted into producing four beams per week. The carpenters on an average produced 4.0 m^2 per man day. Each beam comprises of a shuttering area of around 90 m^2 taking one half on each side of the bay into consideration and a concrete quantity of 13 m^3 per beam including the *cast-in-situ* slab portion.

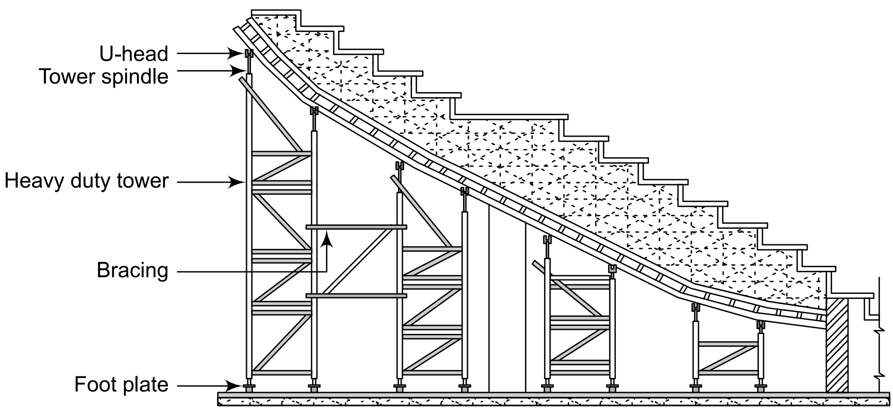


Figure 7.16 Typical Profile of the Curve Shaped Gallery Beam.

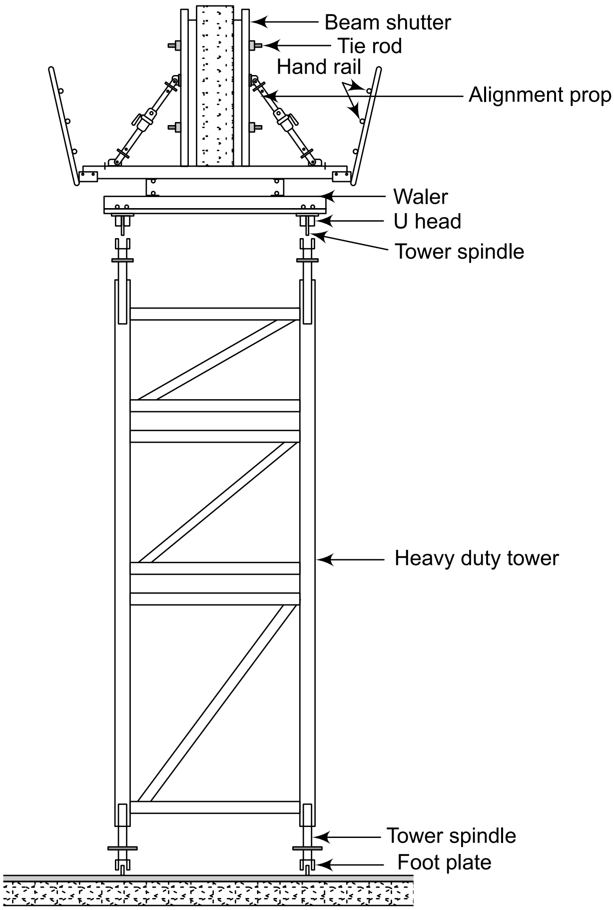


Figure 7.17 Typical Formwork Arrangement.

7.4 BEAM AND SLAB FORMWORK SOLUTION BY PERI

Some of the solutions for slab and beam formwork offered by PERI are: (1) Skydeck Aluminum Panel Slab Formwork, (2) Gridflex Aluminum Grid Slab Formwork, and (3) Multiflex Girder Slab Formwork. The features of each of these slab and beam formworks are given in the following sections:

7.4.1 Skydeck Aluminum Panel Slab Formwork

This system is suitable for forming slabs up to thickness of 950 mm, and beams of up to 300 mm width and 600 mm depth. The system consists primarily of the main beam, the panel, the drophead, and the cover strip. The aluminum panel (see Fig. 7.18) comes in various sizes such as 1,500 mm × 750 mm, 1,500 mm × 500 mm, 1,500 mm × 375 mm, 750 mm × 750 mm, 750 mm × 500 mm, and 750 mm × 375 mm, to suit various applications. The main beam is of an aluminum section and comes in a standard size of 2,250 mm. Filler beams of 1,500 mm and the beams for the cantilever portion of 3,750 mm length are also offered in this system. The system offers various choices for the dropheads to suit different applications. These dropheads enable most of the formwork materials to be removed quickly which can be reused at other locations. The system can be fitted with platforms for enhanced safety. The platforms have provisions for guard rails. The system also offers cover strips, an edge beam and filler timber for forming various irregular shapes in the plan. All the components are light in weight and thus suitable for hand set form applications. The skydeck panels and the main beams have self draining edges thus requiring lesser time for cleaning compared to the other systems. Figures 7.19 and 7.20 show the details of the Skydeck aluminum panel slab formwork.

Figure 7.18 Aluminum Panel Used in Skydeck Aluminum Panel Slab Formwork (Courtesy PERI).

Figure 7.19 Skydeck Aluminum Panel Slab Formwork (Courtesy PERI).

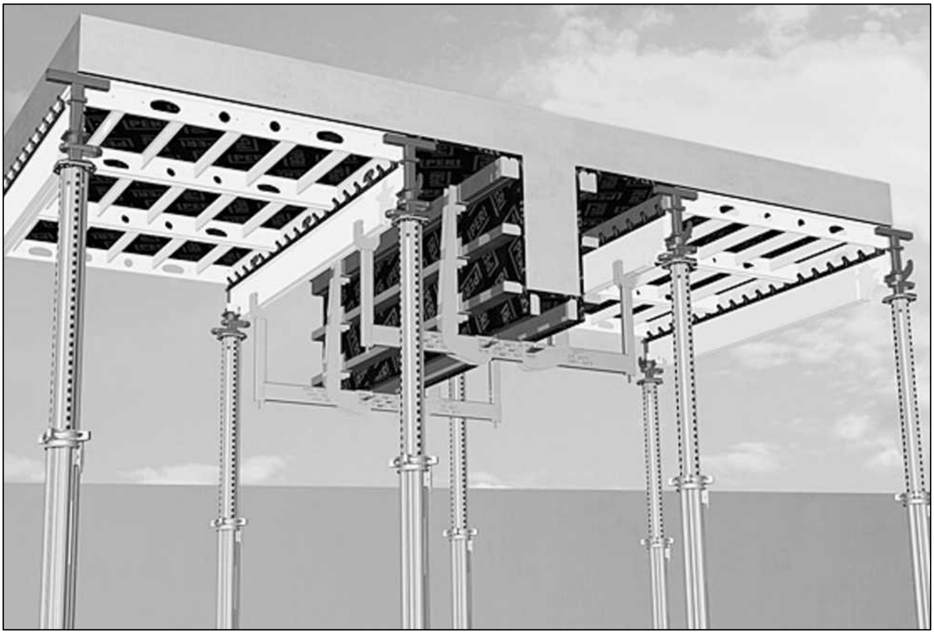


Figure 7.20 Arrangement of Formwork for Beam and Slab in Skydeck Aluminum Panel Slab Formwork (Courtesy PERI).

7.4.2 Gridflex Aluminum Grid Slab Formwork

A typical Gridflex Aluminum Grid Slab Formwork is shown in Figs. 7.21 and 7.22. This system of formwork consists of the following components:

(i) Grid elements of 2,000 mm length and 1,000 mm width;

(ii) Filler elements of various sizes to suit the different requirements in transverse and longitudinal directions;

(iii) Prop heads and Props;

(iv) Wall support.

Each of the above mentioned components are light in weight and thus ideal for hand set forming. The grid elements can be positioned at the desired elevation either from the ground or from the existing floor level. The system offers great safety due to the existence of a panel grid system which allows safe access for laying the sheathing material. Plywood panels are used as sheathing members in this system. The wall supports shown in Fig. 7.21 are used to fix the gridflex both longitudinally and transversely to the building, and guarantees a high level of safety from the very beginning itself. The system can be used for any building shape in the plan.

For early striking, the gridflex beams are installed. These beams continue to be supported even after concreting has taken place. However, the other components can be removed and can be used for the next cycle.

Figure 7.21 Wall Supports Being Installed for Both the Longitudinal and Transverse Directions (Courtesy PERI).

Figure 7.22 Gridflex Panels Being Erected from the Existing Floor (Courtesy PERI).

7.4.3 Multiflex Girder Slab Formwork

This system of formwork consists of plywood as the sheathing member, formwork girder, props, cross heads, and galvanized tripods. The girders could be either GT 24 or VT 20K depending on the thicknesses of the slab. For higher thicknesses, the heavier girder GT 24 is used while for lesser slab thicknesses, the lighter girder VT 20K is used. The system has light components and is suitable for hand forming. The system can be used for any ground plan. The props, if replaced with shoring towers, can enable the system to be used for virtually any height. Figures 7.23–7.25 show arrangements of the different components in the Multiflex girder slab formwork.

Figure 7.23 Multiflex Girder Slab Formwork (Courtesy PERI).

Figure 7.24 Multiflex Girder Slab Formwork (View showing Primary, Secondary Beams and Shores with Tripod) (Courtesy PERI).

Figure 7.25 Multiflex Girder Slab Formwork (All Components in Position) (Courtesy PERI).

7.5 BEAM AND SLAB FORMWORK SOLUTION BY MIVAN

The use of aluminum formwork for the construction of residential houses in mass housing projects is also getting popular these days. A large number of aluminum formwork manufacturers exists worldwide. However, in the following sections, we discuss Mivan formwork in detail which has found a number of applications in different cities such as Delhi, Mumbai, Gurgaon, etc. in the recent past. Mivan formwork is found to be appropriate in housing projects where large numbers of similar houses are to be constructed in a short duration. In this system of formwork, all the elements of a building namely, load bearing walls, columns, beams, floor slabs, stairs, balconies etc. are cast in one continuous pour of concrete resulting in a monolithic structure.

Some of the advantages offered by the Mivan formwork system and other equivalent systems are given below:

7.5.1 Advantages

- The system is simple, adaptable (flexible) and cost-effective. The materials used in such formwork systems are strong, sturdy, precisely-engineered, and accurate in dimension. The form system can be repeated a large number of times (the possible reuse could be even up to 250).
- It produces total quality work. The concrete formed with such formwork systems is more durable.

- It is possible to develop a customized solution using such formwork systems for any type of building; for example, a framed structure involving column beam-slab elements or for a box-type structure involving a slab-walls combination.
- Due to the modular nature of the formwork system, easy fixing and removal of the formwork is possible, resulting into a faster construction cycle. The result is a typical 4 to 5 day cycle for a floor-to-floor construction.
- The operations involved are simple, and can be even performed by unskilled crew members. The system does not require any sophisticated tool and most of the time, most operations can be performed with a hammer.
- The erection of the formwork components used in this system is achieved manually, and thus, no mechanical equipment for handling and erection is required.

7.5.2 Components of Mivan Formwork

The basic component of the Mivan formwork is the lightweight panel made of an extruded aluminum rail section and welded to an aluminum sheet. The panels are made from a high strength aluminum alloy with a 4 mm thick skin plate and 6 mm thick stiffeners. The panel has good stiffness to weight ratio and thus, ensures minimal deflection under the application of loads. Panels are manufactured in the size and shape to suit the requirements of the specific projects.

Figure 7.26 shows the major components used in aluminum formwork. The purpose of these components is also mentioned in the figure.

Mivan formwork components

Beam components

1. Beam side panel
   - to form the side of the beams.
2. Prop head for beam soffit
   - to form the soffit beam.
3. Beam soffit panel
   - to support the soffit beam.
4. Beam soffit bulkhead
   - as the bulkhead for the beam.

Deck components

1. Deck panel
   - to form the horizontal surface for the casting of the slabs.
2. Deck prop
   - to support the deck and to support the load form the deck panel.
3. Prop length
   - length of the prop.
4. Deck mid beam
   - to support the middle portion of the beam.
5. Soffit length
   - to provide support to the edge of the deck panels.
6. Deck beam bar
   - to support the deck and the beam.

Wall components

1. Wall panel
   - to form the face of the wall.
2. Rocker
   - to support the wall component.
3. Kicker
   - to form the wall face at the top of the panels.
4. Stub pin
   - Can be used to join two wall panels .

Other components

1. Internal soffit corner
   - to form the vertical and the horizontal internal cornice.
2. External soffit corner
   - to form the external corner between the components.
3. External corner
   - to form the external corner of the formwork system.
4. Internal corner
   - to connect the two pieces of the vertical formwork at their exterior intersections.

Figure 7.26 Aluminum Formwork Components and their Purposes.

The application of the above components in wall, beam, and slab formwork is shown in the schematic Figs. 7.27–7.29. Figure 7.27 shows the application of the internal corner, the external corner, the wall panel, and the wall ties in the wall formwork assembly. Figure 7.28 shows the application of the prop head for soffit beam, the internal beam side panel, the soffit length, the external beam side panel, and the kicker panel in assembling the beam formwork. Figure 7.29 shows the application of the deck prop, the beam bar, the mid beam in the slab formwork assembly.

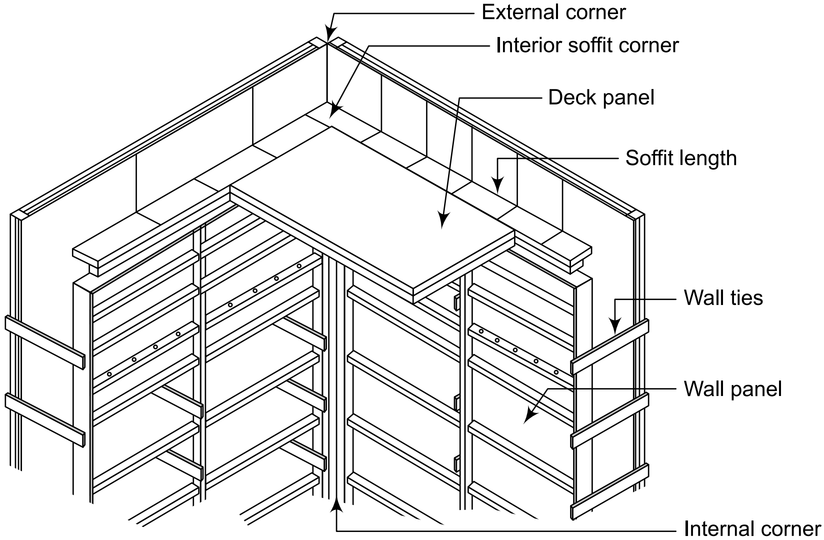


Figure 7.27 Wall Assembly Details (Courtesy Mivan).

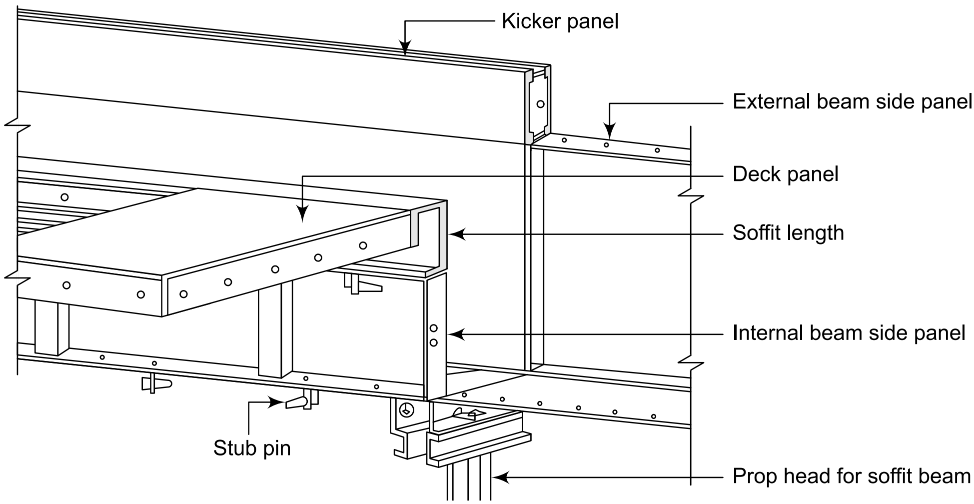


Figure 7.28 Beam Assembly Details (Courtesy Mivan).

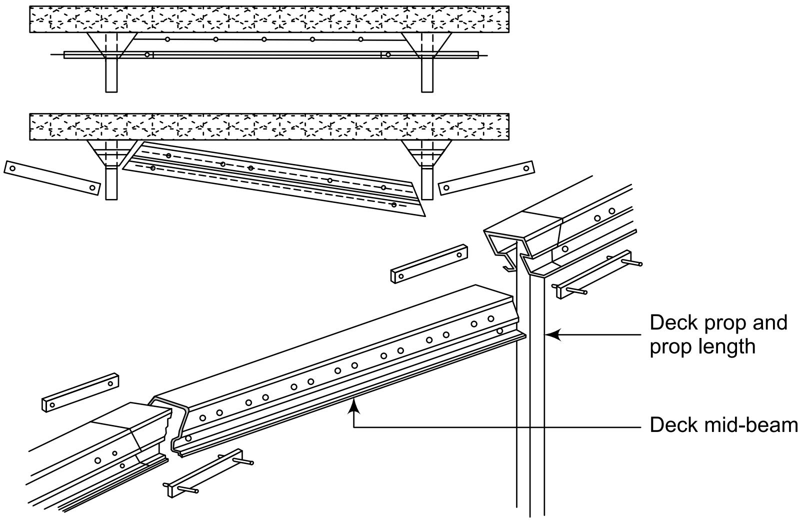


Figure 7.29 Assembly of Slab Formwork (Courtesy Mivan).

7.5.3 Construction Steps in Mivan Formwork Application

Step 1: Setting out and the survey; adjustment, if any, and timber stay fixing along the setting out line. The level of the previously cast floors is ascertained. The high spots on the concrete floors are marked (+) while the low spots are marked with a (–) sign. The (+) marked spots are chipped off to the required level while the (–) marked levels are provided packing using the plywood or timber pieces of required thicknesses. The level surveys need to be carried out on the top of the kickers after concreting, to maintain the verticality of the formwork. In order to ensure the formwork erection to the setting out lines of the wall and the columns, timber or ply stays are nailed in the concrete close to the internal and the external corners of the wall and the columns. This is shown in Fig. 7.30.

Figure 7.30 Setting Out and Timber Stay Fixing.

Step 2: Vertical rebar fixing and the first stage mechanical and electrical works.

Reinforcements for the wall and the columns are tied as per the drawings. The first stage works involved with the mechanical and electrical works are also attended to in this step. The reinforcement tying for the wall is shown in Fig. 7.31 while the mechanical and electrical works in progress can be seen from Fig. 7.32.

Figure 7.31 Vertical Rebar Fixing.

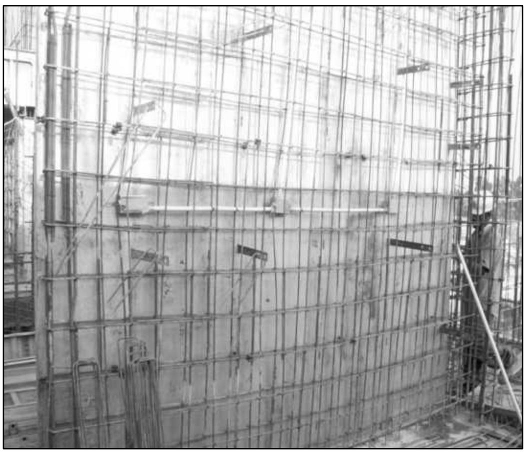


Figure 7.32 Mechanical and Electrical Works in Progress.

Step 3: Wall, column, and beam formwork erection.

Before erecting the wall formwork, the form panels are cleaned and coated with the form release agents. The wall formwork is erected from the internal corner in both the directions for stability reasons. The form panels are connected with pins and wedges with a hammer operation. Wall ties which are also coated with the release agents for easy removal, are provided. In case of a long wall or windy conditions, some temporary bracings to the wall formwork may need to be provided. The erection of the wall formwork is shown in Fig. 7.33. The provision of a door opening is also made in the formwork. The arrangement for the door opening is shown in Fig. 7.34.

Figure 7.33 Wall Erection in Progress.

Figure 7.34 Door Opening.

Erection of the column formwork.

Normally, the column reinforcement is so congested, it's difficult to fix the standard wall ties. In this case, steel walers are to be provided to withstand the concrete pressure using a DP tie rod and a wing nut. A column more than 1,000 mm long requires additional tie rods in the center of the column. The arrangement for the column formwork is shown in Fig. 7.35.

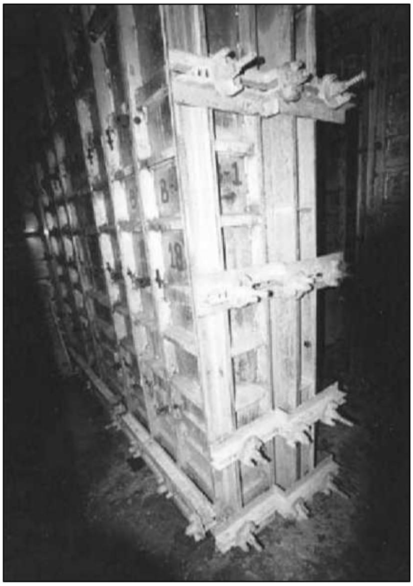


Figure 7.35 Column Erection.

Erection of the beam formwork.

The beam soffit to the prop head is pinned. The prop length is fitted to the prop head. The beam soffit is lifted in to the position and pinned to the column collar. After completing the beam soffit, the beam side panel is erected. The arrangement is shown in Fig. 7.36.

Figure 7.36 Beam Erection.

Step 4: Deck/soffit formwork erection.

All soffit corners and soffit lengths are fixed prior to the deck erection above the wall and the beam. When connecting the soffit corner and soffit length to the vertical formwork, the pin is inserted from the top to prevent the pin from falling out during concreting. After erecting the soffit corner and the soffit length, a deck panel is pinned to the corner to retain its diagonal. The beams with the deck prop are connected by using the beam bar as per the assembly drawings. The pre assembled beam sets are lifted to the location and pinned to the deck panels. The same procedure is followed for closing the deck formwork. The arrangements are shown in Figs. 7.37 and 7.38.

Figure 7.37 Deck Erection.

Figure 7.38 View from Top of the Deck.

Step 5: Slab rebar and the second M&E works

Reinforcement for the slab is tied as per the drawings. The second stage works involved with the mechanical and electrical works are also attended to in this step. The reinforcement tying for the slab and the mechanical and electrical works in progress can be seen in Fig. 7.39.

Figure 7.39 Deck Rebar and Second M&E Works in Progress.

Step 6: Sunken portion formwork fixing, the spacer and the vertical soldier fixing.

The formwork for the sunken portion is fixed besides fixing of the spacer and the vertical soldier. The vertical soldier is used to maintain the verticality of the beam side by using the tie rod and the wing nuts. Spacing of the vertical soldier is approximately 1,000 mm. The same slot can be used for fixing the walkway brackets

Step 7: Vertical and horizontal alignment and checking.

The vertical and horizontal alignments are checked.

Step 8: Concrete pouring.

Concrete is poured with appropriate equipments. Figure 7.40 shows the concrete pouring in progress.

Figure 7.40 Concrete Pouring in Progress.

Step 9: Striking of the wall formwork.

After the concrete has gained the required strength, the striking of the wall formwork commences. The wall forms are struck after about 12 hours of concrete pouring. The relevant standards should be referred to while striking the wall formwork.

The striking of the wall formwork is shown in Fig. 7.41. Sometimes, the wall formwork can be shifted to the upper floors using predefined openings made in the slab. These openings are filled up later. This arrangement of shifting of the formwork through the opening is shown in Fig. 7.42.

Figure 7.41 Striking of Wall Formwork.

Figure 7.42 Lifting through Openings.

Step 10: Striking the deck form with the left out props

Erection of the bracket for the next floor (see Fig. 7.43) commences immediately after the striking of the wall formwork for the previous floor. The props are left out for the duration specified by the relevant standards (see Fig. 7.44). The reshores are also left upto the specified duration and in the requisite number of floors as suggested by the designer.

Figure 7.43 Erection of Brackets for the Upper Floor.

Figure 7.44 Left out Props in Position.

7.5.4 Construction Cycle

Aluminum formwork compares favorably with any other type of formwork when it comes to delivering a competitive construction cycle time. It is possible to achieve as low as a 3-day construction cycle. The cycle time gets affected by a number of factors such as: the complexity and the shape of the structure, the layout of the floors, the dependence between the various activities, the skill level of the workmen, the available working hours, and the permissible striking times, etc. In some cases, the cycle time can go up to six days. However, the most typical cycle time for estimation purposes could be considered as a 4-day cycle. For illustrating a 4-day cycle, let's consider a typical floor plan of a multi-story building (see Fig. 7.45). It may be pertinent to note that the typical cycle time may not be applicable for the first few floors of the building. As can be seen, the floor has been divided into four regions.

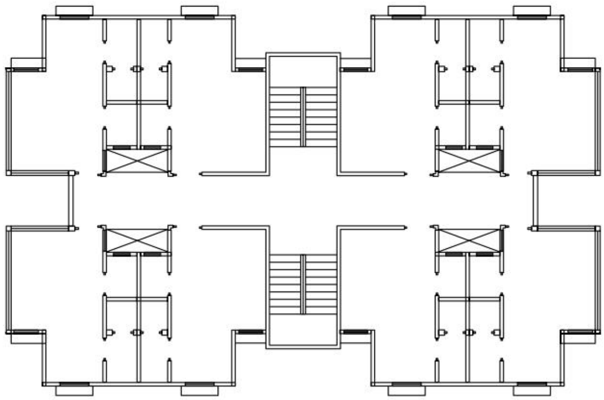


Figure 7.45 A Typical Floor Plan of a Multi-story Building to Illustrate the Cycle Time.

The day-wise details of the activities carried out to achieve a 4-day cycle time is given in Fig. 7.46.

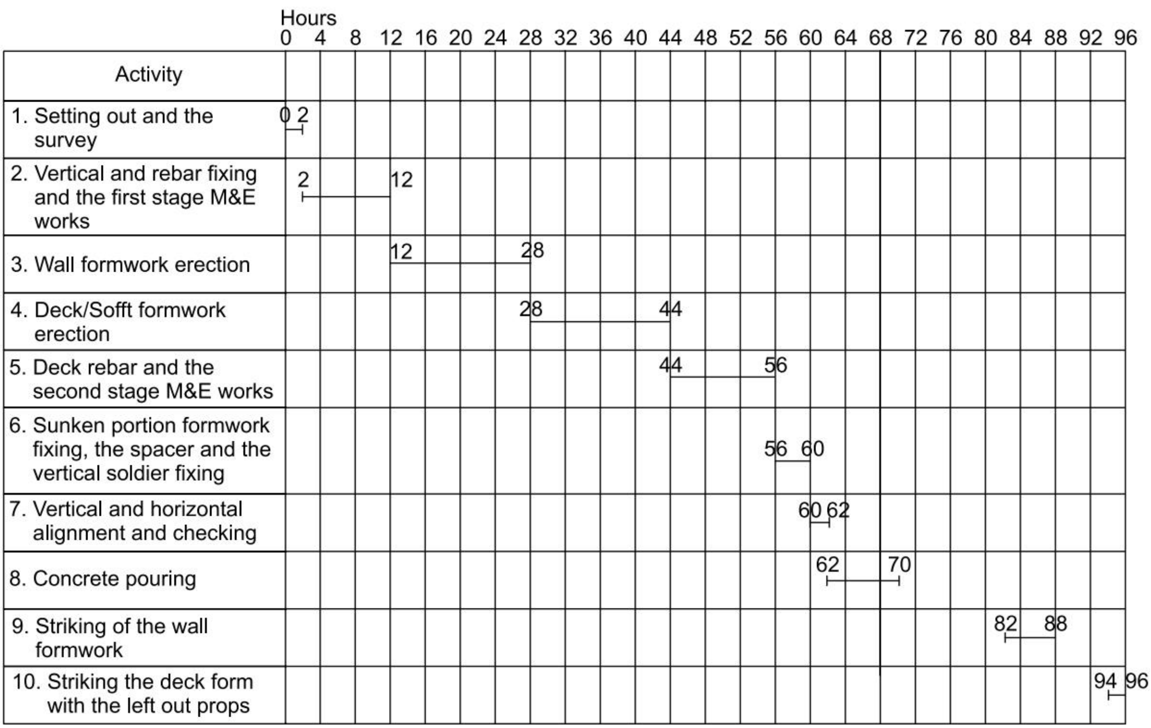
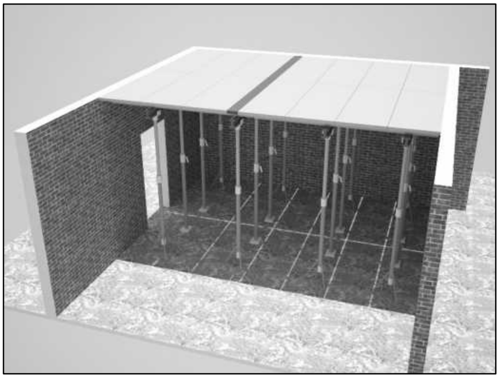


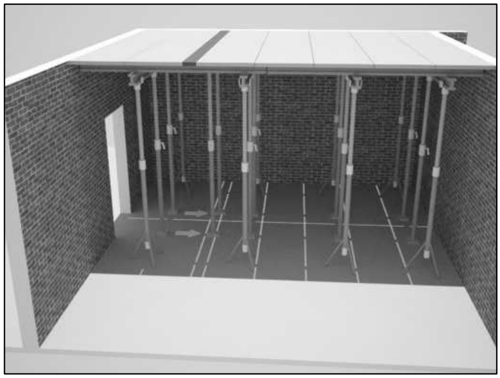
Figure 7.46 Illustration of a 4-day Cycle for Aluminum Formwork.

7.6 ACHIEVING ECONOMY IN SLAB CONSTRUCTION

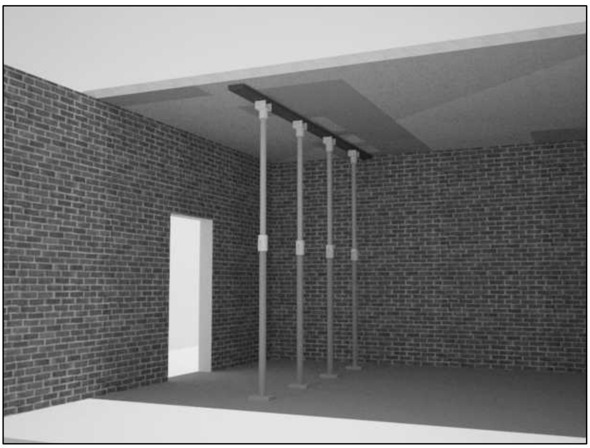
Sometimes, a small strip of plywood is inserted between the full sizes of the plywood sheets at the designated re-propping location. This is shown in the sequence 1 of Fig. 7.47. Props are placed below this re-propping strip shown in the sequence 2. Finally, after the concrete is set, the formwork materials are deshuttered leaving only the props under the re-propping strip. This is shown in the sequence 3 wherein the view of the slab after deshuttering is shown with the props left under the re-propping strip.



Sequence 1: Re-propping strip in position



Sequence 2: Props placed below the re-propping strip



Sequence 3: Props placed below the re-propping strip

Figure 7.47 Plywood Strip Fixing Arrangement.

7.7 DESIGN OF SLAB AND BEAM FORMWORK

The load flow in the slab and the beam formwork is quite straightforward. The load flows through the sheathing to the joists and then to the stringer. The stringer transfers the load to the shores which in turn transfer it to the supporting stratum below (ground or road or another slab). The steps shown in Fig. 7.48 are performed for the design of the slab and the beam formwork.

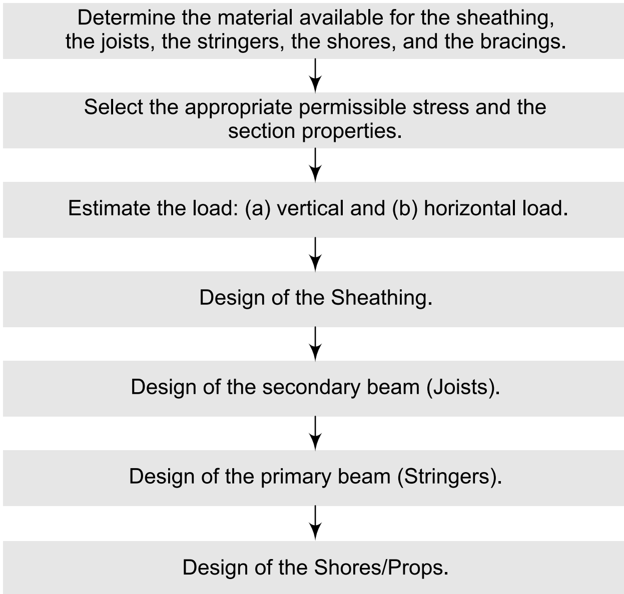


Figure 7.48 Steps in the Slab Formwork Design.

The commonly used sheathing materials in case of the slab and the beam formwork are (1) timber sheathing, (2) plywood sheathing, and (3) steel plates or floor forms.

7.7.1 The Timber Sheathing

The joint spacing of the timber sheathing is determined from Table 7.1 for the various thicknesses of the timber sheathing for the different concrete slab thicknesses and the design loads. The spacing is based on a permissible deflection of span/360. The deflection and not the bending stress govern the design in all these cases. Where bending stress governs, the maximum permissible stress in bending for outdoor locations is considered as 6.0 N/mm^2 for Grade 2 timber of Group C classification as per IS: 3629–1986.

The design load considered here includes the dead and live loads. As can be seen from Table 7.1, a minimum load of 7.5 kN/m^2 has been considered. The timber sheathing is assumed to belong to Group C as per IS: 3629–1986 with E = 5,600 N/mm^2. All spans are considered to be simply supported.

Table 7.1 Joint Spacing of Timber Sheathings (mm) for Different Slab Thicknesses and Design Loads

Concrete slab thickness (mm)	Design load (kN/m^2)	Timber sheathing thickness					
		25 mm	30 mm	35 mm	40 mm	45 mm	50 mm
200	7.50	60	71	82.5	94	106	120
225	8.13	57.5	69	80	92	103	115
250	8.75	56	67	78	90	101	112
275	9.38	55	66	77	88	99	110
300	10.00	54	64	75	86	96	107

7.8 ILLUSTRATION OF SLAB AND BEAM FORMWORK DESIGN

For the following data, design the formwork for the slab and the beam.

(a) Slab thickness = 150 mm, (b) 12 mm plywood as the sheathing, 100 mm × 100 mm timber beams as the secondary and the primary beams are available, (c) floor props CT as the staging. The section property of 100 mm × 100 mm timber is given below.

Solution:

7.8.1 Input Data

Section properties of 100 mm × 100 mm timber

1. $B = 100$ mm
2. $D = 100$ mm
3. Area $A = B \times D = 10{,}000$ mm^2
4. $I = \dfrac{100 \times 100^3}{12} = 83{,}33{,}300$ mm^4
5. $Z_{xx} = \dfrac{I}{D/2} = 1{,}66{,}670$ mm^3
6. $E = 7{,}700$ N/mm^2

Allowable stresses

$$\begin{aligned}\text{Allowable bending stress, } f_b &= 7.0 \text{ N/mm}^2\\ \text{Allowable shear stress, } f_q &= 0.60 \text{ N/mm}^2\\ \text{Bending moment capacity} &= f_b \times Z_{xx} = 1.167 \text{ kNm}\\ \text{Shear force capacity} &= f_q \times A = 6 \text{ kN}\\ EI &= 7{,}700 \times 83{,}33{,}300 = 64.16 \text{ kNm}^2\end{aligned}$$

7.8.2 Computation of Loads

$$\begin{aligned}\text{Self–weight of concrete slab} &= 0.15 \times 26 = 3.9 \text{ kN/m}^2\\ \text{Imposed load} &= 1.5 \text{ kN/m}^2\\ \text{Load of formwork} &= 0.3 \text{ kN/m}^2\\ \text{Total load on formwork} &= \text{sum of above loads } w = 5.7 \text{ kN/m}^2\end{aligned}$$

7.8.3 Sheathing Design

For designing the sheathing, the end span would be providing the critical condition compared to the intermediate spans (see Fig 7.49). For the end span, the end condition is assumed that of a propped cantilever. Let l_p be the effective span of the sheathing. The effective span here is taken as *clear span* + t, where t is the thickness of the plywood. For design and analysis, 1 meter width of the plywood is assumed.

Span based on bending moment

$$M = \frac{w \times l_p^2}{8} \le 0.2$$

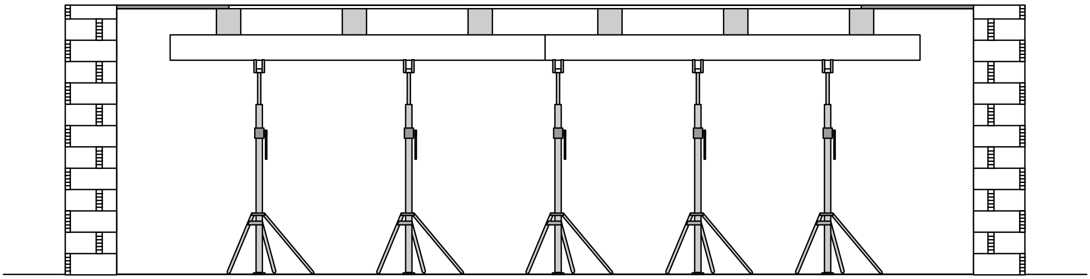


Figure 7.49 Schematic sketch of slab formwork

$$M = \frac{5.7 \times l_p^2}{8} \le 0.2$$

$$\Rightarrow \qquad l_p \le \sqrt{\frac{8 \times 0.2}{5.7}}$$

$$\Rightarrow \qquad l_p \le 0.529 \text{ m} = 529 \text{ mm}$$

Span based on shear force

$$Q = \frac{5 \times w \times l_p}{8} \le 6.16$$

$$Q = \frac{5 \times 5.7 \times l_p}{8} \le 6.16$$

$$\Rightarrow \qquad l_p \le \frac{8 \times 6.16}{5 \times 5.7}$$

$$\Rightarrow \qquad l_p \le 1.73 \text{ m} = 1{,}730 \text{ mm}$$

Span based on shear force

$$\delta = \frac{w \times l_p^4}{185 \times E \times I} \le \frac{l_p}{360}$$

$$\Rightarrow \qquad l_p \le \sqrt[3]{\frac{185 \times E \times I}{360 \times w}} \Rightarrow l_p \le \sqrt[3]{\frac{185 \times 1.07}{360 \times 5.7}}$$

$$\Rightarrow \qquad l_p \le 0.458 \text{ m} = \text{say } 460 \text{ mm}$$

Minimum of the above three values = 460 mm.

Therefore, the effective span = 460 mm.

Using 100 mm × 100 mm timber beams, the center to center span will be = *effective span* – 12 + 100 = 460 – 12 + 100 = 548 mm, say 540 mm.

7.8.4 Design of Secondary Beams

Simply supported end condition is assumed.

Spacing = 540 mm

Width of loading = 540 mm

w_{sb} = loading intensity on the secondary beam = 0.54 × 5.7 = 3.08 kN/m

Span based on bending moment

$$M = \frac{w_{sb} \times l_{sb}^2}{8} \leq 1.167 \text{ kNm, where } l_{sb} \text{ is the span of the secondary beam.}$$

$$\Rightarrow \quad l_{sb} \leq \sqrt{\frac{8 \times 1.167}{3.08}}$$

$$\Rightarrow \quad l_{sb} \leq 1.74 \text{ m} = 1{,}740 \text{ mm}$$

Span based on shear force

$$Q = \frac{w_{sb} \times l_{sb}}{2} \leq 6 \frac{\text{kN}}{\text{m}}$$

$$\Rightarrow \quad l_{sb} \leq \frac{2 \times 6}{3.08} = 3.896 \text{ m} = 3{,}896 \text{ mm}$$

Span based on deflection condition

$$\delta = \frac{5 \times w_{sb} \times l_{sb}^4}{384 \times E \times I} \leq \frac{l_{sb}}{360}$$

$$\Rightarrow \quad l_{sb} \leq \sqrt[3]{\frac{384 \times E \times I}{5 \times w_{sb} \times 360}} \quad \Rightarrow l_{sb} \leq \sqrt[3]{\frac{384 \times 64.17}{5 \times 3.08 \times 360}}$$

$$\Rightarrow \quad l_{sb} \leq 1.644 \text{ m} = \text{say } 1{,}640 \text{ mm}$$

Minimum of the above values = 1,640 mm.

Therefore, the maximum span of the secondary beam = 1,640 mm, say 1,600 mm.

7.8.5 Design of Primary Beams

Simply supported end condition is assumed.

Spacing = 1,600 mm

Width of loading = 1,600 mm = 1.6 m

w_{pb} = loading intensity on the primary beam = 1.6 × 5.7 = 9.12 kN/m

Span based on bending moment

$$M = \frac{w_{pb} \times l_{pb}^2}{8} \leq 1.167 \text{ kNm, where } l_{pb} \text{ is the span of the primary beam.}$$

$$\Rightarrow \quad l_{pb} \leq \sqrt{\frac{8 \times 1.167}{9.12}} = 1.012 \text{ m} = 1{,}012 \text{ mm}$$

Span based on shear force

$$Q = \frac{w_{pb} \times l_{pb}}{2} \leq 6 \frac{\text{kN}}{\text{m}}$$

$$\Rightarrow \quad l_{pb} \leq \frac{2 \times 6}{9.12} = 1.315 \text{ m} = 1{,}315 \text{ mm}$$

Span based on deflection

$$\delta = \frac{5 \times w_{pb} \times l_{pb}^4}{384 \times E \times I} \le \frac{l_{pb}}{360}$$

$$\Rightarrow \qquad l_{pb} \le \sqrt[3]{\frac{384 \times E \times I}{5 \times w_{pb} \times 360}} \quad \Rightarrow \quad l_{pb} \le \sqrt[3]{\frac{384 \times 64.17}{5 \times 9.12 \times 360}}$$

$$\Rightarrow \qquad l_{pb} \le 1.145 \text{ m} = 1{,}145 \text{ mm}$$

Minimum of the above values = 1,012 mm.

Therefore, the maximum span of the primary beam = 1,000 mm.

7.8.6 Design of Shores/Props

The total load on the intermediate shores/props = Total load on the formworks (w) × spacing of the primary beams × span of the secondary beams = 5.7 × 1.0 × 1.6 kN = 9.12 kN

Assuming the required prop height as 3,100 mm, the permissible load carrying capacity of the props from the manufacturer's data = 22.5 kN. Thus, provide CT 340 Props at 1,000 mm centre to centre.

7.9 ILLUSTRATION OF PROPRIETARY SLAB FORMWORK DESIGN

Design a slab formwork using 12mm plywood, H-16 beams and CT-Props as the staging for a slab thickness of 150 mm. You can use H- 16 beams both as the secondary and the primary beams.

Solution

7.9.1 Computation of Loads

Calculate the load for 150 mm thick slab.

Self –weight of concrete slab = 0.15 × 26 = 3.9 kN/m^2

Imposed load = 1.5 kN/m^2

Load of formwork = 0.3 kN/m^2

Total load on formwork = sum of above loads = 5.7 kN/m^2

7.9.2 Sheathing Design

For designing the sheathing, the end span would be providing critical the condition compared to the intermediate spans (see Fig 7.50). For the end span, the end condition is assumed that of a propped cantilever. Let l_p be the effective span of the sheathing. The effective span here is taken as *clear span* + t, where t is the thickness of the plywood. For design and analysis, 1 meter width of the plywood is assumed.

Span based on bending moment

$$M = \frac{w \times l_p^2}{8} \le 0.2$$

$$M = \frac{5.7 \times l_p^2}{8} \le 0.2$$

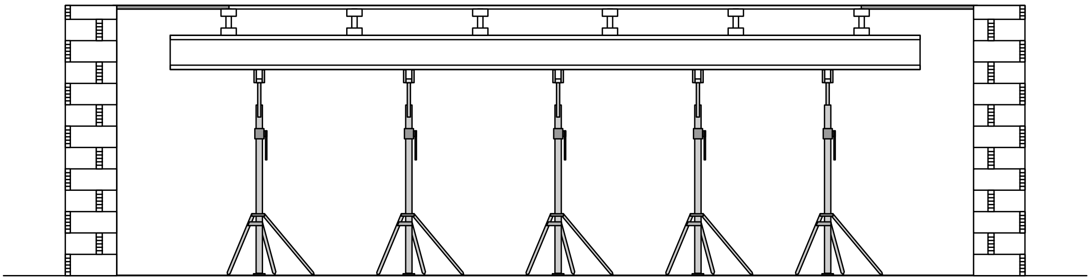


Figure 7.50 Schematic Sketch of Slab Formwork.

$\Rightarrow \qquad l_p \leq \sqrt{\dfrac{8 \times 0.2}{5.7}}$

$\Rightarrow \qquad l_p \leq 0.529 \text{ m} = 529 \text{ mm}$

Span based on shear force

$$Q = \frac{5 \times w \times l_p}{8} \leq 6.16$$

$$Q = \frac{5 \times 5.7 \times l_p}{8} \leq 6.16$$

$\Rightarrow \qquad l_p \leq \dfrac{8 \times 6.16}{5 \times 5.7}$

$\Rightarrow \qquad l_p \leq 1.73 \text{ m} = 1{,}730 \text{ mm}$

Span based on deflection

$$\delta = \frac{w \times l_p^4}{185 \times E \times I} \leq \frac{l_p}{360}$$

$\Rightarrow \qquad l_p \leq \sqrt[3]{\dfrac{185 \times E \times I}{360 \times w}} \quad \Rightarrow \quad l_p \leq \sqrt[3]{\dfrac{185 \times 1.07}{360 \times 5.7}}$

$\Rightarrow \qquad l_p \leq 0.458 \text{ m} = \text{say } 450 \text{ mm}$

Minimum of the above three values = 450 mm.

7.9.3 Design of Secondary Beams

Using H-16 beams, the center to center span will be = *effective span*, 450 – 12 + 65 = 503 mm, say 500 mm. Simply supported end condition is assumed.

Spacing = 500 mm

Width of loading = 500 mm

= loading intensity on the secondary beam = 0.5 × 5.7 = 2.85 kN/m

Span based on bending moment

$$M = \frac{w_{sb} \times l_{sb}^2}{8} \leq 3.0 \text{ kNm, where } l_{sb} \text{ is the span of the secondary beam.}$$

$$\Rightarrow \quad l_{sb} \le \sqrt{\frac{8 \times 3.0}{2.85}}$$

$$\Rightarrow \quad l_{sb} \le 2.902 \text{ m} = 2{,}902 \text{ mm}$$

Span based on shear force

$$Q = \frac{w_{sb} \times l_{sb}}{2} \le 6\frac{\text{kN}}{\text{m}}$$

$$\Rightarrow \quad l_{sb} \le \frac{2 \times 6}{2.85} = 4.211 \text{ m} = 4{,}211 \text{ mm}$$

Span based on deflection

$$\delta = \frac{5 \times w_{sb} \times l_{sb}^4}{384 \times E \times I} \le \frac{l_{sb}}{360}$$

$$\Rightarrow \quad l_{sb} \le \sqrt[3]{\frac{384 \times E \times I}{5 \times w_{sb} \times 360}} \quad \Rightarrow \quad l_{sb} \le \sqrt[3]{\frac{384 \times 145}{5 \times 2.85 \times 360}}$$

$$\Rightarrow \quad l_{sb} \le 2.214 \text{ m} = 2{,}214 \text{ mm}$$

Minimum of the above values = 2,214 mm.

Therefore, the maximum span of the secondary H-16 = 2,214 mm, say 2,000 mm.

7.9.4 Design of Primary Beams

Simply supported end condition is assumed.

Spacing = 2,000 mm

Width of loading = 2,000 mm = 2.0 m

w_{pb} = loading intensity on the primary beam = 2.0 × 5.7 = 11.4 kN /m

Span based on bending moment

$$M = \frac{w_{pb} \times l_{pb}^2}{8} \le 3.0 \text{ kNm},$$ where l_{pb} is the span of the primary beam.

$$\Rightarrow \quad l_{pb} \le \sqrt{\frac{8 \times 3.0}{11.4}} = 1.451 \text{ m} = 1{,}451 \text{ mm}$$

Span based on shear force

$$Q = \frac{w_{pb} \times l_{pb}}{2} \le 6\frac{\text{kN}}{\text{m}}$$

$$\Rightarrow \quad l_{pb} \le \frac{2 \times 6}{11.4} = 1.053 \text{ m} = 1{,}053 \text{ mm}$$

Span based on deflection

$$\delta = \frac{5 \times w_{pb} \times l_{pb}^4}{384 \times E \times I} \le \frac{l_{pb}}{360}$$

$$\Rightarrow \quad l_{pb} \le \sqrt[3]{\frac{384 \times E \times I}{5 \times w_{pb} \times 360}} \quad \Rightarrow \quad l_{pb} \le \sqrt[3]{\frac{384 \times 145}{5 \times 11.4 \times 360}}$$

$$\Rightarrow \qquad l_{pb} \leq 1.395 \text{ m} = 1{,}395 \text{ mm}$$

Minimum of the above values = 1.053 m say 1,050 mm.

Therefore, the maximum span of the primary beam = 1,050 mm.

7.10 ANOTHER ILLUSTRATION OF SLAB FORMWORK DESIGN

Thickness of decking is 20 mm, concrete slab = 150 mm, live load = 3.5 kN/m^2 with 25% additional for impact, and allowable stress in the deck bending = 12 N/mm^2, shear = 0.5 N/mm^2, E = 10,000 N/mm^2, it is decided to use 75 mm × 100 mm batten as the joist. What is the spacing of the joist and what shall be the spacing of the stringer, clear span of the slab is 3.5 m. Maximum permissible bending stress on the timber is 8 N/mm^2, shear = 0.5 N/mm^2, E = 10,000 N/mm^2. Use 100 mm × 150 mm batten as the stringer. Compressive strength normal to the grain = 2.75N/mm^2. Compression parallel to the grain is 11 N/mm^2, E = 4,000 N/mm^2. Permissible deflection in the sheathing is 1.6 mm.

Solution

7.10.1 Computation of Loads

The proposed arrangement of the sheathing, the joists, the stringers, and the shores is shown in Figs 7.51 and 7.52. Calculate the load for the 150 mm thick slab.

Self –weight of concrete slab = 25 × 0.15 = 3.75 kN/m^2

Live load = 3.5 kN/m^2

Additional load for impact = 25% of live load (given) = 0.25 × 3.5= 0.875 kN/m^2

Load of formwork = 0.70 kN/m^2

Total load on formwork, w = 8.825 kN/m^2

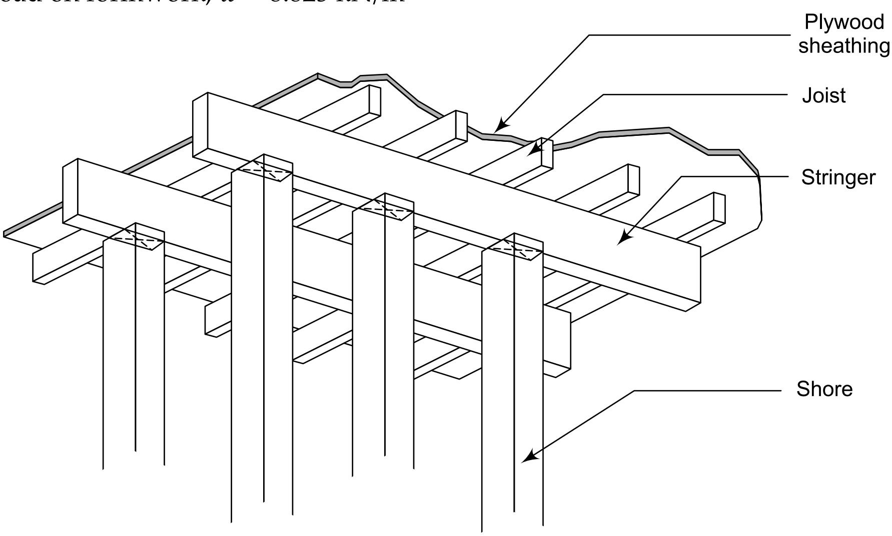


Fig. 7.51 Span Lengths and Components of the Slab.

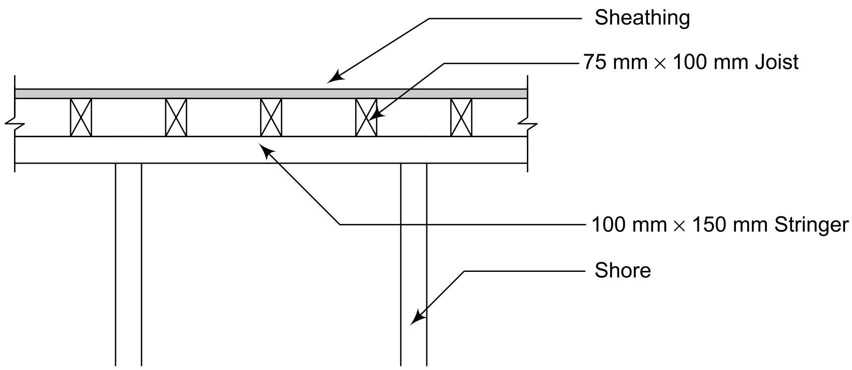


Fig. 7.52 Slab Form.

7.10.2 Design of Sheathing

End condition is assumed to be that of a continuous beam. Let 'l_p' be the effective span of the plywood sheathing. For analysis and design, 1 meter width of the plywood is assumed.

Span based on bending moment

$$M = \frac{w \times l_p^2}{10}$$

$$M = \sigma_b \times z = (12 \times 1{,}000 \times 20 \times 20)/6 = 800{,}000 \text{ Nmm} = 0.8 \text{ kNm}$$

$$M = \frac{8.825 \times l_p^2}{10}$$

$$\Rightarrow \quad l_p \le \sqrt{\frac{10 \times 0.8}{8.825}}$$

$$\Rightarrow \quad l_p \le 0.952 \text{ m} = 950 \text{ mm}$$

Span based on shear force

$$V = \frac{5 \times w \times l_p}{8}$$

$$\tau = \frac{V \times Q}{I \times b}$$

$$\Rightarrow \quad V = \frac{\tau\, I\, b}{Q}$$

$$\Rightarrow \quad V = \frac{0.5 \times \left(\dfrac{1{,}000 \times 20^3}{12}\right) \times 1{,}000}{1{,}000 \times 20^2/8} = 6.67 \text{ kN}$$

Thus, $$V = \frac{5 \times w \times l_p}{8} = 6.67 \text{ kN}$$

$$l_p = \frac{8V}{5w} = \frac{8 \times 6.67}{5 \times 8.825} = 1.209 \text{ m} = 1{,}209 \text{ mm}$$

Span based on deflection

$$\delta = \frac{w \times l_p^4}{145 \times E \times I} \leq 1.6 \text{ mm}$$

$$\Rightarrow \quad \delta = \frac{w \times l_p^4}{145 \times E \times I} \leq \frac{1.6}{1,000}$$

$$E = 10,000 \text{ N/mm}^2$$

$$I = 1,000 \times 20^3/12 \text{ mm}^4$$

$$EI = 6.67 \text{ kNm}^2$$

$$\Rightarrow \quad l_p = \sqrt[4]{\frac{145 \times 1.6 \times E \times I}{1,000 \times w}} \quad \Rightarrow \quad l_p = \sqrt[4]{\frac{145 \times 1.6 \times 6.67}{1,000 \times 8.825}}$$

$$\Rightarrow \quad l_p = 0.647 \text{ m} = \text{say } 650 \text{ mm}$$

Minimum of the above three values = Min (0.95 m, 1.209 m, 0.65 m) = 0.65 m.

Therefore, the span of the sheathing = 650 mm.

7.10.3 Design of Joist

Continuous end conditions are assumed.

Using 75 mm × 100 mm timber beams at the center to center spacing of 650 mm,

Spacing = 650 mm

Width of loading = 650 mm

= loading intensity on one joist = 0.65 × 8.825 = 5.74 kN/m

Span based on bending moment

$$M = \frac{w_j \times l_j^2}{10} = 1.0 \text{ kNm, where } l_j \text{ is the span of the joists.}$$

Also, $M = \sigma_b \times Z$

Given, $\sigma_b = 8 \text{ N/mm}^2$

$$Z = \frac{75 \times 100^2}{6} = 1,25,000 \text{ mm}^3$$

Thus, $M = 8 \times 1,25,000 \text{ Nmm} = 1 \text{ kNm}$

$$\Rightarrow \quad l_j = \sqrt{\frac{1 \times 10}{5.74}}$$

$$\Rightarrow \quad l_j = 1.32 \text{ m} = 1,320 \text{ mm}$$

Span based on shear force

$$\tau = \frac{VQ}{Ib}$$

$$\Rightarrow \quad V = \frac{\tau Ib}{Q}, \; I = 75 \times 100^3/12 \text{ mm}^4$$

$$\Rightarrow \qquad V = \frac{0.5 \times 75 \times 100^3 \times 75 \times 8}{12 \times 75 \times 100^2} = 2{,}500 \text{ N} = 2.5 \text{ kN}$$

Also, $$V = \frac{5 \times w_j \times l_j}{8} = 2.5 \text{ kN}$$

$$\Rightarrow \qquad l_j = \frac{8 \times 2.5}{5 \times 5.74} = 0.696 \text{ m} = 696 \text{ mm}$$

Span based on deflection

Permissible deflection = 3 mm = 3/1,000 m = 0.003 m,

$$E = 10{,}000 \text{ N/mm}^2$$

$$I = 75 \times 100^3/12 \text{ mm}^4$$

$$EI = 62.5 \text{ kNm}^2$$

$$\delta = \frac{1 \times w_j \times l_j^4}{145 \times E \times I} = 0.003$$

$$\Rightarrow \qquad l_j \le \sqrt[4]{\frac{145 \times E \times I \times 0.003}{1 \times w_j}} \quad \Rightarrow \quad l_j = \sqrt[4]{\frac{145 \times 62.5 \times 0.003}{1 \times 5.74}}$$

$$\Rightarrow \qquad l_j = 1.475 \text{ m} = \text{say } 1{,}475 \text{ mm}$$

Minimum of the above values = Min (1,320 mm, 696 mm, 1,475 mm) = 696 mm.

7.10.4 Design of Stringer

Assumed cross section of the stringer is 100 mm × 150 mm.

$$E = 10{,}000 \text{ N/mm}^2$$

$$I = \frac{100 \times 150^3}{12} = 28.125 \times 10^6 \text{ mm}^4$$

$$EI = 281.25 \text{ kNm}^2$$

$$w_{st} = \text{Loading intensity on the stringer} = 8.825 \times 0.696 = 6.14 \text{ kN/m}$$

Span based on bending moment

$$E = 10{,}000 \text{ N/mm}^2$$

$$I = 28.125 \times 10^6 \text{ mm}^4$$

$$Z = \frac{100 \times 150^2}{6} = 375 \times 10^3 \text{ mm}^3$$

$$M = \sigma_b \times Z$$

$$M = 8 \times 375 \times 10^3 = 3 \times 10^6 \text{ Nmm} = 3 \text{ kNm}$$

$$M = \frac{w_{st} \times l_{st}^2}{10} = 3.0 \text{ kNm}$$

$$\Rightarrow \qquad l_{st} = \sqrt{\frac{3 \times 10}{6.14}}$$

$$\Rightarrow \qquad l_{st} = 2.210 \text{ m} = 2{,}210 \text{ mm}$$

Span based on shear force

$$\tau = \frac{V \times Q}{I \times b}$$

$$\Rightarrow \qquad V = \frac{\tau\, I\, b}{Q}$$

$$\Rightarrow \qquad V = \frac{0.5 \times 100 \times 150^3 \times 100 \times 8}{12 \times 100 \times 150^2} = 5{,}000 \text{ N} = 5.0 \text{ kN}$$

Also,
$$V = \frac{5w_{st} \times l_{st}}{8} = 5 \text{ kN}$$

$$\Rightarrow \qquad l_{st} = \frac{5 \times 8}{5 \times 6.14}$$

$$\Rightarrow \qquad l_{st} = 1.303 \text{ m} = 1{,}303 \text{ mm}$$

Span based on deflection

Permissible deflection = 3 mm = 3/1,000 m = 0.003 m

$$\delta = \frac{1 \times w_{st} \times l_{st}^4}{145 \times E \times I} \leq 0.003$$

$$\Rightarrow \qquad l_{st} \leq \sqrt[4]{\frac{145 \times E \times I \times 0.003}{1 \times w_{st}}} \qquad \Rightarrow \qquad l_{st} \leq \sqrt[4]{\frac{145 \times 281.25 \times 0.003}{1 \times 6.14}}$$

$$\Rightarrow \qquad l_{st} = 2.113 \text{ m} = \text{say } 2{,}113 \text{ mm}$$

Minimum of the above values = Min (2,210 mm, 1,303 mm, 2,113 mm) = 1,303 mm.

7.10.5 Design of Shores

The cross section of the timber shores assumed is 100 mm × 100 mm.

Shores are compression members. Depending on the slenderness ratio, the shores can be designed as short, intermediate, or long timber columns.

The slenderness ratio is defined as below:

$$\text{Slenderness ratio } \lambda = \frac{l_{\text{unsupported}}}{\text{Least dimension}}$$

$$l_{\text{unsupported}} = 3.5 - 0.150 - 0.020 - 0.100 - 0.150 = 3.080 \text{ m}$$

$$\text{Least dimension} = 100 \text{ mm} = 0.100 \text{ m}$$

$$\lambda = \frac{3.080}{0.100} = 30.8 \leq 50$$

$$K_8 = 0.702 \times \sqrt{\frac{E}{f_{c.p.}}}$$

$$f_{c.p.} = \text{compression parallel to the grains} = 11 \text{ N/mm}^2$$

$$E = 4{,}000 \text{ N/mm}^2.$$

$$K_8 = 0.702 \times \sqrt{\frac{4{,}000}{11}} = 13.38$$

Since $\lambda \geq 13.38$, the shores shall be designed as a long column.

Permissible compressive stress for a long column is given by the following expression (Refer IS: 883–1994):

$$f_c = \frac{0.329 \times E}{(\lambda)^2}$$

$$f_c = \frac{0.329 \times 4,000}{(30.82)^2}$$

$$f_c = 1.39 \text{ N/mm}^2$$

Thus, the permissible load on the shores = f_c × *Net area of the cross section* = 1.39 × 100 × 100 = 13.9 kN.

Actual load on the shore = *load intensity w* × *area covered by one shore,*

⇒ Actual load on the shore = 8.825 × 0.696 × 1.3 = 7.985 kN < 13.9 kN.

Hence safe.

Brace all shores.

REVIEW QUESTIONS

Q1. True or False

(a) The main components of a slab and beam formwork are: sheathing, joists, stringers, shores, and bracing.

(b) In traditional slab and beam formwork, emphasis is on all timber and all steel formwork.

(c) Traditional slab and beam formwork is characterized by labor intensive and time consuming operation.

(d) In traditional slab and beam formwork, activity of formwork construction is critical and hence affecting the time, cost, quality, and safety.

(e) Typical components of beam formwork with slab forming are- bevel strip, T-head shore, beam soffit, and plywood sheathing.

(f) Examples of proprietary slab and beam formwork are –L&T slab and beam formwork, PERI slab and beam formwork, Mivan.

(g) With respect to joints- the two important factors needing attention are (i) joints between the posts and the joists; and (ii) framework of the posts through a system of horizontal braces and inclined shores.

(h) The three major L&T slab and beam formwork systems are –(i) L&T flex, (ii) Beam forming support system, and (iii) Heavy duty tower.

(i) PERI solutions for slab and beam formwork are –(i) skydeck aluminum panel ; (ii) gridflex aluminum grid slab (iii) Multiflex girder slab.

(j) Mivan formwork is found to be appropriate in housing projects where large numbers of similar houses are to be constructed in a short duration.

(k) In Mivan formwork, all elements viz., load bearing walls, columns, beams, floor slabs, stairs, and balconies are cast in one continuous pour of concrete resulting in a monolithic structure.

(l) Commonly used sheathing materials in case of slab and beam formwork are - (i) timber sheathing; (ii) plywood sheathing; (iii) steel plates or floor forms.

(m) Formwork construction cycle time is affected by the layout of the floors.

(n) Formwork construction cycle time is affected by the dependence among the various activities.

(o) Formwork construction cycle time is affected by the skill level of the workmen.

(p) Formwork construction cycle time is affected by the permissible striking times.

Q2. Match the following:

(i) Joints between the posts and the joists	(a) Reduce the effects of constructional eccentricity.
(ii) Joints between the beam bottoms and the posts	(b) Contribute to adequate structural stability.

Q3. Match the following:

(i) L&T flex	(a) Useful since shuttering is uneconomical due to the beams having a small section.
(ii) L&T beam forming head	(b) L&T props; tripod; four-way head, and H-16 beams.
(iii) L&T heavy duty tower	(c) > 4.1 m; large spacing, self-supporting, transportation easy.

Q4. Match the following:

(i) Skydeck aluminum panel	(a) Plywood as sheathing, formwork girder, props, crossheads, galvanized props, GT24 and VT20K girder.
(ii) Gridflex aluminum grid slab	(b) Slabs up to thickness of 950 mm and beams of up to 300 mm width and 600 mm depth; drop heads; enhanced safety.
(iii) Multi-flex girder slab formwork	(c) Grid elements of 2,000 mm length and 1,000 mm width; wall support; filler elements of various sizes to suit different requirements in transverse and longitudinal directions.

Q5. Sequence the following in the context of achieving economy in plywood strip fixing arrangement: (i) re-propping strip along with the props at the construction site (ii) re-propping strip in position (iii) props placed below the re-propping strip.

Q6. Sequence the following in the context of slab and beam formwork design: (i) design of the sheathing; (ii) design of the primary beam;(iii) design of the secondary beam (joists); (iv) design of the shores/ props; (v) determine the material available for the sheathing, the joists, the stringer, the shores, and the bracings ; (vi) estimate or the vertical and horizontal load; (vii) select the appropriate permissible stress and the section properties.

Q7. Prepare a summary report for L&T slab and beam formwork and discuss all the major characteristics of the system.

Q8. List out the advantages of Mivan formwork.

Q9. List out and discuss the characteristics of Mivan formwork.

Q10. List out the different construction steps in a Mivan formwork application.

Q11. Prepare a bar chart giving details of the various activities involved in Mivan formwork operation for a 4-day cycle.

Q12. Design the formwork for the slab and beam for the given data: (a) Thickness of the floor: 120mm (b) Center to center spacing of the beam: 3 m (c) Width of the beam: 300 mm and depth 400 mm below the slab (d) Height of the ceiling of the roof: 4m above the floor. Take live load on the sheathing as 4 kN/m^2 and the dead weight of the wet concrete = 26.5 kN/m^3. A 19 mm thick plywood is available for formwork sheathing for which the EI = 2.73 kNm^2/m, and the permissible deflection = 1.5 mm and the permissible bending moment = 0.34 kN m. The permissible bending stress in the timber = 7 N/mm^2.

Chapter

8 Formwork for Special Structures

Contents: Introduction; Shells; Domes; Folded Plates; Overhead Water Tanks; Natural Draft Cooling Tower; Nuclear Reactor; Tunnel; Lift Shaft

8.1 INTRODUCTION

In this chapter we cover a wide range of formworks for special structures. Special structures here mean the kind of structures not used in day-to-day construction. These structures include shells, domes, folded plates, natural draft cooling towers, nuclear reactors, tunnels, and lift shafts, etc. As would be seen, some of these structures are formed as *cast-in-situ* structures while some of them as precast elements. Examples of forms from fields have also been illustrated.

8.2 SHELLS

The shell structures as roofs are required for various applications such as nuclear reactors, indoor stadiums, aircraft hangars and so on. The formwork for these structures requires the design and execution of two distinct components: (1) decking and (2) arrangement of shoring towers, commonly known as centering. The decking member usually consists of timber and plywood. The roof profile is normally expressed in terms of span length (s), height ('h' also called as 'rise') at mid span (centre), and thickness (t) of the roof slab. Sometimes, the radius (R) may also be specified by the designer. However, even if the radius is not given, the same can be found out from the rise specified at the mid span and the span length. For example, consider the shell roof (Fig. 8.1) having a circular profile and having s, t, and h values as 20 m, 200 mm, and 3 m respectively.

The radius can be computed from the following expression:

$$R = \frac{s^2}{8h} + \frac{h}{2} \tag{8.1}$$

Thus, the radius R for the above mentioned values of s, t, and h can be computed as

$$R = \frac{20^2}{8 \times 3} + \frac{3}{2} = 18.17 \text{ m}$$

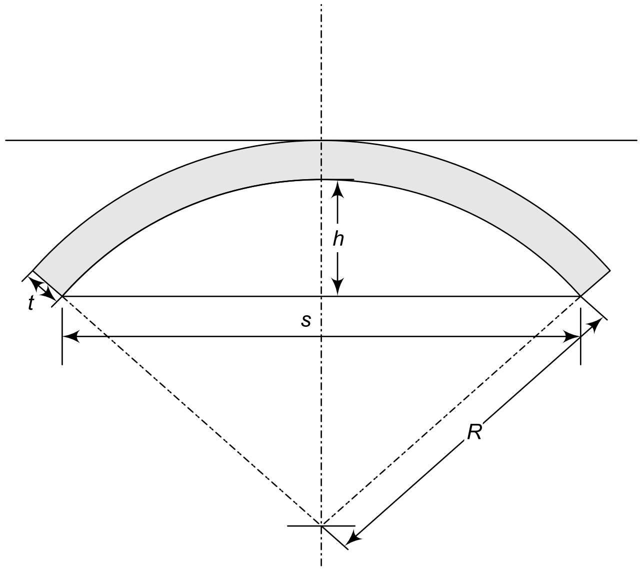


Figure 8.1 Geometry of a Circular Shell Roof.

For forming the deck for the shell roof, it is important to convert the geometry envisaged for the shell roof as closely as possible in reality. To match the exact geometrical profile, it is essential to generate the coordinates of a sufficient number of points on the decking. For example, let's try to locate the points for forming the deck of a shell roof having circular profile given by *XYZ* (Fig. 8.2). Let's assume the s, *h*, and *R* values as 24 m, 4 m, and 20 m respectively, for the new example.

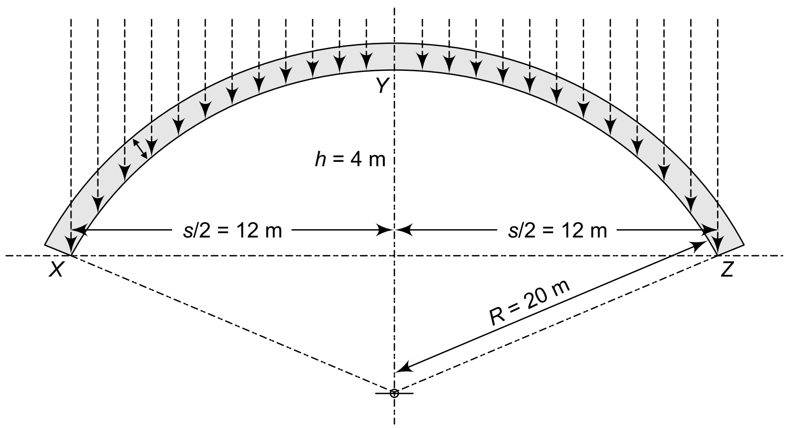


Figure 8.2 Circular Shell Roof-Derivation of Geometric Profile.

Thus, the points *X*, *Y*, and *Z* are the points on the bottom of the shell roof. For the formwork, these *X*, *Y*, and *Z* points are on the top of the deck formwork. Knowing these three points alone will not be enough for the formwork crews to translate the exact geometry in reality. Thus, the coordinates of some more points are needed. Assuming that coordinates at a distance of 1 m along the span length would be sufficient, the following method can be used to derive the coordinates.

An expression for y at different distances from the midpoint is given by

$$y_i = R - \sqrt{R^2 - x_i^2} \tag{8.2}$$

Assuming $x_i = \frac{s}{24}$, Table 8.1 provides the values of y_i which are self explanatory.

Table 8.1 Computation of y_i

Point	Distance from midpoint Y (m)	Expression to compute y_i	The value of y_i (m)
A1	1.0	$y_1 = R - \sqrt{(R^2 - 1.0^2)}$	$y_1 = 0.025$
A2	2.0	$y_2 = R - \sqrt{(R^2 - 2.0^2)}$	$y_2 = 0.100$
A3	3.0	$y_3 = R - \sqrt{(R^2 - 3.0^2)}$	$y_3 = 0.226$
A4	4.0	$y_4 = R - \sqrt{(R^2 - 4.0^2)}$	$y_4 = 0.404$
A5	5.0	$y_5 = R - \sqrt{(R^2 - 5.0^2)}$	$y_5 = 0.635$
A6	6.0	$y_6 = R - \sqrt{(R^2 - 6.0^2)}$	$y_6 = 0.921$
A7	7.0	$y_7 = R - \sqrt{(R^2 - 7.0^2)}$	$y_7 = 1.265$
A8	8.0	$y_8 = R - \sqrt{(R^2 - 8.0^2)}$	$y_8 = 1.669$
A9	9.0	$y_9 = R - \sqrt{(R^2 - 9.0^2)}$	$y_9 = 2.139$
A10	10.0	$y_{10} = R - \sqrt{(R^2 - 10.0^2)}$	$y_{10} = 2.679$
A11	11.0	$y_{11} = R - \sqrt{(R^2 - 11.0^2)}$	$y_{11} = 3.297$
A12	12.0	$y_{12} = R - \sqrt{(R^2 - 12.0^2)}$	$y_{12} = 4.000$

Due to the relatively thin sections of shells, it is very challenging to work within the specified tolerance limits. The complicated nature of the shell geometry also poses challenges in computing the stresses at different points. The removal of formwork for shell structures is again a critical task. All these issues make the construction of shell structures a difficult task. The sequence of removal of formwork for these structures needs careful consideration and should be planned in such a manner that it does not result in the concentration of stresses at any point which is not envisaged by the form designer.

8.2.1 System of Shells for National Spiritual Assembly of Bahai Faith at New Delhi

A complicated system of shells (see Fig. 8.3) was constructed for the National Spiritual Assembly of Bahai faith at New Delhi.

Most of these spherical shells were to be concreted in single pour, and hence the formwork for soffit had to be erected and aligned for full height. The formwork consisted of timber shutter panels shaped to suit the curvature of shells with plywood sheathing bent and fixed on top. The timber shutters were supported by timber joists spanning over structural steel purlins. The staging consisted of a number of space trusses generated in the same way as that for the shell surface.

The back form consisted of curved structural steel soldiers, to which shaped timbers of 230 mm width were fixed in the exact geometry of the top surface. The top shutters were of smaller height for ease of handling, and they were fixed to the soldiers progressively, as the concreting proceeded. The shaped timber ensured that the back forms were fixed to the correct geometry of the top surface. Thorough bolts at large intervals connected the back form to the bottom staging.

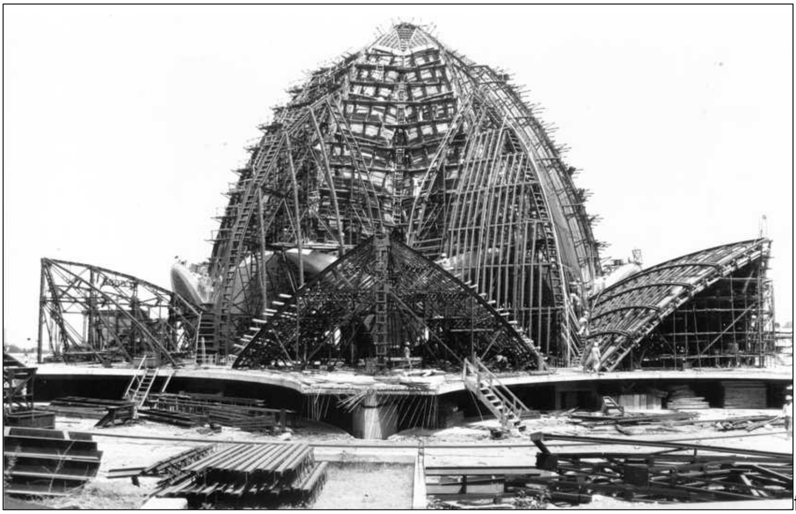


Figure 8.3 Formwork Arrangement for Shells for National Spiritual Assembly of Bahai Faith.

Deflection was an important consideration in the design of the formwork besides the various loads to which the formworks were subjected. The dead load on the formwork, the live load on the formwork, and the wind load on the formwork considered in the design were 0.750 kN/m^2 of surface area, 2 kN/m^2 of plan area, and 1 kN/m^2 respectively. The concrete pressure on the formwork was computed using the ACI formula. For designing the formwork, various combinations of loads were considered and the most adverse loading condition was considered for the design. The maximum deflection in the design was limited to 3 mm per m. The maximum deflection value included the possible errors in fabrication and erection.

In order to avoid the joints, casting of most of the shells was done in a single continuous pour. Also in order to avoid the lateral loads on the shores supporting the formwork, the concreting of shells was done 3 at a time, 120° apart in a symmetrical manner.

The formwork consisted of space frames made up of structural steel with welded joints. Timber joists were used to support the form panel. The slope of the shells was such that it required a top formwork besides the bottom formwork. Each form panel of the top and bottom formwork was fabricated according to the dimension computed from the geometry. The theoretical dimensions were cross checked practically by the measurements from a full scale mock-up erected on the ground. The dimensions were verified at different levels both within the contractor organization and by the client organization. Plywood was used as the sheathing material. The bottom formwork as mentioned, was thus fully fixed from the bottom to the top for every petal, and was aligned accurately. The sheathing joint in each form panel was sealed with putty, and a protective coating was applied over the plywood surface. The outer formwork panels were made to the exact dimensions and their positions were verified in the mock up exercise. These panels were kept ready for installation as and when the need arose. The outer formwork panels were fixed one after the other as the concrete progressed.

The pipe supports were erected at selected points on the inner leaf staging. The selection of the pipe support points was done based on architectural considerations. The pipes were supporting the structural steel grid prepared for the outer surface of the shells. The steel grid in turn was supporting the loads exerted on the outer formwork. The ties connecting the inner and outer forms were also provided at select locations to reduce the load on the steel shoring.

8.3 DOMES

8.3.1 Construction of the Dome Roof of a Reactor Building

The shuttering arrangement used for the construction of the dome roof of a reactor building is shown in Figs. 8.4 and 8.5. The arrangement consisted of radial trusses supported at the bottom, on the circular walls of the reactor, with a central compression ring on top and the ties with the tension ring to take the lateral thrust at the bottom support. The circumferential purlins between the radial trusses supported the timber panels shaped to the required curvature.

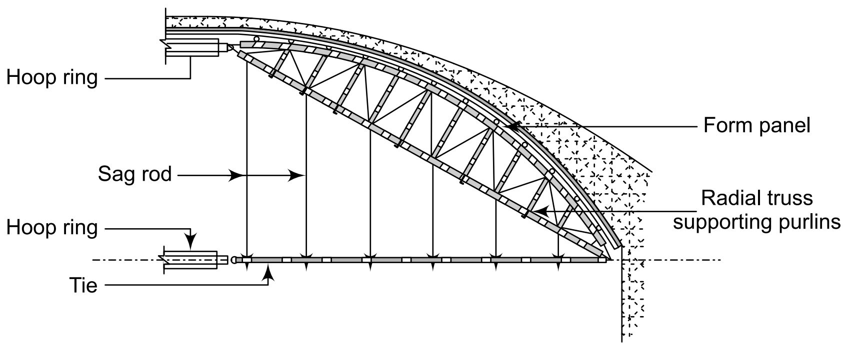


Figure 8.4 Elevation of Dome Formwork.

Jacks were provided for gradual and sequential deshuttering operations. With this arrangement, it was possible to work both on the internals and the roof simultaneously. In cases where there was no constraint, the shuttering could also be supported on conventional staging.

Figures 8.6 and 8.7 show the dome formwork under preparation for the construction of a nuclear reactor building. The dome for the containment is cast on the formwork supported by a framework in structural steel. The framework consists of a number of radial trusses with circumferential stiffener members. The framework is supported on a number of brackets fixed to the containment wall. The formwork supporting structure is first assembled on the ground to avoid any mismatch, and the erection inside the reactor building is carried out later in segments (Fig. 8.6). After erection of the formwork supporting structures, the formwork and the rebar are laid in place (Fig. 8.7).

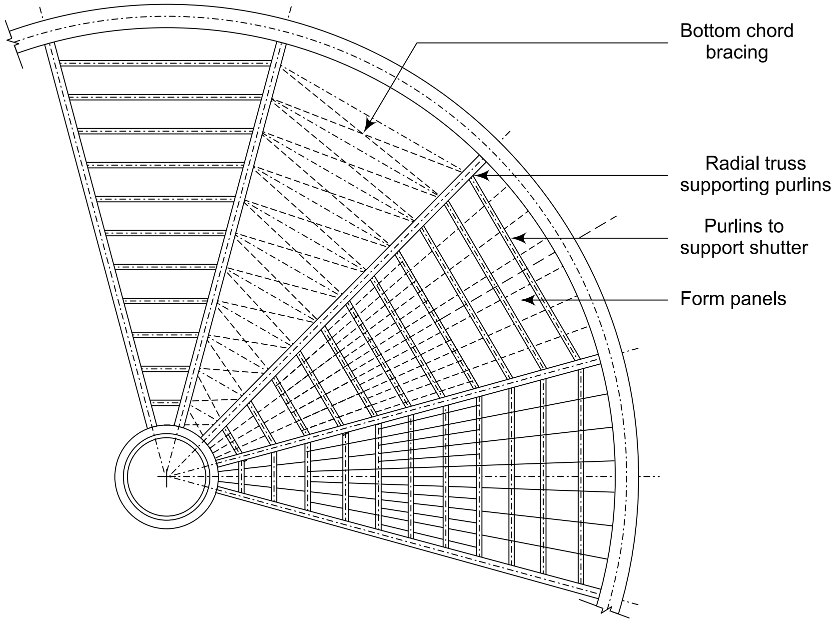


Figure 8.5 Plan of Dome Formwork.

Figure 8.6 Erection of Dome Formwork.

Figure 8.7 Dome Formwork Fixing.

8.3.2 Construction of Central Secretariat Rotary Dome for Delhi Metro Rail Corporation

Rotary dome is a part of the main Central Secretariat Station for the Delhi Metro Rail Corporation MC1B package and is located between Patel Chowk and Rail Bhavan. Radius of the dome is 9.0 m in the vertical plane, the apex height is 3.5 m, and the plan radius is 29.8 m at the ring beam level. The complete formwork arrangement is designed and developed indigenously by the design team of Larsen and Toubro Limited. The total formwork area for the dome is 3,500 m^2 for both the top and bottom shutters.

Initially, the planning and design of the formwork were carried out assuming that normal concrete shall be used. Accordingly, the windows in the top shutter were provided for concreting. The formwork construction turned out to be so good that the client insisted on using self compacted concrete. Thus the plan and design of the formwork was modified to take care of self compacting concrete. The window opening left for normal concreting was closed since it was felt that it would not be needed for self compacting concrete. The loads were transferred to the main frame and it was found that if concrete was poured at 400 mm to 500 mm per hour, there would not be excessive deflection. The formwork crews were present all time during the concreting operation. During concreting, some

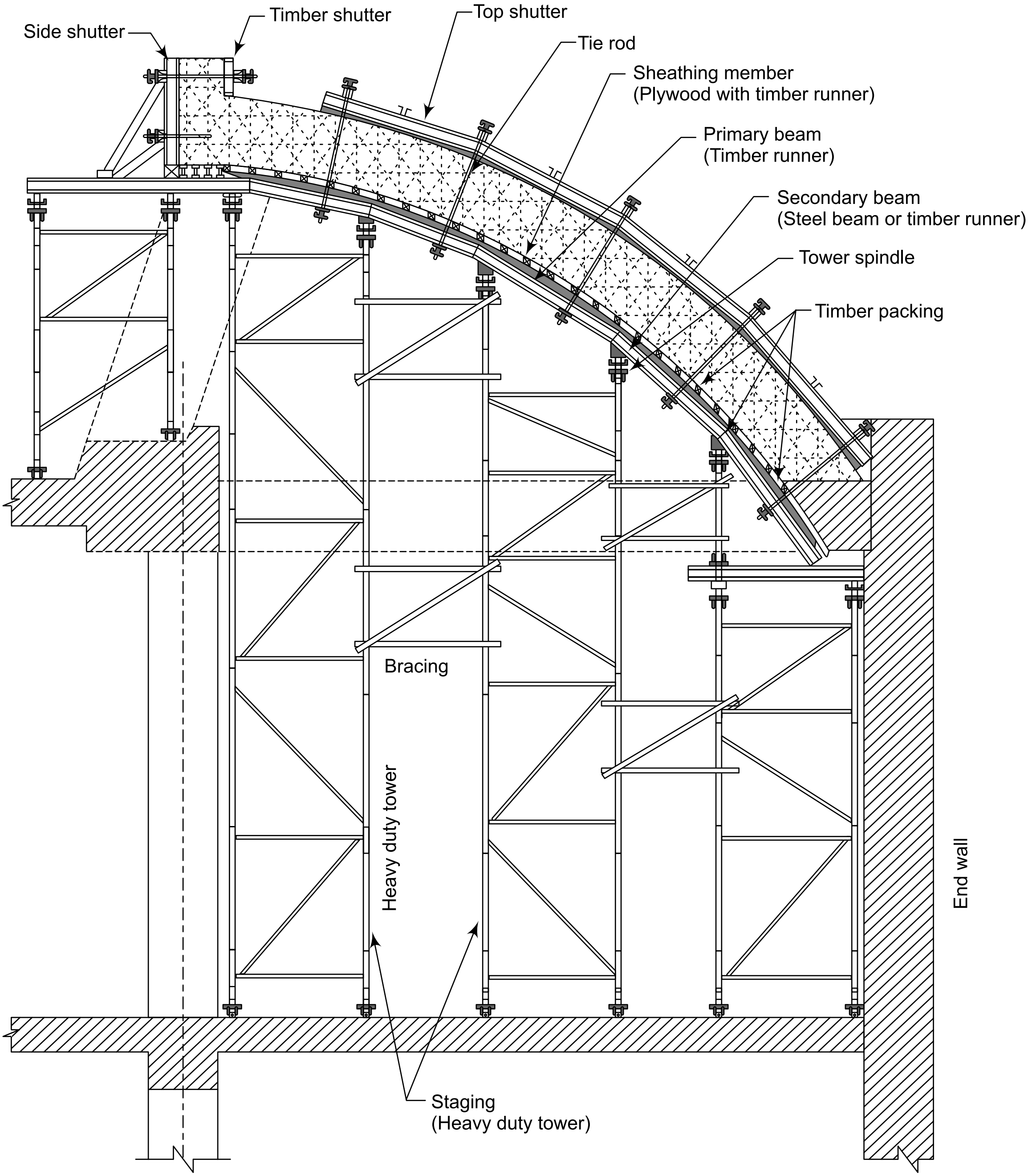


Figure 8.8 Cross Section of Dome.

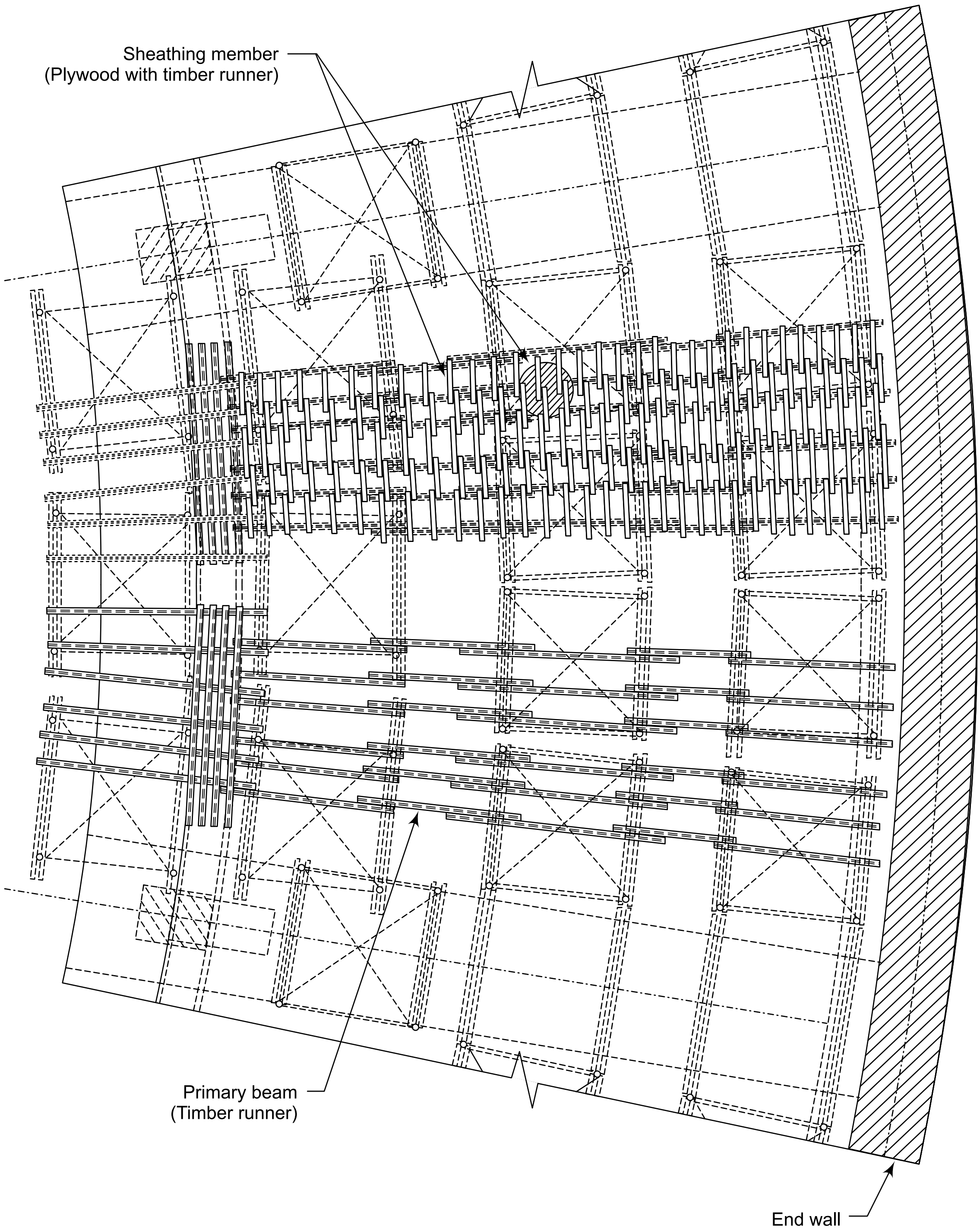


Figure 8.9 Part Plan Showing Bottom Shutter Arrangement.

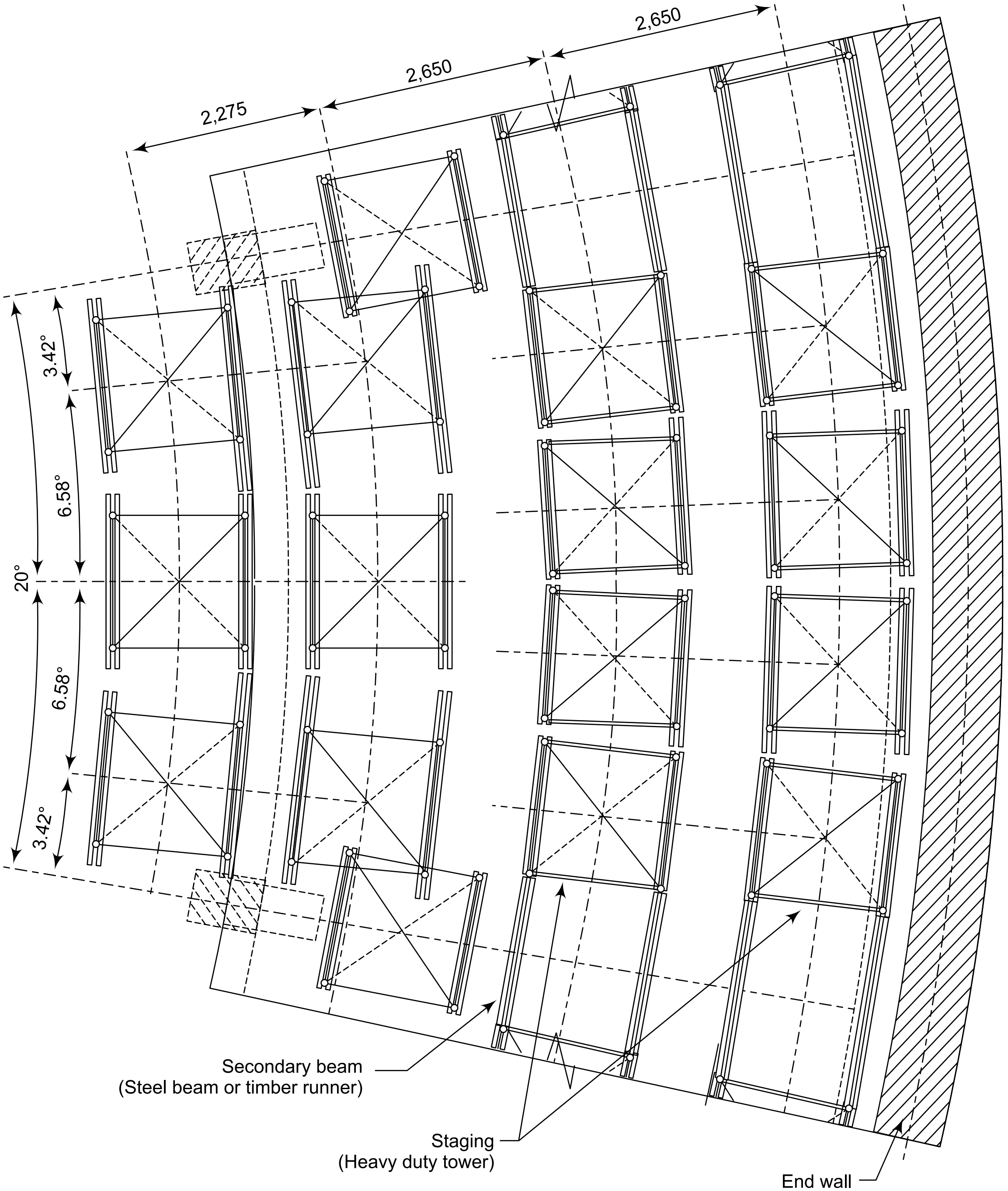


Figure 8.10 Part Plan Showing Shoring Arrangement and Secondary Beam.

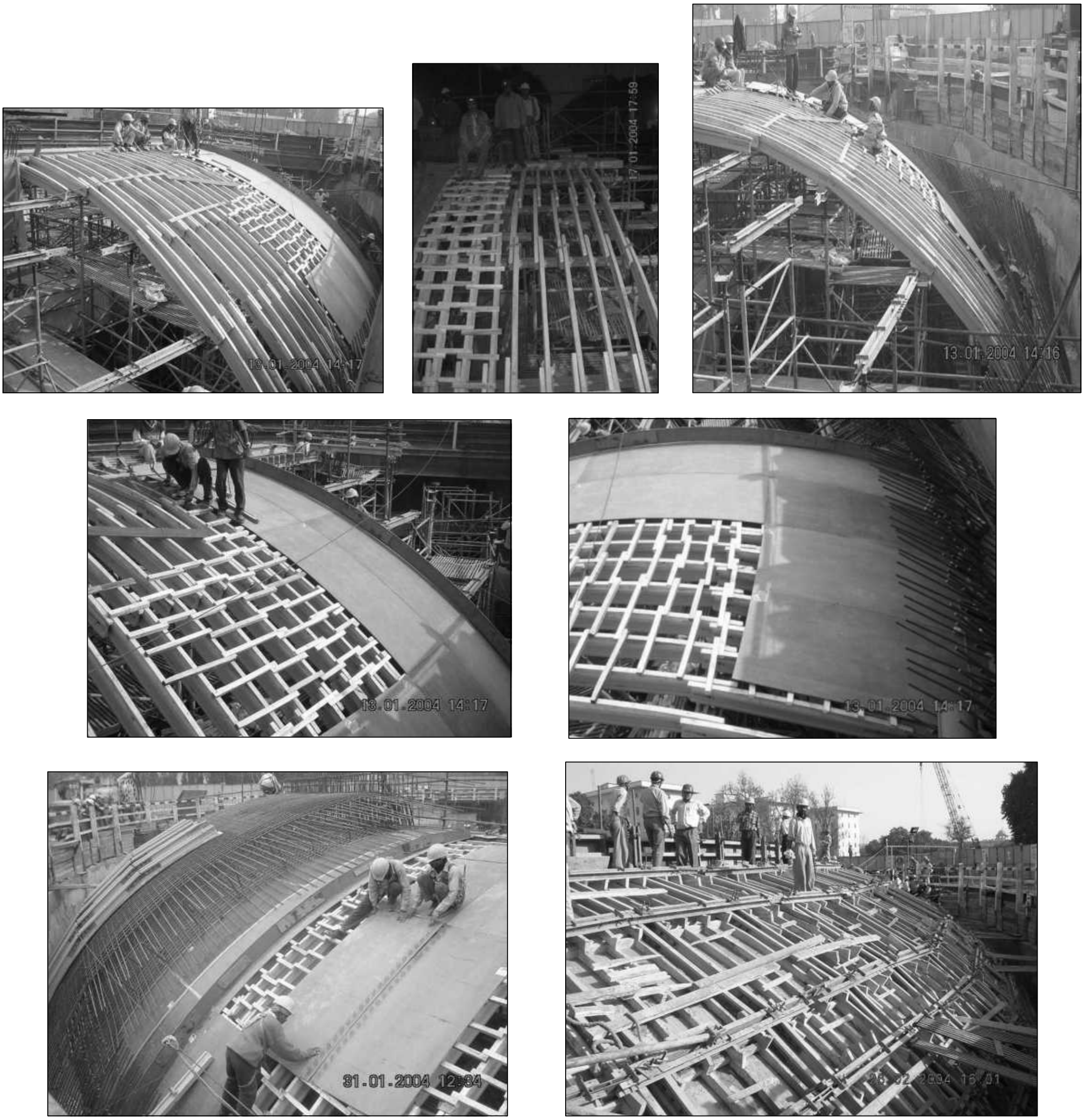


Figure 8.11 Different Views of Dome Formwork Under Construction.

leakages were observed which were immediately addressed. The first pour of 200 m^3 concrete was cast successfully on 27.02.04. This is perhaps the first shell structure with self compacting concrete. No deflection was noticed in the top and bottom shutter. After the removal of the shutters, a very good finish was obtained both at the top and the bottom of slab.

Figure 8.8 shows the cross section of the dome while Figs. 8.9 and 8.10 show part plan showing bottom shutter arrangement and part plan showing shoring arrangement and secondary beam respectively. Figure 8.11 shows different views of dome formwork under construction.

8.3.3 Construction of Elliptical Dome for Delhi Metro

A massive elliptical dome spanning 86 m on the major axis and 66 m on the minor axis was constructed recently for one of the stations for Delhi Metro. The elliptical dome with a plan area of 4,438 m^2 is supported on 44 columns having 5° outward inclination (see Figs. 8.12 and 8.13). The dome shell is mounted on continuously supported radial beams. The radial beams in turn rest on internal ring beams on one end and external columns on the other end. The shoring towers were erected for the entire dome in one go, to crash the construction schedule. Plywood was used as the sheathing material which was supported on the H-16 beams.

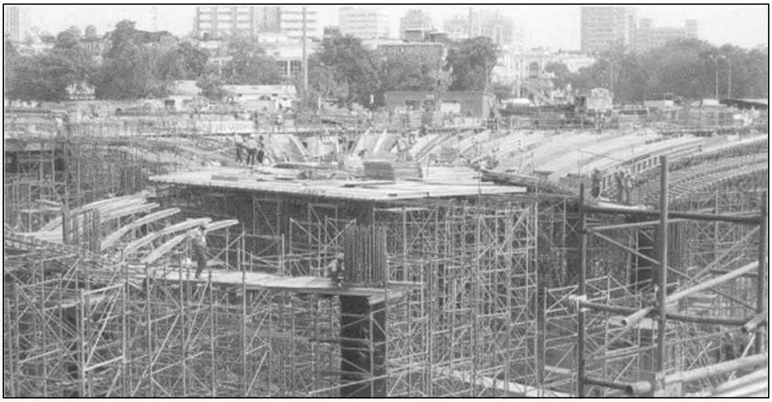


Figure 8.12 View Showing Shoring Towers Supporting the Dome Formwork Under Progress.

Figure 8.13 Completed View of the Dome Shell.

8.4 FOLDED PLATES

An interesting system of formwork was used for an *in-situ* folded plate roof (Fig. 8.14). Each folded plate element was 22 m long and 7.5 m wide. The formwork consisted of structural steel frame work, and timber or plywood shuttering. The entire formwork unit was assembled on the ground and lifted to its casting position with the help of 4 winches and by lifting the four corners of the unit, and supported on trestles 7 days after concreting. The entire assembly was lowered and shifted to its new position.

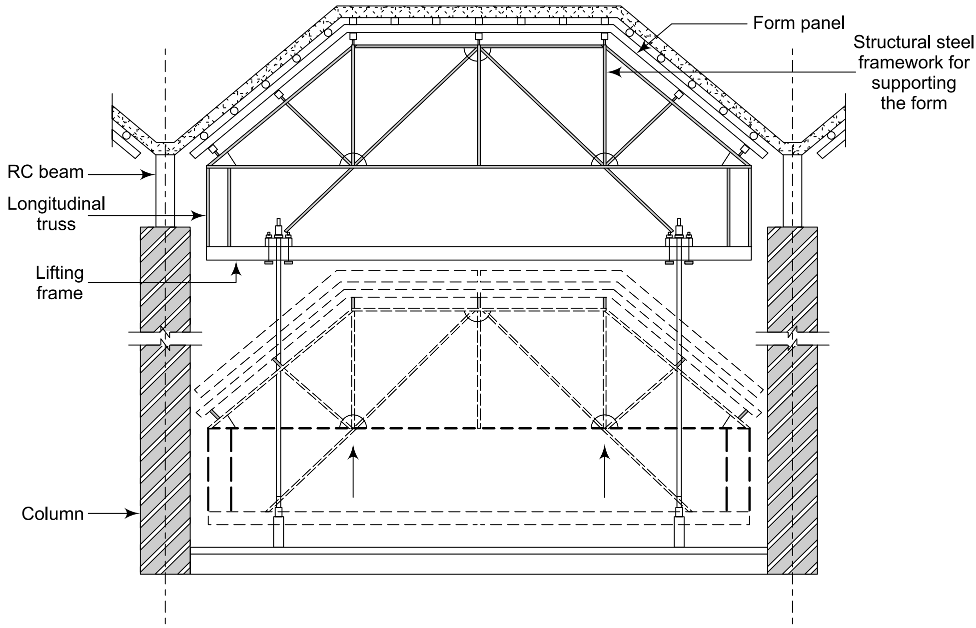


Figure 8.14 Folded Plate Formwork.

8.4.1 *Cast-in-situ* Folded Plates

The arrangement for constructing *cast-in-situ* folded plates for Air Hangar at Mumbai is shown in Figs. 8.15 and 8.16. The arrangement consists of plywood as the sheathing member supported on H-16 beams, which in turn are resting on modified walers. The modified walers are fabricated using standard walers discussed elsewhere in the text. The entire assembly is resting on heavy duty towers.

Figure 8.15 Formwork Using Standard Components of L&T Formwork Systems to Form the Folded Plate Slab of Air Hangar at Mumbai.

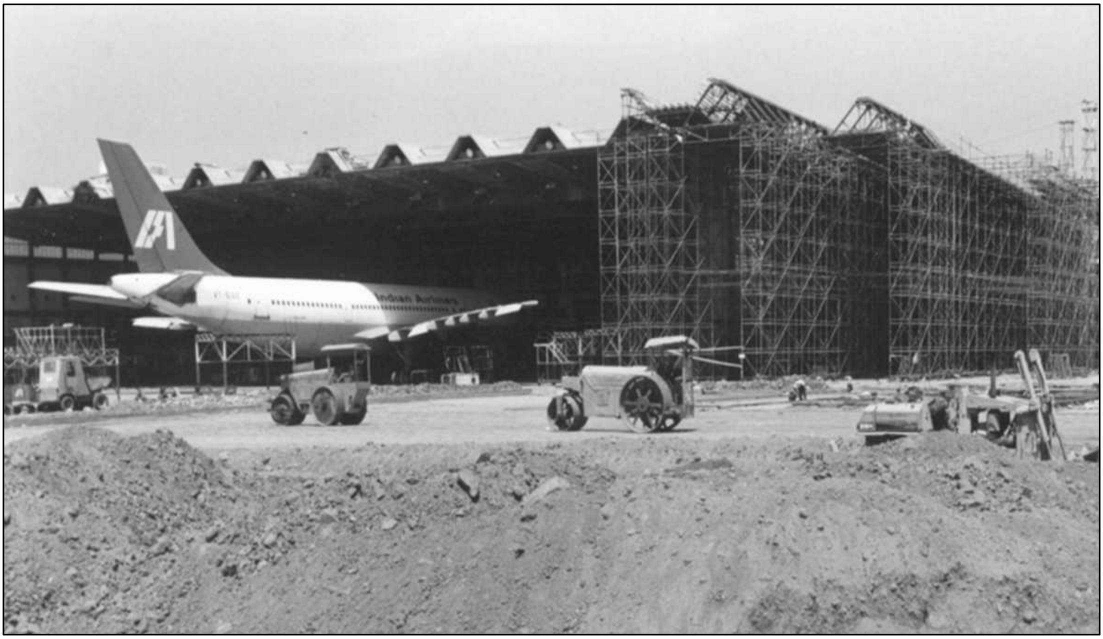


Figure 8.16 Staging and Shuttering in Position for Construction of the Indian Airlines Hangar at Mumbai Project Executed by L&T-ECC Construction Group for IRCON.

8.4.2 Pre-cast Folded Plate

A typical cross section of folded plates is shown in Figs. 8.17 and 8.18, and it can be seen that the precasting of folded plates was carried out in two stages. In the first stage, after cleaning the moulds and tying the reinforcement for two flaps, concrete was cast. Now, these flaps were lifted after 3 hours of concreting, and then aligned as required. In the second stage, the diaphragm and the bottom portion of the folded plates were cast.

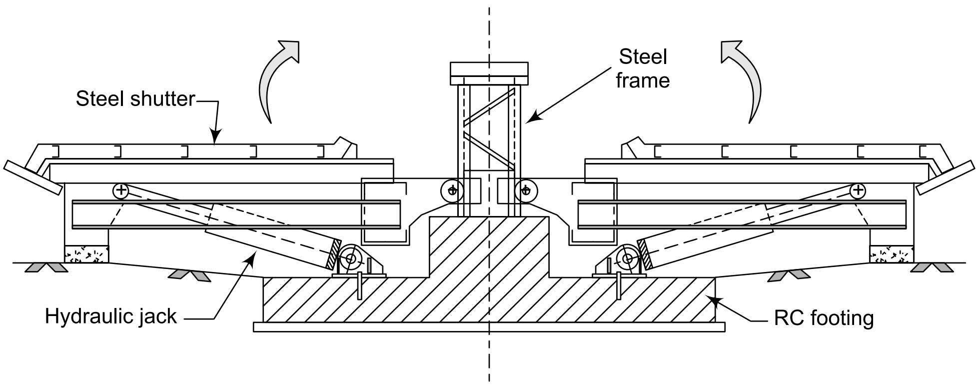


Figure 8.17 Casting of Folded Plates- Stage 1.

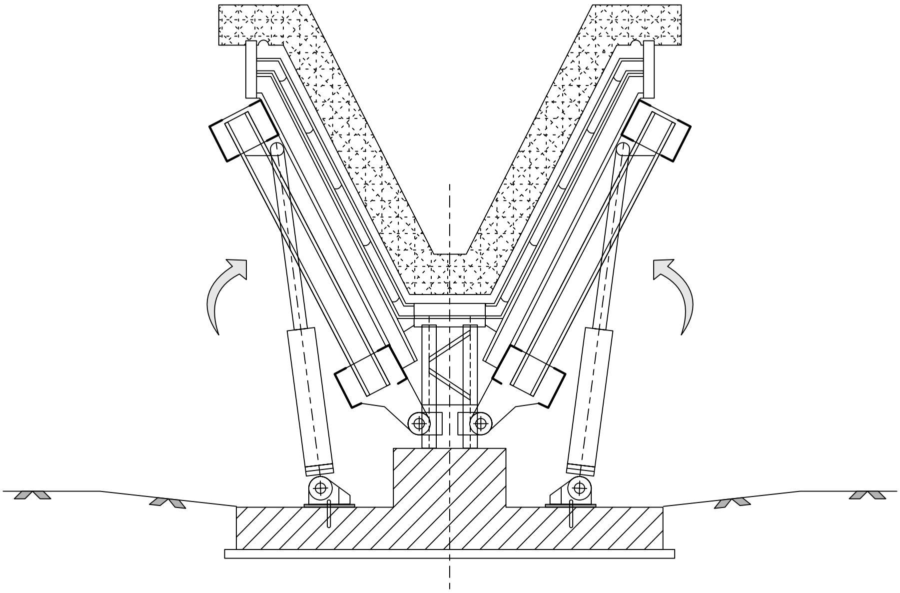


Figure 8.18 Casting of Folded Plates- Stage 2.

8.5 OVERHEAD WATER TANKS

In case of tall towers, the shuttering for the tank portion at the top can be supported on cantilever trusses mounted on the water tank tower structure. A typical arrangement is shown in Fig. 8.19.

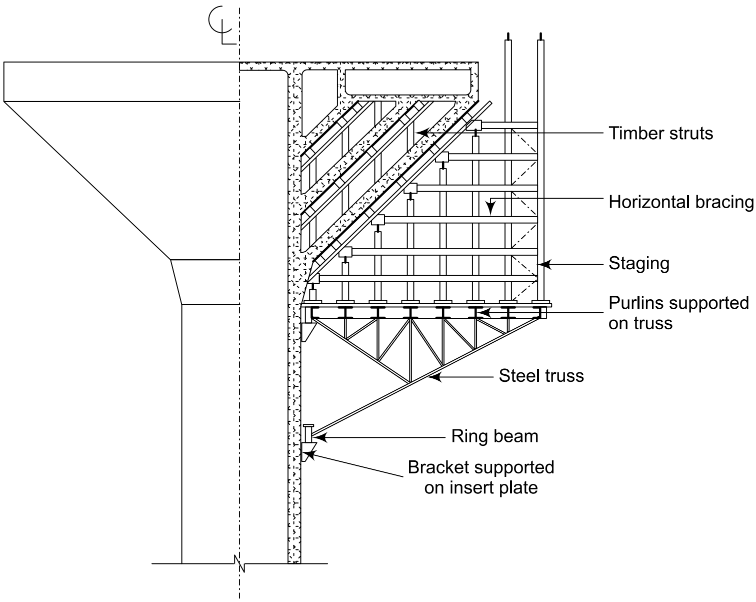


Figure 8.19 Formwork Arrangement for Overhead Water Tank.

8.6 NATURAL DRAFT COOLING TOWER

The automatic climbing formwork for cooling towers is shown in Fig. 8.20. This formwork system is a fully mechanized, automatic-climbing large area formwork. It maximizes rationalization and minimizes manual labor— a short familiarization period is all that is necessary to achieve a one day cycle for pouring sections. The system ensures that units are repositioned without any crane assistance. The system simplifies cleaning of the formwork with 600 mm retraction. It enables the entire scaffold to be adapted to the angle of inclination by turning a single, central spindle. It expedites the reinforcing work, because the scaffold has integrated reinforcement holders. It is possible to pour up to 1,500 mm in height in a one day cycle.

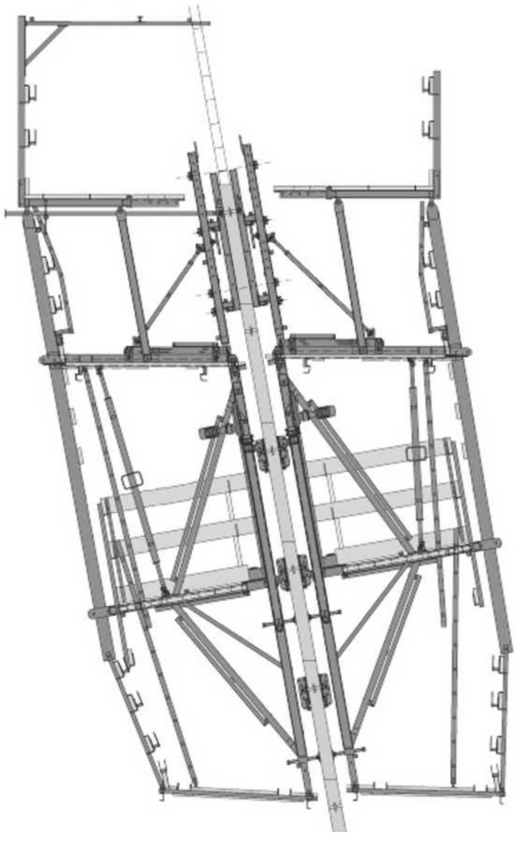


Figure 8.20 Cooling Tower Formwork SK 175 (Courtesy L&T Formwork).

The system ensures high level of safety even when the wind blows strongly well above the ground level, because it is firmly anchored to the structure in all phases. With telescopic working platforms, they are gapless even when climbing is in progress, so the working environment remains safe.

The applications of the cooling-tower formwork SK 175 in the construction of natural draft cooling-tower constructed at Neyveli, Tamilnadu is shown in Figs. 8.21 and 8.22. The project was executed by the L&T-ECC Group.

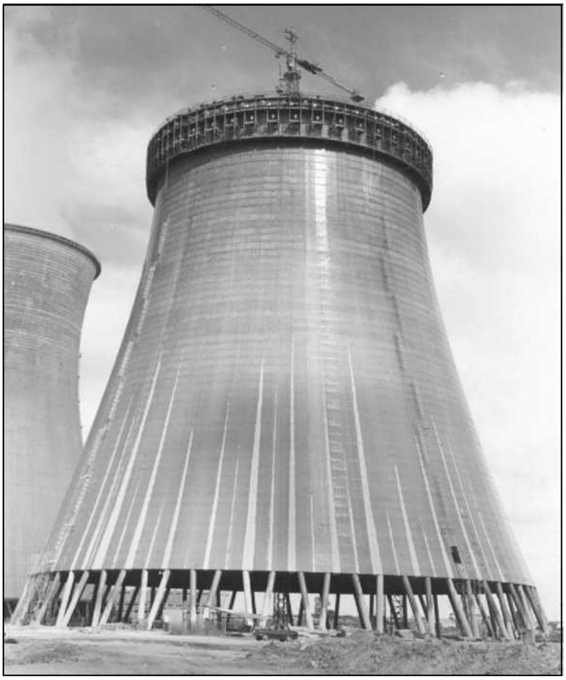


Figure 8.21 Natural Draft Cooling Tower Construction.

Figure 8.22 Inside View of the Natural Draft Cooling Tower Being Constructed Using Cooling-Tower Formwork.

8.7 NUCLEAR REACTOR

Automated self-climbing formwork can be used successfully for the speedy construction of the inner containment wall with large pour heights. This system avoids the provision of staging from the ground level and shuttering for the higher levels is supported on fixtures provided in the lower pours. Tower cranes are used for lifting of the shutters, for handling the reinforcing bars and cables and for moving around the buckets for concreting.

It is difficult to adopt slipforming for this wall, as a large number of circumferential and vertical prestressing cables, many embedded parts and through penetrations have to be provided. The prestressing cables have to have deviations from regular geometries wherever they clash with the embedded parts or penetrations. The embedded parts have to be placed with a very high degree of accuracy.

Figures 8.23 and 8.24 show views of the containment wall under construction at the Kaiga Atomic Power Project.

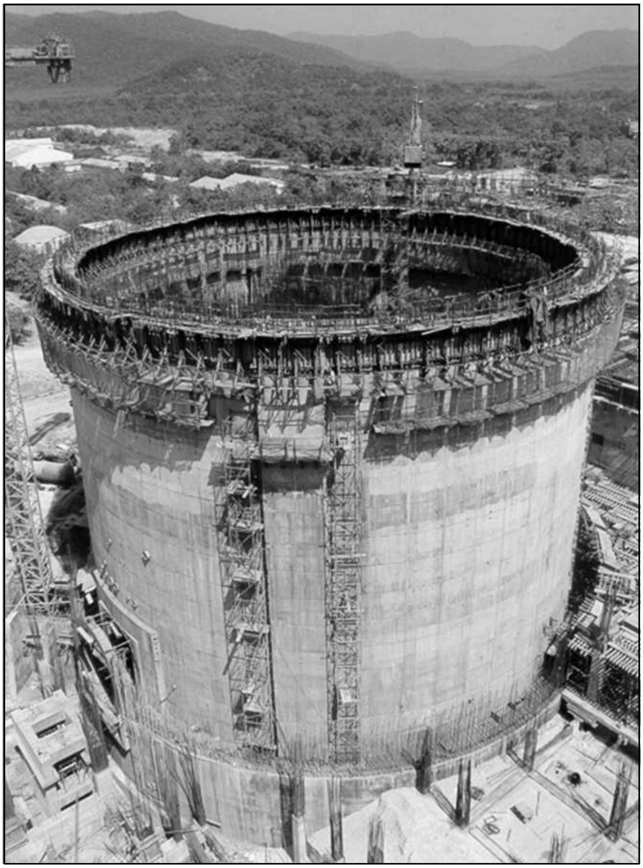


Figure 8.23 View of Containment Wall Under Construction at Kaiga Atomic Power Project. Construction by L&T Limited, ECC - Construction Group.

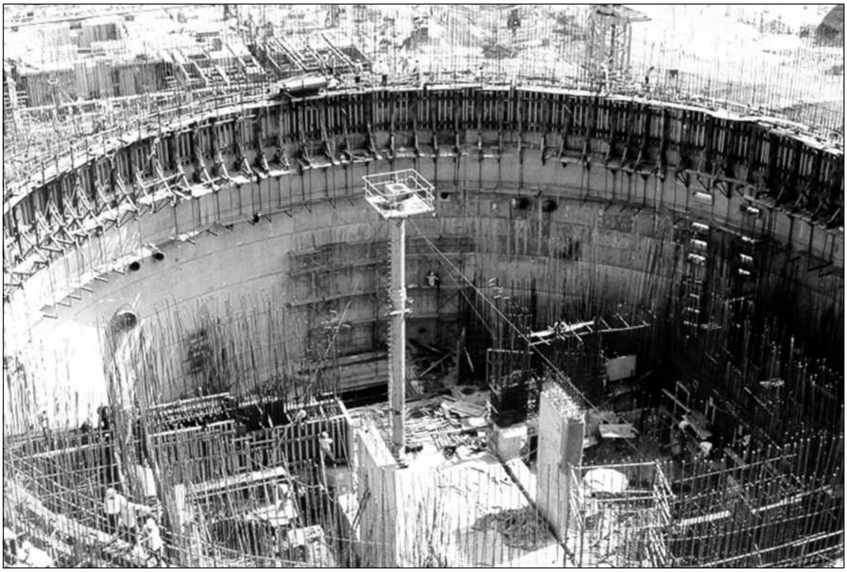


Figure 8.24 Work on the Container Shell in Progress at Kaiga. Project Executed by L&T Limited, ECC - Construction Group.

8.8 TUNNEL

From the consideration of formwork, the tunnel can be thought of consisting of four distinct components: curb, invert, wall and arch. The first two components are easier to form compared to the wall and arch form. A typical tunnel cross section showing the four components is shown in Fig. 8.25.

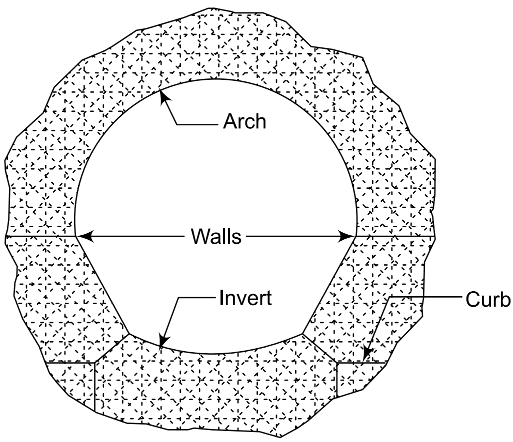


Figure 8.25 Components of a Typical Tunnel.

The most common sequence of constructing these components is in the order – curb, followed by invert, wall, and finally the arch. In some of the cases however, arch may be constructed ahead of the invert. This is possible by erecting the centering on the excavated ground itself. In some cases, the inverts may be cast without the curbs. In some cases, the full lining of the tunnel in a given section of the tunnel can also be placed as the tunnel progresses. This is not commonly practiced however.

The curb form can be made out of timber or metal. The selection criterion is primarily to maximize the reuse. In curb, only the formwork for supporting the vertical face is needed. Arrangements are made to resist the pressure exerted by the concrete on the vertical formwork. For this, suitable inclined braces are designed. Arrangements such as bolting of braces to the rocks etc. are made.

While casting the curbs, arrangements are also made to create the support for the invert form. This is usually done by leaving the anchors when the concrete for the curbs is relatively green.

Traveling bridge is also sometimes used in the construction of a large tunnel. The bridge is employed where continuous casting of the invert is desirable. The traveling bridge is capable of laying side forms of the invert and concreting it. Concrete finishing platforms are also sometimes attached to the traveling bridge. While casting the inverts, provisions are also made to provide anchor bolts which become useful for supporting the wall forms.

For casting the arch form, the sheathing member is supported on a structural frame consisting of a number of circumferential ribs and longitudinal stringers. The sheathing member could be timber or steel. The ribs are braced suitably according to the design. A typical steel arch form is shown in Fig. 8.26. The length of a tunnel formwork is decided depending on the requirement. The tunnel forms are composed of various sections or units.

Once the concrete for a given section of the tunnel is completed, there are two ways in which the form can be shifted to its new location. In one of the methods, the arch form is collapsed partially and moved to its new location on a traveler or a jumbo. The forms are able to be moved to their location through the in-place adjacent forms. Such type of forms are also referred to as 'telescoping forms'. In another method, the tunnel forms are collapsed sufficiently for stripping. These forms are then moved to a new location beyond the recently completed sections of the tunnel. Such type of forms are also referred to as 'non telescoping forms'.

The formwork arrangement for the tunnels is governed by factors such as (a) dimension of the tunnel, (b) concrete placing sequence, and (c) construction joint configuration of the lining.

For example, when the tunnel is relatively shallow, cut and cover method of tunnel construction can be used. In this method, the support for the outer formwork for wall is taken from the excavated surface while the inner formwork for wall is similar to that used in the normal wall application. Form ties are used to resist the pressure exerted on the wall form. Spreaders are used to maintain the desired distance between the outer and the inner formwork. The formwork for arch portion of the tunnel is usually placed from outside using a crane. Normally for some distance near the crown, the outer formwork for the arch portion is not provided. This space is utilized for pouring the concrete. The area where the top form is not provided, is hand finished later when the concrete in the adjoining area is complete.

The schematic arrangement of the formwork in cut and cover construction method is shown in Fig. 8.27. In cut and cover, sometimes the traveler is designed to let the site traffic pass beneath it unhindered. This aspect is clearly shown in Fig. 8.27.

A field application of cut and cover method of tunnel construction provided by Doka is shown in Fig. 8.28. The tunnel is on the route of a new high speed twin-track railway line built in central Greece. The client opted for Doka formwork on the inside, combined with custom steel formwork on the outside. The cross section of the tunnel is circular having an inside width of 12.12 m and an inside height of 8.85 m. The tunnel was constructed in 48 steps with each step of 12.5 m. On an average, two steps a week were cast.

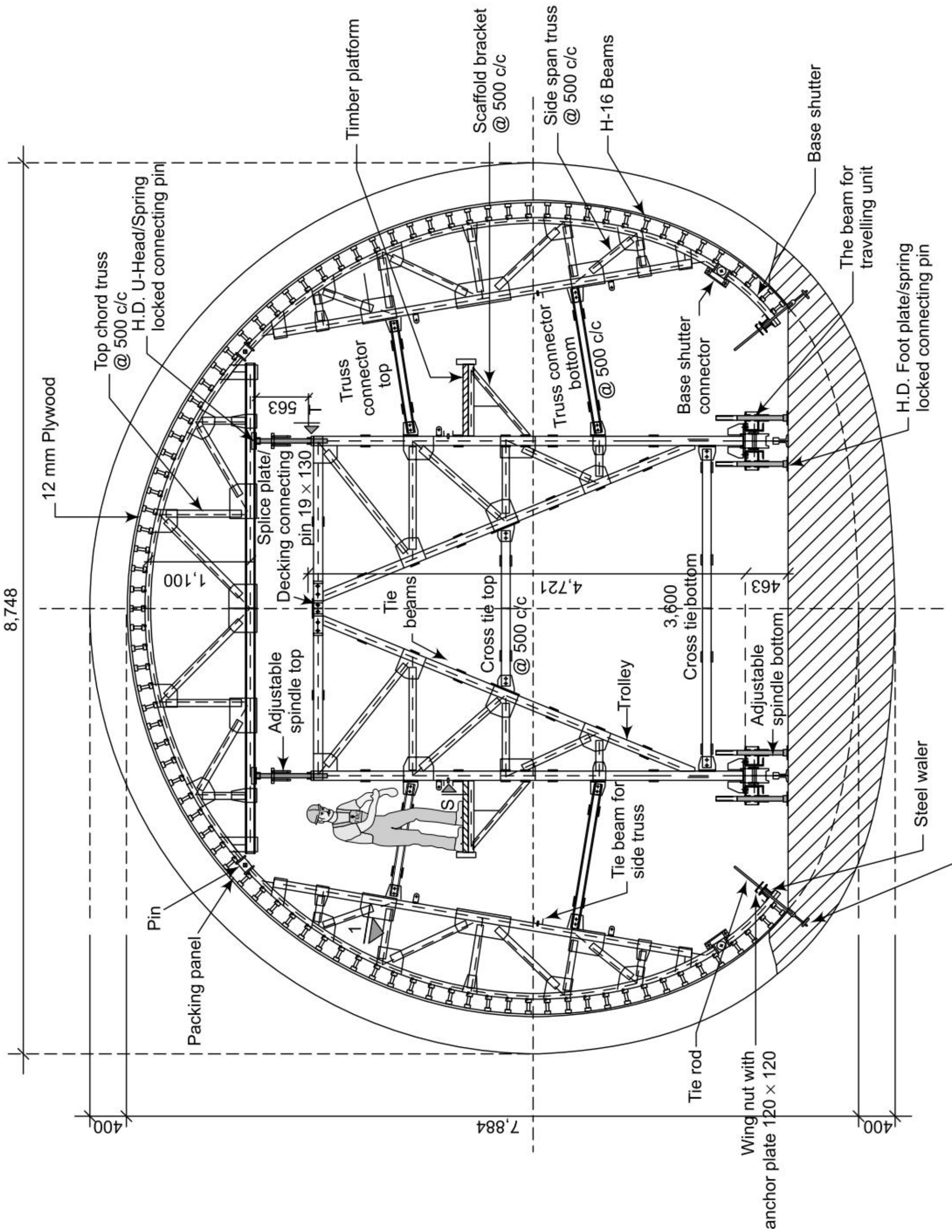


Figure 8.26 A Typical Steel Tunnel Form.

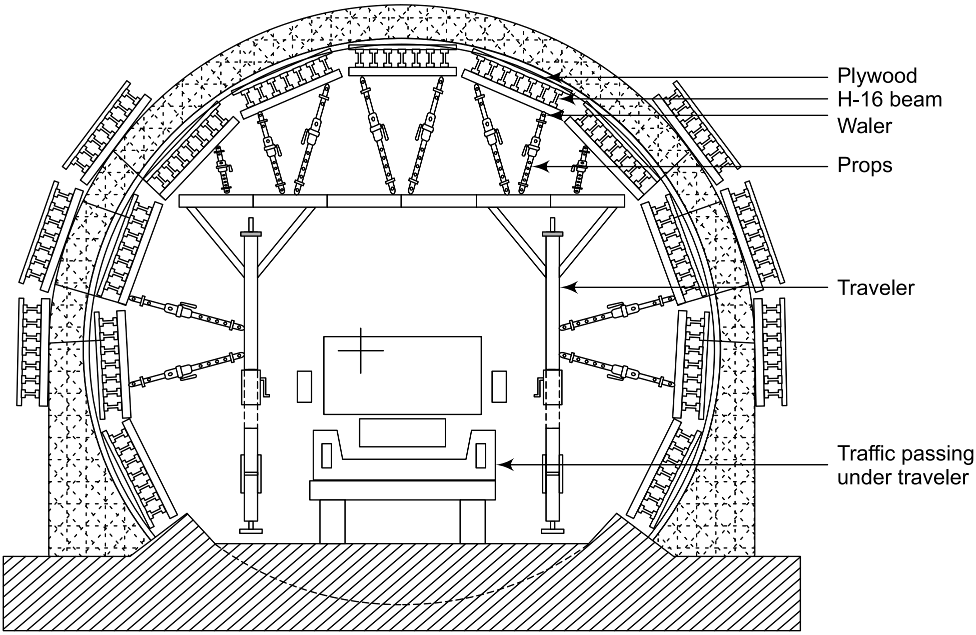


Figure 8.27 Cut and Cover Method of Construction (Courtesy Doka).

Figure 8.28 Cut and Cover Construction Method in Use for Tunnel Construction (Courtesy Doka).

When the concreting operation is planned to be continuous except for the holidays and the breakdowns, the advancing slope method of tunnel construction is used. In this method, the concrete is placed from near the crown. The arch form is prepared in units or sections. The length of a unit or section varies depending on the requirement. In addition, there could be a number of sections employed at a time depending on the requirement. The sections are stripped in a given sequence.

They are then collapsed and telescoped through other sections. Form traveler is used to re-erect them at their new location. Form traveler is also known as 'jumbo'.

Figure 8.29 shows the formwork arrangement for the construction of the junction of the inlet shaft and tunnel for B.M.C. at Bandra, Mumbai. The project was executed by AFCONS.

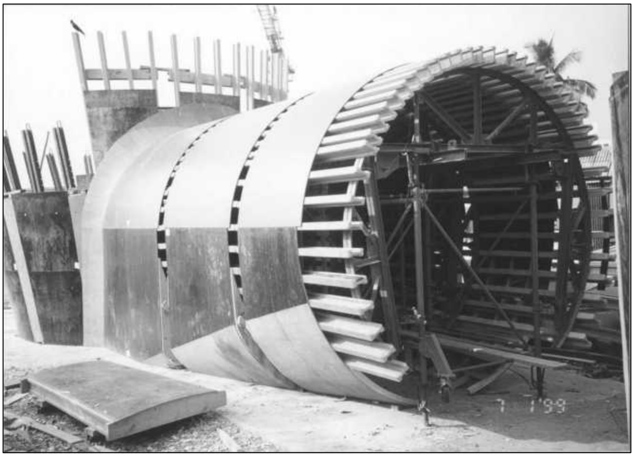


Figure 8.29 Shuttering in Position for Construction of Junction of Inlet Shaft and Tunnel (Courtesy L&T Formwork).

A special custom built tunnel formwork was used to construct two tunnels each of 300 m length for Chawri Bazar Station for Delhi Metro Rail Corporation (Fig. 8.30). The tunnel has a diameter of 8.750 m and the thickness of the lining is 400 mm. The length of each concrete pour is 9.60 m. The contractor mobilized 3 sets of tunnel forms with 5 panels each of 2 m length. The form-work consisted of 12 mm plywood as sheathing materials supported on H-16 beams. These were supported on one top truss and two side trusses. Truss connectors were used to transfer the load from the trusses to the trolley. The trolleys had tower spindles at the bottom for height adjustment.

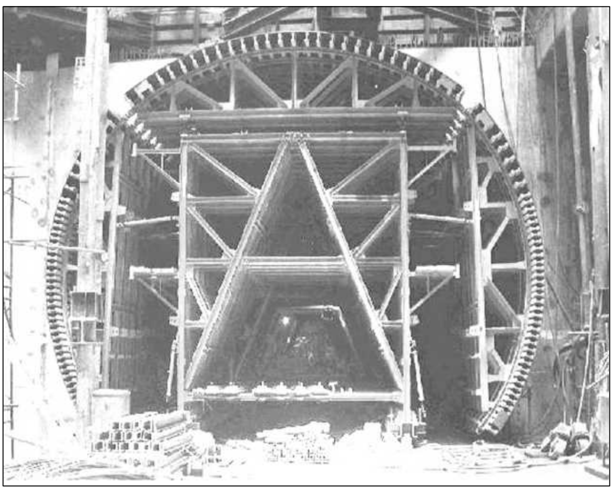


Figure 8.30 Formwork in Position for Construction of Tunnel (Courtesy L&T Formwork).

Trusses were also connected to the trolleys through turnbuckles. The turnbuckles were used for the alignment of the formwork. Walls were cast in two stages. The top slab was cast after the walls. Movement of the whole formwork system from one location to another was carried out manually using wheels mounted on rails. The contractor achieved a cycle time of 8 days on an average.

Figure 8.31 shows the Doka tunnel formwork in use for constructing a circular tunnel of inside width 9.70 m and an inside height of 6.8 m. The length of the tunnel formwork was 12 m. The tunnel formwork had all-hydraulic formwork traveler. The all-hydraulic tunnel formwork traveler was easy to operate which enabled very short cycle times to be achieved.

Figure 8.31 Mining Method of Construction in Use for a Tunnel Jobsite (Courtesy Doka).

Some more schematic sketches for tunnel formwork are shown in Figs. 8.32 and 8.33.As can be observed, depending on the requirement and dimension of the tunnel, the cross section of the central formwork member changes.

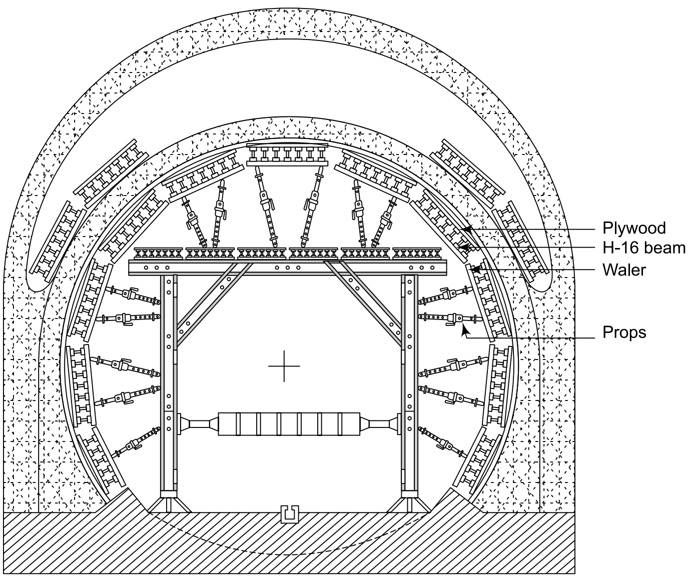


Figure 8.32 Formwork Arrangement for Tunnel Construction (Courtesy Doka).

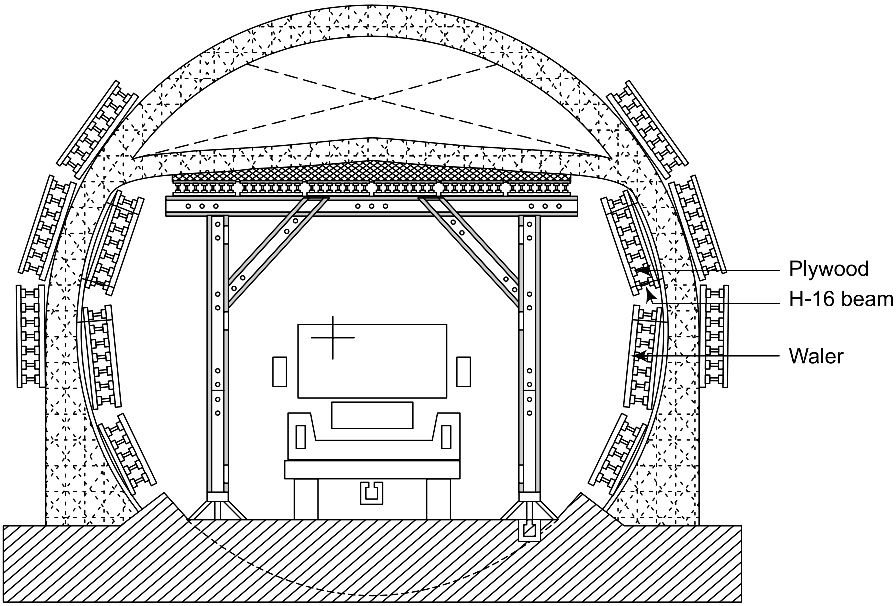


Figure 8.33 Formwork Arrangement for Tunnel Construction (Courtesy Doka).

8.9 LIFT SHAFT

The formwork for the lift shaft provides a platform for the shutter and the workmen inside the closed area of the liftwell, and a deshuttering mechanism for stripping of the formwork, without dismantling the panels or their sheathing. Self positioning climbing pawls eliminate the operations involving anchorage from threaded components on the inside surface for making the platform. Panels can be lifted integrally in deshuttered position along with the platform and erected for the next pour. The formwork for the lift shaft uses standard wall formwork components with a few additional accessories. The system uses the following components:

1. shaft climbing pawl,
2. shaft corner plate,
3. shaft beam extensions,
4. shaft main beam end,
5. shaft corner spindle,
6. corner waling, and
7. waler square plate.

Figure 8.34 shows the arrangement of lift formwork in plan. The outer formwork is not shown for clarity. Figure 8.35 shows another view of the lift formwork. Here again the outside formwork is not shown.

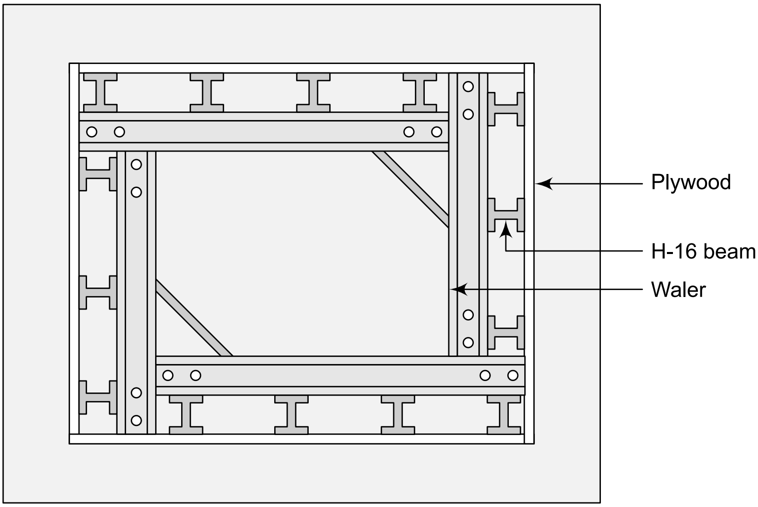


Figure 8.34 Arrangement of Lift Formwork in Plan.

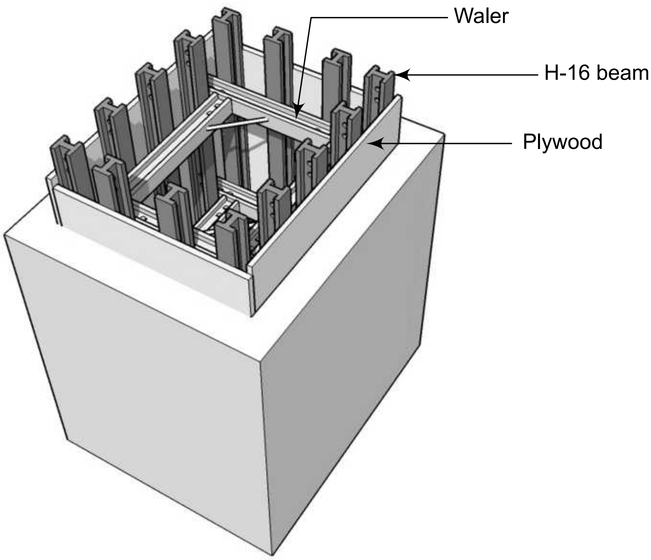


Figure 8.35 Lift Formwork (inside).

REVIEW QUESTIONS

Q1. True or False

(a) Some of the special structures are: shells, domes, folded plates, natural draft cooling towers, nuclear reactor, tunnels, lift shafts, etc.

(b) Formwork arrangement for the tunnels is governed by factors such as (i) dimension of the tunnel (ii) concrete placing sequences (iii) construction of joint configuration of the lining

(c) Components for the lift shaft are—shaft climbing pawl, corner plate, beam extensions, main beam, corner spindle, corner waling, and waler square plate.

(d) The expression Y_i for circular shell roof is = $R - \sqrt{(R^2 - x_i^2)}$

(e) Decking and centering are the two major components for formwork design of special structures.

(f) The four distinct components of tunnel are: curb, invert, wall, and arch.

Q2. Prepare a summary report on formwork issues involved in

(a) National construction of spiritual assembly of Bahai faith at New Delhi (for shells)

(b) Construction of dome roof of a reactor building

(c) Construction of central secretariat rotary dome in DMRC project

(d) Construction of elliptical dome in DMRC project

(e) *Cast-in-situ* folded plates for air hangar at Mumbai.

(f) Overhead water tank construction

(g) Natural draft cooling tower construction

(h) Kaiga nuclear reactor construction

(i) Tunnel construction

Q3. Discuss the cut and cover method of tunnel construction.

Chapter

9 Formwork for Bridge Structures

Contents: Introduction; Formwork Arrangement for Caisson; Formwork for Piers and Pier Caps; Formwork for Bridge Superstructures; Formwork for Bridge Railings/Parapets/Edge Beams; Cases in Failure of Temporary Support Structures of Bridges

9.1 INTRODUCTION

In this chapter the forms required for the substructure and superstructure of a bridge are discussed. The substructure consists of the foundation of the bridge, which supports two or more piers joined together at the top by a pier cap. On the pier cap, the superstructure rests, consisting of the deck of the bridge. The bridge designer can come up with several types of foundation. The foundation type is dependent on a number of factors, such as the span of the bridge, location of the bridge, bearing capacity of the material on which the foundation rests, and so on. In this chapter, formwork for caissons are discussed. Caissons are hollow structures sunk to a very great depth below the ground or river bed. In caissons, the formwork for cutting edges, curbs, and caisson wall are discussed. The formwork for some commonly used pier shapes, pier caps, *cast-in-situ* bridge girders, and deck slabs are also discussed.

The planning and building of formwork for the components mentioned are very similar to what has already been discussed in the chapters on column formwork, wall formwork, and slab and beam formwork. For example, a short pier is constructed in a similar manner to the column formwork discussed in Chapter 6. Similarly, the deck slab construction illustrated in a later part of this chapter is also very similar to the slab and beam formwork explained in Chapter 7. Even the architectural finishes for piers, pier caps, *cast-in-situ* girders, and deck slabs are provided in a similar manner as illustrated in previous chapters. Site conditions are important factors in the planning and building the forms for bridge elements.

Steel is a very common formwork material for forming bridge elements, though timber can also be used. Some proprietary systems such as Doka, etc. also offer a number of solutions for bridge formwork. The formwork systems are also discussed in this chapter.

Some cases on the failure of temporary supports for bridge are also discussed in this chapter.

9.2 FORMWORK ARRANGEMENTS FOR CAISSONS

As mentioned earlier, a caisson is a hollow structure which is sunk to a very great depth below the ground or river bed. The process of sinking proceeds with excavation inside the caisson or underneath it. Caissons of varied shapes, such as circular, rectangular, double D-shape, etc., are designed depending on the requirements. For the purpose of formwork, a caisson has three broad parts: it cutting edge, curb, and wall. The formwork arrangement for each of them is discussed briefly in the following sections.

9.2.1 Cutting Edge

The caisson bottom has a sharp cutting edge which assists the caisson in sinking below the ground or the river bed. The cutting edge may sometimes be tipped with steel.

The cutting edge is cast in a steel fabricated unit which is left in place (Fig. 9.1). The fabricated steel form produces a hard sharp edge which assists the caisson in moving down. Contrary to the outside form, the primary requirement of the inside or internal forms is the ease with which they are able to be removed and replaced.

In order to avoid the possibility of settlement during curb concreting, the annulus on which the cutting edge is to be placed is leveled and compacted uniformly. The cutting edge is usually fabricated in four to five pieces. Each piece is lifted and placed in position with the help of a crane and temporary welding is carried out. The joints are properly welded after final alignment. Minor adjustments to the leveling of the cutting edge are done by providing shim plates between the cutting edges and the timber planks. Wooden wedges are packed between the cutting edge and timber plank to allow for distribution loading on the timber planks during curb concreting. The cutting edge is placed at about 300 mm above the prevailing water level. A cutting edge in position for a circular well is shown in Fig. 9.2. It can be noted that the cutting edge is resting on timber sleepers. The sleepers need to be cut or removed before the start of sinking.

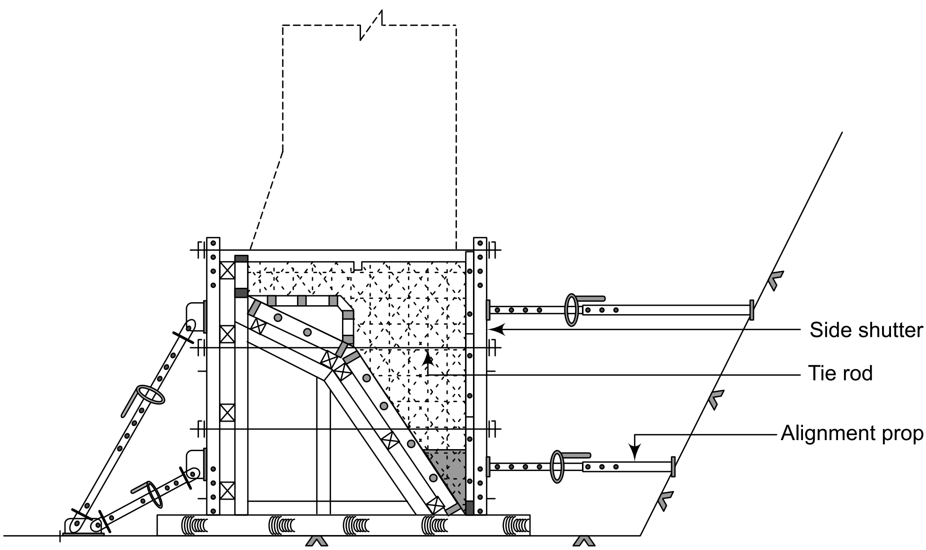


Figure 9.1 Typical Section of Cutting Edge Formwork.

Figure 9.2 Cutting Edge of a Caisson for a Circular Well.

9.2.2 Construction of the Curb

The internal shutters are fixed and aligned correctly and supported by suitable props. Curb reinforcement is fixed and a starter bar is welded to the cutting edge. Vertical water jetting pipes are fixed at the middle of the steining, and rest on the curb shutter. The external shutter is fixed and aligned.

Concreting is carried out in single pour. Concrete is usually transported by transit mixers and placed in position either using a concrete pump or by a crane and bucket arrangement.

The outside shutters are stripped after 24 hours of the concrete pour. The inside shutters are stripped usually stripped after 72 hours of concrete pouring.

Grounding operations shall commence by removing the sleepers diametrically opposite to each other in pairs, to prevent undue tilting to any particular side. Regular check for tilt has to be taken in both directions, and accordingly the removal of the sleepers can be regulated. For this, one needs to count the number and mark the number of sleepers during grounding.

Once all the sleepers and sand bags are removed, grabbing of soil shall start in a controlled manner, aided intermittently by a manual laborer removing any soil beyond the reach of the grab in blind zones and stacking it in the dredged hole centre, to be subsequently removed by the grab excavator. This process shall continue until the water level is reached, at which point only mechanical grabbing is permitted. The curb shall sink to a maximum of 2.0 m, keeping 0.5 m above ground level to facilitate construction of first steining.

9.2.3 Construction of Well Steining

Before start of shuttering operations, the concrete surface shall be cleaned by water/air jetting to clear of all debris and muck from the sinking process. The inside shuttering and rebar fixing works are to be taken up; the first alignment is checked and thereafter the outside shutter shall be fixed and aligned correctly. Care shall be taken to see that the shuttering is aligned true to tilt.

Figure 9.3 View Showing Inside Shutter in Position.

Figure 9.4 View Showing Reinforcement in Position.

Concreting is done in a similar fashion as for the curb. De-shuttering is done after final setting, usually after about12 hours. The first three lifts of steining are cast singly and then sunk so that sufficient grip is obtained for stability purposes. Thereafter the lifts are normally cast in a two lift cycle, some time also three lifts.

Figure 9.5 Reinforcement Tying for Next Lift in Progress for One Well and Cutting Edge Work in Progress in Other Well.

Figure 9.6 View of Inside Shutter in Position for Next Lift.

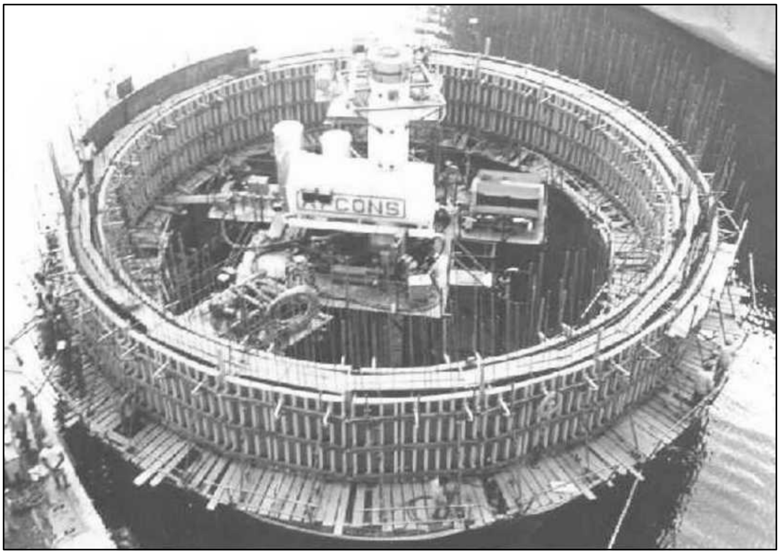


Figure 9.7 L&T- Wall Formwork with Climbing System in Use for Casting 20 m High Caisson Wall of 18 m Diameter (Contractor: AFCONS).

9.3 FORMWORK FOR PIERS AND PIER CAPS

In piers we discuss two cases: piers with relatively low heights and piers of significant heights. Small piers can be constructed similar to the column construction discussed earlier. Piers of significant heights can be slipformed or can be constructed using climbing formwork.

Piers can be of varied shapes, such as rectangular, circular, octagonal, or some other complex shape for architectural purposes. Pier sides may be vertical, stepped or battered. The type of pier formwork used is dependent on the shape, height, and number of piers in a bridge. For example, where considerable repetitions are envisaged the pier formwork can be custom made in steel, while for a small pier where there are very few repetitions one can consider using standard formwork components for pier forming. In cases where the height of the pier is significant, pier forms can be repeated vertically in successive lifts. One can consider using the climbing formwork explained in previous chapters for this purpose. However this is only possible where the pier has a constant cross–section for considerable heights.

Where a large number of similar piers of relatively small height are to be formed, the pier form may be planned to move horizontally. However, this can only be a solution for achieving maximum reuse if time is not a constraint.

Figure 9.8 shows three circular piers for the construction of a flyover for a road project. The circular piers have a relatively small diameter and the contractor has used steel formwork for constructing them. The casting is done in small lifts and the formwork arrangement is similar to any other circular column using steel forms. The supports for the column formwork are provided from the brackets fixed on the previously cast concrete. The circular forms are similar to that discussed in Chapter 6.

Figure 9.8 Round Pier Under Construction (LMNHP-Package 12).

Figure 9.9 illustrates the formwork for pier construction for the Delhi Metro Project. The formwork components are made of steel and casting has been done in a single lift. The formwork is fabricated using steel sheathing and steel stiffeners. The contractor has used custom made walers and alignment props. The contractor could afford to do this because of a large number of similar piers to be constructed for the project. The working platform at the top for the crew members can also be seen. The completed piers using the steel formwork can also be seen in the figure.

Figure 9.9 Formwork for Pier (Note: Casting in Single Pour).

Piers of significant height can be formed using climbing formwork or slipform. Figures 9.10–9.13 illustrate the application of climbing formwork for the construction of an octagonal pier. Figure 9.10 shows the octagonal pier in plan. In Fig. 9.11 a part plan of the formwork is shown for the octagonal pier. The formwork arrangement is like any other wall and column formwork explained in previous chapters.

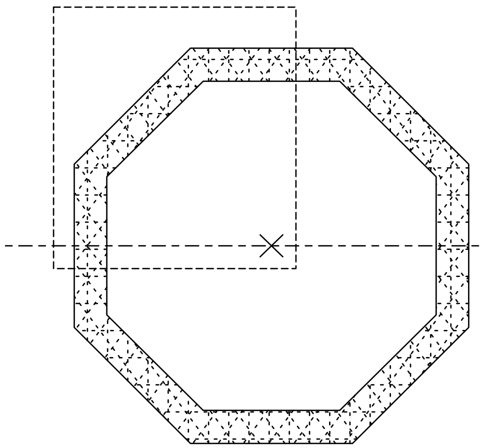


Figure 9.10 Plan of an Octagonal Pier.

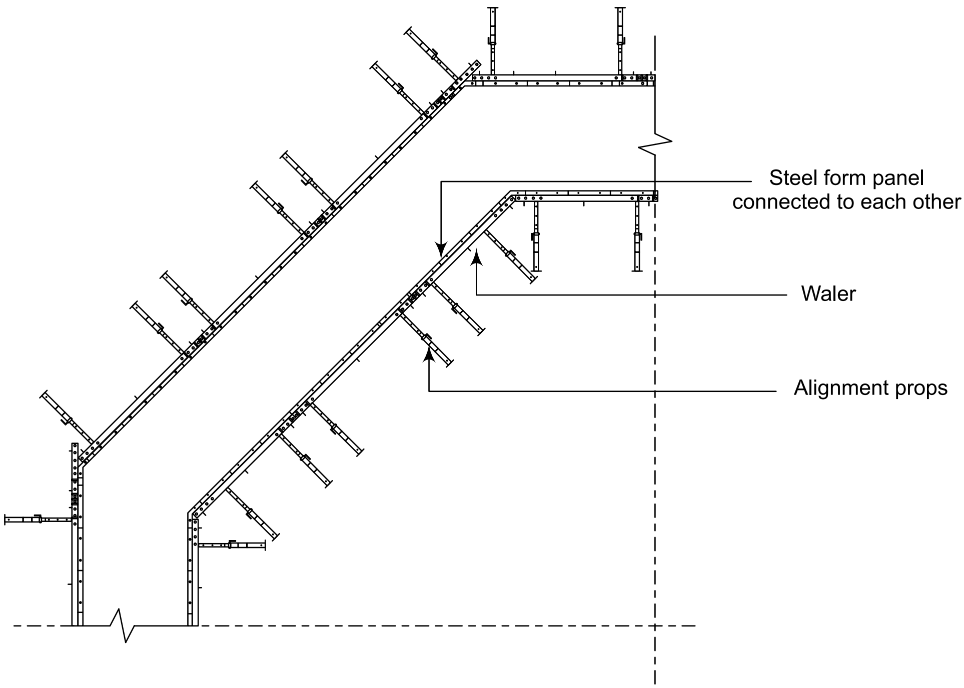


Figure 9.11 Part Plan of Formwork Arrangement for Octagonal Pier.

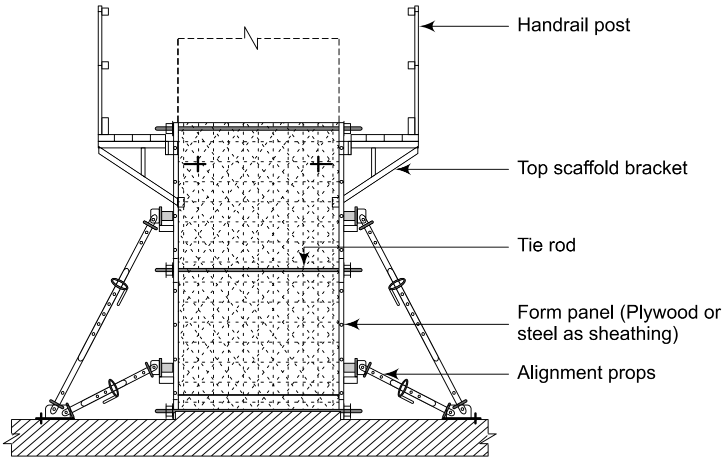


Figure 9.12 Sectional View of Formwork Arrangement for Lower Lift.

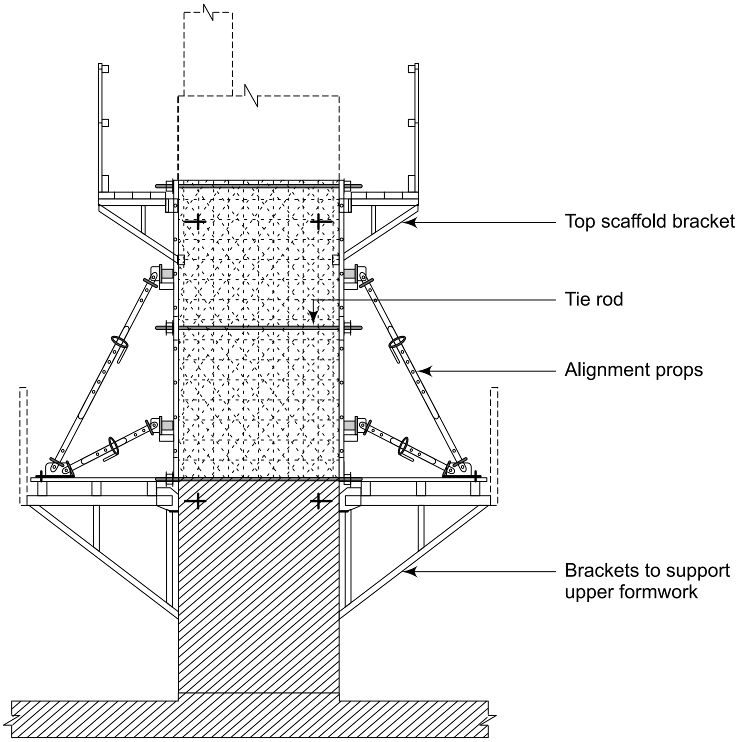


Figure 9.13 Sectional View of Formwork Arrangement for Upper Lift.

Figure 9.12 shows the formwork arrangement for casting the first lift of the octagonal pier. Such piers can be constructed using both plywood and steel sheathing. The selection of sheathing material is dependent on the number of possible reuses in the project. Figure 9.13 shows the formwork arrangement for the second lift. It can be seen that the formwork for the second lift is supported on the brackets fixed onto the previously cast concrete. The arrangement is very similar to that discussed in Chapter 5 under climbing formwork.

Figures 9.14 and 9.15 show the application of climbing formwork for the construction of a hollow tapered circular pier. The pier was constructed for the Jammu Udhampur Rail Link project. For the construction of another pier for the Jammu Udhampur Rail Link slipform was used. The arrangement is shown in Fig. 9.16. The pier is located in an extremely difficult terrain. Slipform was also used to construct very tall piers for the Konkan Rail project. More details of the application of slipform are given in Chapter 11.

Figure 9.14 Construction of Pier Using Climbing Form (Jammu Udhampur Raillink Project).

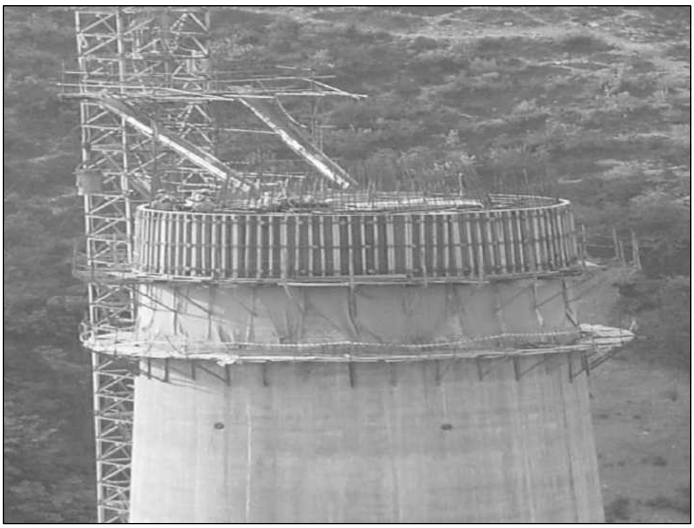


Figure 9.15 Closer View of Construction of Pier Using Climbing Form (Jammu Udhampur Raillink Project).

Figure 9.16 Construction of Pier Using Slipform (Jammu Udhampur Raillink Project).

The application of custom–made steel formwork for short piers and pier caps is shown in Figs. 9.17 and 9.18. The formwork arrangement shown in Fig. 9.17 was adopted for the construction of a pier and pier cap for the Delhi Metro construction project, and Fig. 9.18 shows the formwork arrangement adopted for a bridge project in Delhi. In Fig. 9.17, components such as the props and walers are all customized for the project. In Fig. 9.18, the forms are made up of modular steel forms which can be used for different formwork applications.

Figure 9.17 Formwork Arrangement for Pier and Pier Cap for Delhi Metro Project.

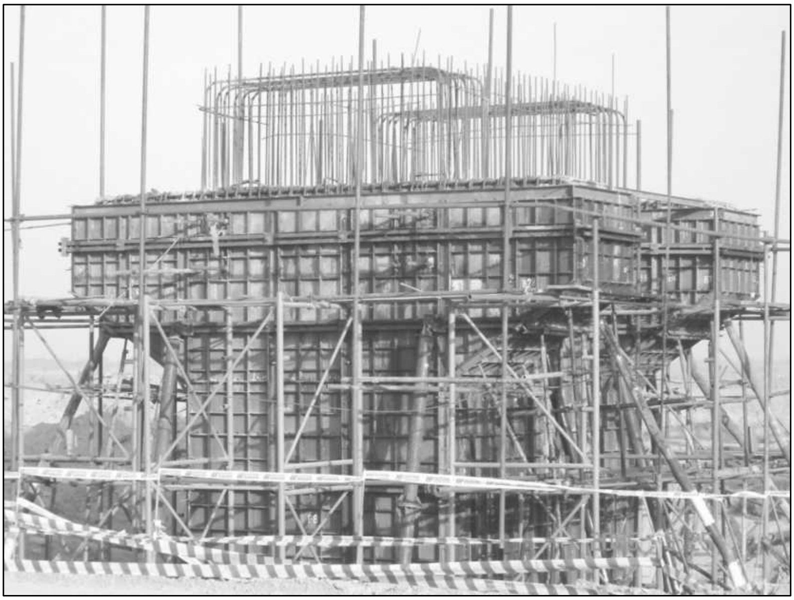


Figure 9.18 Formwork Arrangement for Pier and Pier Cap for a Bridge Project in Delhi.

The formwork arrangement for a pier cap for a flyover project in Delhi is shown in Figs. 9.19–9.22. Figures. 9.19–9.21 show the schematic formwork arrangement, and Fig. 9.22 shows the actual photograph of the formwork arrangement adopted on site. The formwork arrangement for the pier cap is supported from the pier itself and not from the ground. The pier supports the truss–shaped brackets, which in turn support the props and other components. The sides of the pier caps were formed using steel shutters. The forms were supported using structural steel members.

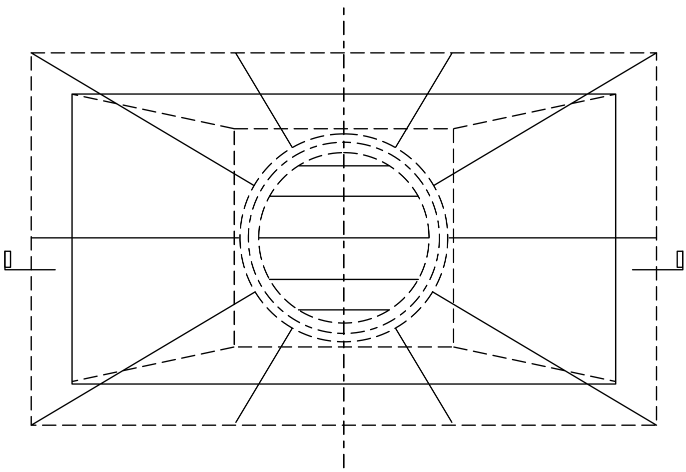


Figure 9.19 Plan Showing Bracket and Beam Arrangement.

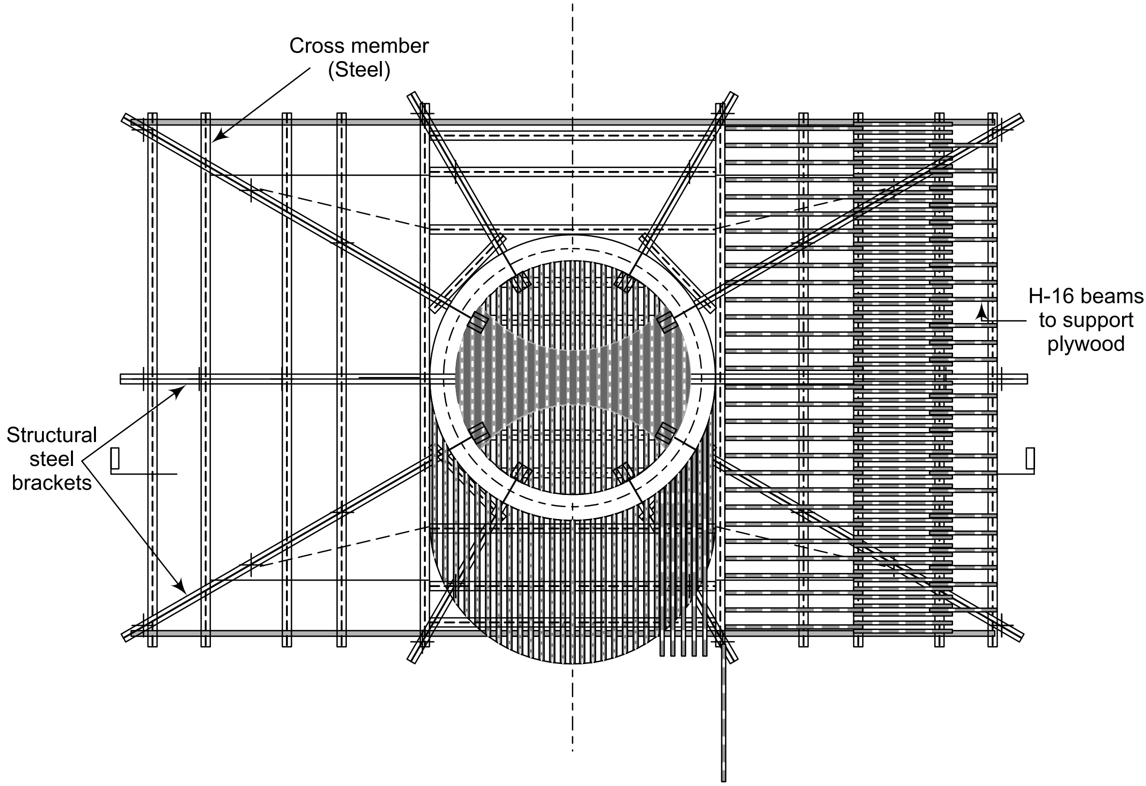


Figure 9.20 Plan Showing Form Support Arrangements.

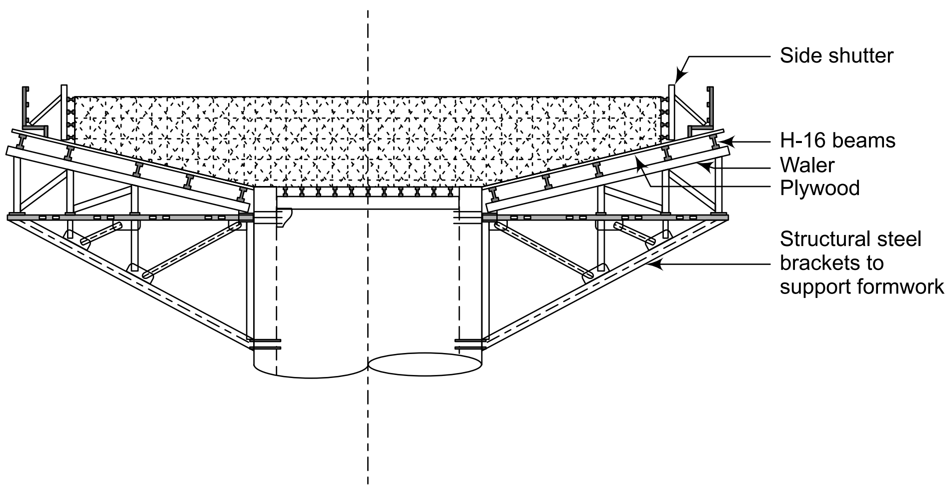


Figure 9.21 Sectional View of Formwork for Pier Cap.

Figure 9.22 Formwork Arrangement for Pier Cap Used in the Construction of Flyover at Delhi.

9.4 BRIDGE SUPERSTRUCTURES

In this section on bridge superstructures, the formwork for the deck slab, bridge girders, and parapet or railings are described. The bridge can have a simple slab arrangement for short spans, while there may be girder and slab arrangement for longer spans. One such arrangement for a short span is shown in Fig. 9.23. The project was executed by V.P.Thrimathy (Nilambur - Kerala). The deck slab is supported on heavy–duty towers. Plywood has been used as the sheathing material to form the deck slab. The sheathing material is supported on H-16 beams which in turn are supported on steel walers. The steel walers are resting on the heavy duty tower. The reader may have noticed the similarity between deck slab formwork used with any other of the slab formwork discussed in Chapter 7.

There are different arrangements for the superstructure followed in bridge design. This requires different ways of forming the deck slab, bridge girders, and parapet or railings. For example, in some bridges, girders may be cast first (or erected if they are precast), followed by the casting of the slab. In some bridges there may be *cast-in-situ* box girders. For casting the box girders, the bottom slab is cast first, followed by the web of the box girders. The casting of the top slab is followed by the casting of the web of box girders. Sometimes the segments of box girders may be precast, which are erected and joined together. Pre-stressing activities may also be required for girders, depending on the design requirements. The formwork arrangements change depending on the type of arrangement of bridge superstructure. Some commonly used formwork and supporting arrangements are discussed in the following sections.

Figure 9.23 Formwork and Supporting Arrangement for Bridge Deck Slab Using Proprietary Formwork.

9.4.1 Deck Slab and Girder Supported by Tubular Steel Scaffolding

Bridge deck forms can be supported on tubular steel scaffolding. For this the ground supporting these scaffolds are leveled and compacted. In some cases the footplates of the scaffolds rest on wooden or concrete sleepers. In some cases, after leveling and compaction, a 100 mm thick layer of plain cement concrete is also provided to improve the ground conditions. Care should be taken to ensure that the footplates rest properly on the concrete sleepers. Also, whilst preparing the ground for erecting the scaffolds the scouring of the base should be prevented. This is accomplished by placing sand bags all around the scaffolding area as shown in Fig. 9.24. This arrangement was adopted in the construction of the Saryu Bridge by Hindustan Construction Company. Furthermore, the bracings as per the design, or according to the manufacturer's advice, should be provided (see Fig. 9.25). Provision of handrails (see Fig. 9.26) and locations for the tying of safety belts should be provided.

The tubular steel scaffolds can be used to support both the casting of the box girders or pre-stressed concrete bridge girders.

In Fig. 9.27, the application of a L&T heavy duty tower has been shown. The system was used by Hindustan Peter Oates (P) Ltd, Chennai, for supporting the bridge deck slab and girder at the Koratalaiyar Bridge project. Another application of L&T heavy duty towers is shown in Fig. 9.28, for supporting the girder and slab at a considerable height at the Koyna Bridge site. The latter project was executed by Soma Enterprises Limited.

The tower legs can rest on timber sleepers or channel sections. Compaction of supporting soil is required in order to avoid uneven settlement. Each tower leg must have a footplate and tower spindle assembly. The tower spindles are helpful for height adjustment. For tower heights of greater than 6 m, the individual towers must have bracings as per the manufacturer's advice to connect to each other. The spacing of towers is decided based on the formwork design loads. The details of design of such shoring towers are given in Chapter 12.

Figure 9.24 Arrangement of Sand Bags to Prevent Erosion of Soil Underneath the Scaffold.

Figure 9.25 Tubular Steel Scaffold in Position (Note the Bracings).

Figure 9.26 Arrangement of Tubular Steel Scaffolds for Casting Bridge Deck.

Figure 9.27 L&T Heavy Duty Tower System in Use for Deck Slab and Girder Formwork Support.

The bridge girders could be *cast-in-situ* or precast. Sometimes the girders could be made of steel. Deck slabs are cast on the precast or *cast-in-situ* girder. The supporting arrangement for girders is similar to any other of the beam support arrangements discussed in Chapter 7. However, when the girders are located at higher elevation, or they are located over water, the support arrangement needs careful planning and design. Also, when the soil supporting the girder formwork support is poor, special arrangements needs to be made. A number of failures have taken place due to the supporting soil giving way.

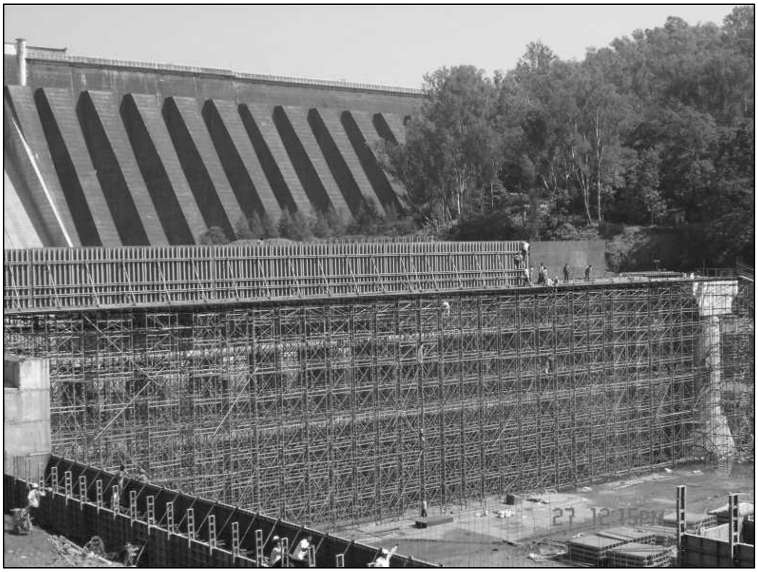


Figure 9.28 L&T Heavy Duty Tower System in Use for Deck Slab and Girder.

Figures 9.29 and 9.30 show the formwork arrangement for a web of box girder for ROB at Khairane, Vashi, in Navi Mumbai. The contractor, Kvaerner Cementation Ltd., used L&T formwork. As can be seen, the formwork consists of plywood, H-16 Beams, steel waler, and tie rods. For the alignment of forms collapsible tube props have been used. The alignment props are supported on a heavy duty tower. The tower legs are supported on a concrete pedestal. The concrete pedestals are placed on well compacted soil.

Figure 9.29 Closer View of Form Arrangement for Web of Box Girder.

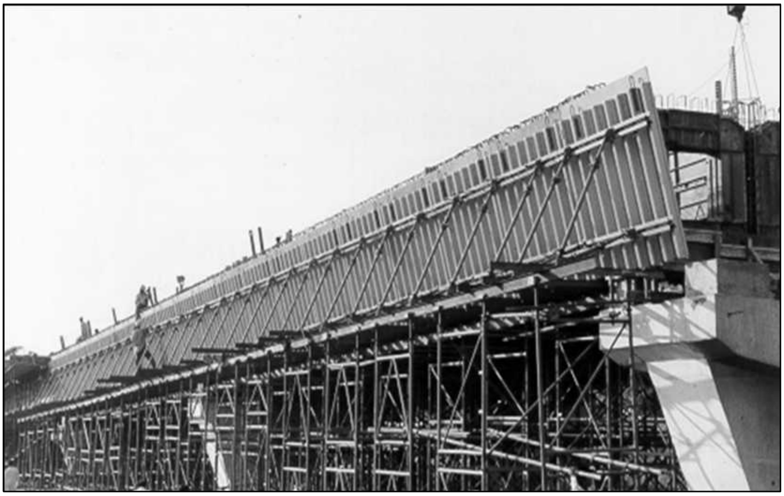


Figure 9.30 Formwork Arrangement for Web of Box Girder.

L&T formwork was used by Technibharathi Ltd. (Cochin) in one of the bridge projects of NHAI. The bridge consisted of three spans of 48.35 m. Each span consisted of three *cast-in-situ* I-girders of 3.86 m depth. The girders were connected with diaphragm walls at the two ends and at the centre of the span.

The form arrangement for the girder sides is shown in Fig. 9.31. It contains usual L&T wall formwork with the exception of walers. As can be seen, the forms required the application of a tailor–made waler to suit the haunch portion of the I-Girders.

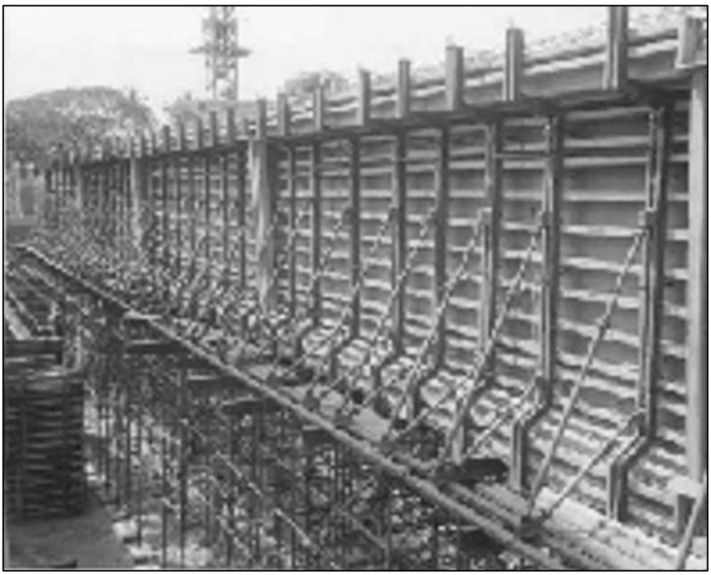


Figure 9.31 Formwork Arrangement of *Cast-in-situ* Girder Using L&T Formwork.

The girder formworks were supported on heavy duty towers. The heavy duty towers in turn were supported on trusses. The trusses were resting on two pile caps in addition to one intermediate support. The heavy duty towers were not supported on the river bed as there was continuous flow of water in the river.

9.4.2 Deck Forms Supported by Steel Cribs/Trestles

It is also common to support the bridge decks using steel cribs or trestles. These cribs are usually fabricated using angle sections. One typical steel crib is shown in Fig. 9.32. Depending on the loads, the designs of cribs vary. The cribs are fabricated in modular units, and are joined together by bolts to arrive at the desired height. The cribs rest on concrete pedestals. One such typical pedestal and crib is shown in Fig. 9.32. The bolting arrangement of the crib with the pedestal is clearly visible in Fig. 9.32. Individual cribs are braced together as per design requirements.

Figure 9.32 Connection Details of Trestle with Concrete Pedestal.

The formwork arrangement for the girders and slab for a major bridge for the LMNHP Project at chainage 442+256, constructed by Madhucon Projects Ltd., is shown in Figs. 9.33 and 9.34. The formwork for girders and slab are supported on trestles. The bearing capacity of the supporting soil was poor. Thus the contractor, after compacting the existing soil, used a 100 mm layer of GSB on which concrete pedestals of dimensions 800 mm × 800 mm and 200 mm thickness were placed. On these pedestals the trestles were mounted. The trestles were connected to the pedestals using 20 dia H.T. bolts. The trestles have dimensions of 600 mm × 600 mm in plan. A total of 9 trestles were used in a row, spaced at about 1,395 mm centers. In the longitudinal direction the trestles were spaced at 1,354 mm centers. The main members of the trestles were fabricated using ISA 65 × 65 × 6, while the lacing members were fabricated using ISA 50 × 50 × 6. The trestles were fabricated in modular units of 2.0 m in height.

For supporting the deck slab the contractor erected 32 NB pipes, with a footplate assembly at the bottom and a U-head assembly at the top. The pipe supports rested on the ISMB 150, which were supported on trestles.

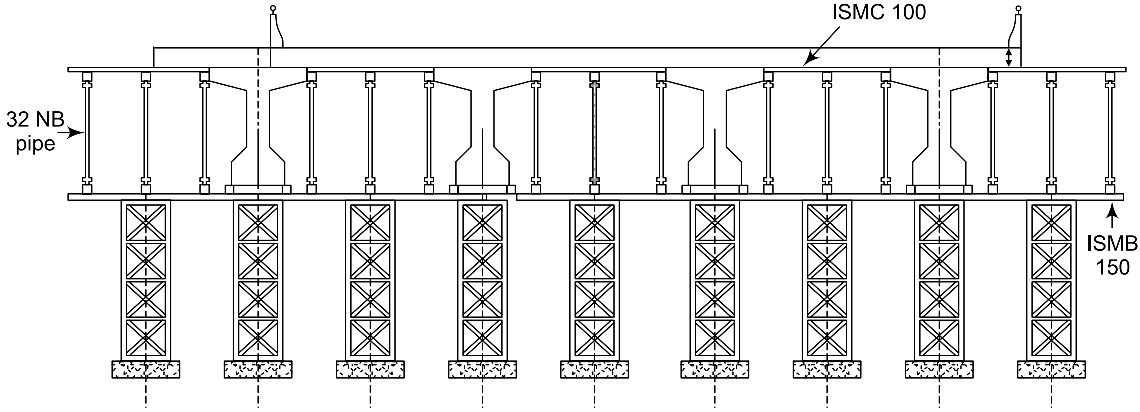


Figure 9.33 Cross Section of Formwork for Bridge Girders and Slab for a Major Bridge.

A more detailed view of the trestle supporting the girder and slab is shown in Fig. 9.34. ISMB 150 × 100 were used to transfer the loads from the girders and slab to the trestles. The forms for the bottom of the girders were fabricated in steel. These had timber packing resting on the ISMC 50 × 100. The ISMC's in turn were resting on the ISMB 150 × 100 mentioned earlier.

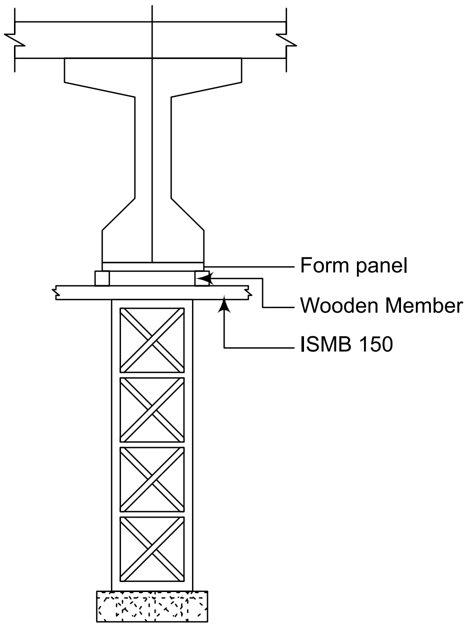


Figure 9.34 Detail of Trestle Supporting Girder and Slab Beam Formwork.

Figures 9.35 and 9.36 show another application of the trestle for supporting a girder and slab for ROB at chainage 124+115 on the Faizabad Bypass. The project is part of LMNHP. The side formwork for the girders consisted of steel sheathing of 3.15 mm and ISA 45 × 45 × 5 angle sections. The stiffeners were angle sections of ISA 45 × 45 × 5 at 250 mm centers. The bottom forms for girders were fabricated with 3.15 mm steel plates and ISA 45 × 45 × 5 angle sections. The stiffeners were also made up of angle sections of ISA 45 × 45 × 5 but they were kept at a spacing of 175 mm centers. The tie rods used were 20 mm in diameter and they were spaced at 1,100 mm centers.

The details of the trestles are shown in Figs. 9.37 and 9.38. The trestles were fabricated in modular units of height 3.0 m. ISA 65 × 65 × 6 sections were used to fabricate the vertical members and ISA 50 × 50 × 6 sections were used as lacings for the trestles. The individual trestles were braced with box sections, made up of two ISA 50 × 50 × 6 sections.

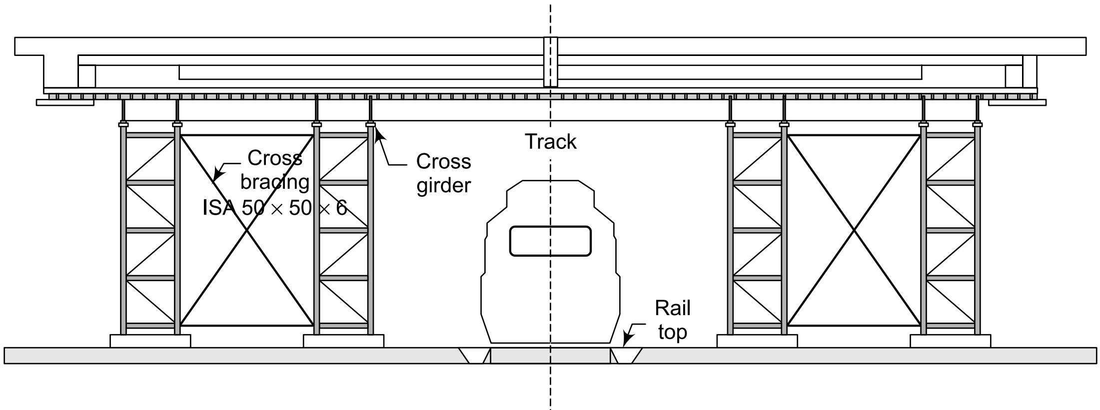


Figure 9.35 Arrangement of Trestles for Supporting Girder and Slab for ROB.

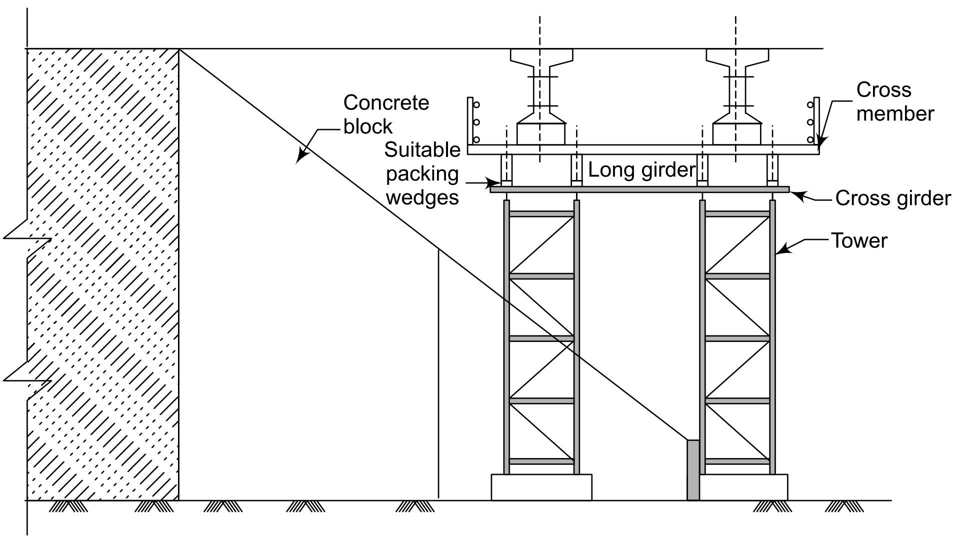


Figure 9.36 Another View of Trestles Arrangement for Supporting Girder and Slab for ROB.

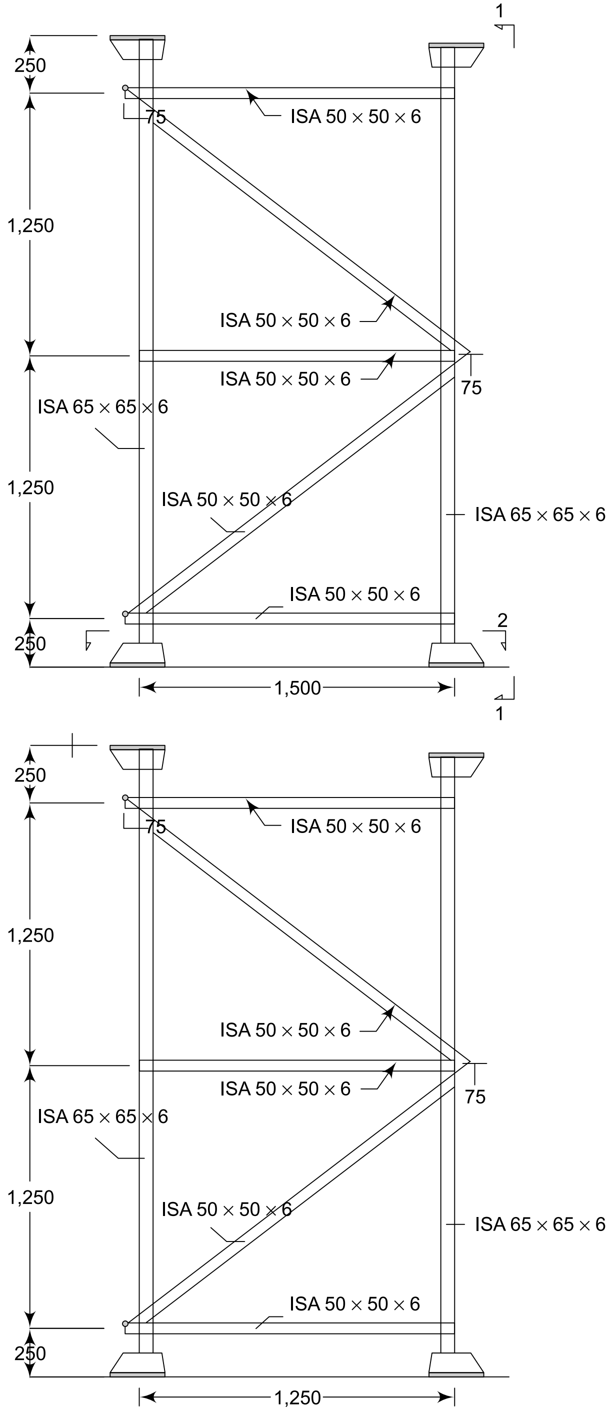


Figure 9.37 Details of Tower Modules 3 m High.

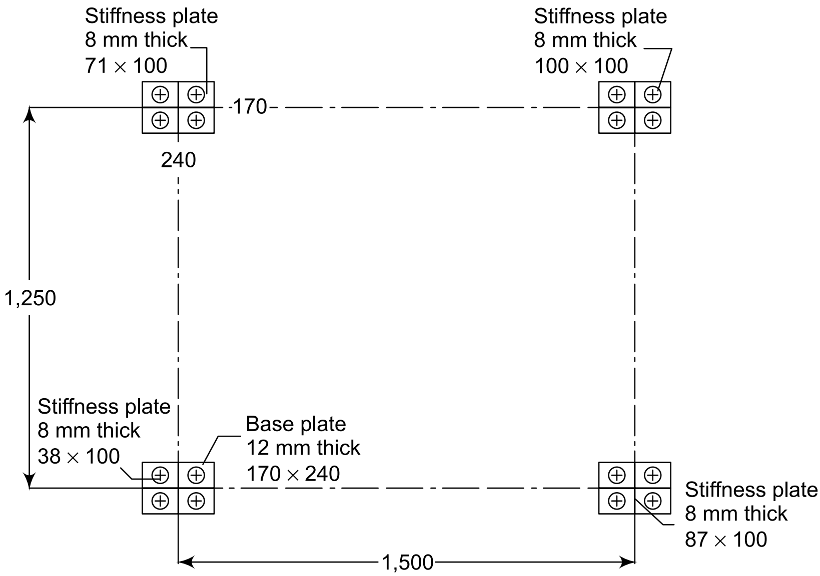


Figure 9.38 Another View of Tower Module.

The necessity of providing such heavy trestles arose from the fact that large clearances were required so that the railway track underneath the ROB could function, even whilst the overhead work was in progress. The railway track is visible in plan and section represented by Figs. 9.35 and 9.36 respectively.

9.4.3 Deck Forms Supported from Girders

When it is impractical to support the form on the ground the deck forms can be supported from a pier, girder, or some other part of the bridge superstructure. The form arrangement in which the deck form is supported from girders is discussed in this section.

The formwork for the deck slab can be supported on the *cast-in-situ* or precast girders as shown schematically in Fig. 9.39. The precast girders have been used to support both the deck slab form and walkway arrangement to fix deck slab. The deck slab form is made up of standard 3.15 mm steel sheathing and an ISA 45 × 45 × 5 mm angle frame. The stiffeners used are also ISA 45 × 45 × 5 sections at 250 mm centers toe welded. The framework to support the deck slab form consists of structural steel sections, ISMB 125 × 70 and 2 ISMC 75 × 40.

The walkway arrangement consists of 2 ISMC 75 × 40 at a distance of 1,250 mm centers. The entire walkway arrangement is supported from girders with a suspended 16 mm diameter steel rod. The walkway arrangement is used to fix the forms for the deck slab by the formwork crew.

Figure 9.40 shows the deck slab formwork arrangement adopted for a bridge project using L&T Flex system components. The sheathing material is plywood, which is supported on H-16 beams. The H-16 beams are supported on walers fastened to the U-head of short props. The shorts props in turn transfer the loads to another waler which is supported from the lower part of the I-girders.

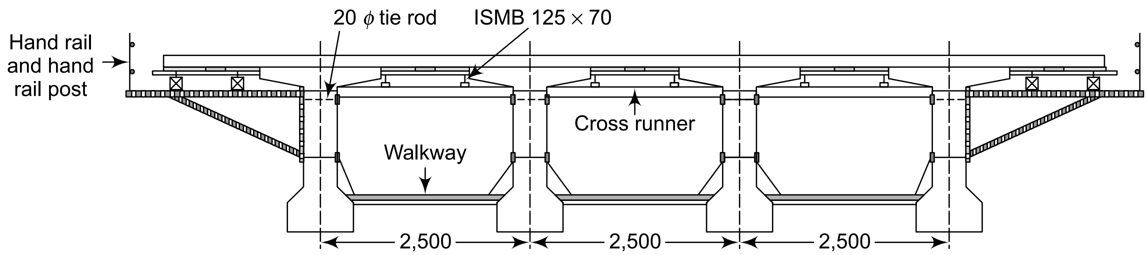


Figure 9.39 Formwork Arrangement for Deck Slab Supported on the Girders.

Figure 9.40 Formwork Arrangement for Deck Slab.

9.5 FORMWORK FOR BRIDGE RAILINGS/PARAPETS/EDGE BEAMS

Railings, parapets and edge beams are perhaps the most neglected bridge elements in bridge construction. After the completion of major elements such as the well, piers, girders, and the deck slab, the contractors feel relaxed and thus little attention is paid to the casting of such elements. The result is that one gets an ugly appearance from the parapets, railings, and edge beam. This also results in the early deterioration of these bridge elements. In this section some commonly used formwork arrangement for railings, parapets and edge beams are discussed.

9.5.1 Forms Accessible from Only One Side

Figure 9.41 shows a typical formwork arrangement for casting the parapets/railings from the edge of a bridge. The form consists of steel sheathing stiffened with angle sections or flats. Top spacers assure a uniform thickness. Depending on the design, the top of forms may have chamfer elements to provide a chamfer to the finished concrete. The form sides are supported by jacks on one side only.

As there is no access to the outside forms, and all arrangement for the of alignment, etc. are performed from the inside only, the resultant concrete is not up to the mark. In most cases the alignment of the exposed concrete is not satisfactory. The lack of access to the outside shutter means that any leakage of slurry from the outside cannot be stopped, and thus the exposed concrete presents a great deal of honeycombing.

Figure 9.41 Formwork Arrangement of Parapets for a Typical Flyover/Bridge Project.

9.5.2 Forms Accessible from Both Sides

In an improved version of the formwork, arrangements are made for an outside platform supported on steel brackets. These brackets are supported from the bridge girders and there is no need to take the support all the way from the ground. This addresses the problem in the previous system. The provision of an outside walkway platform ensures the application of a regular wall formwork. The forms can be made out of plywood and timber, or can be made up of steel plates and angle stiffeners. The arrangement of such formwork for a parapet/railing is shown in Fig. 9.42.The finish and alignment of the exposed concrete in this form of arrangement is definitely superior than that of the previous system.

9.5.3 Precast Railings/Parapets

In another superior arrangement, the railings/parapets are cast in a precast yard in a controlled environment. These are brought to the site and erected with the help of a crane. This results in a superior finish and very good aesthetic appearance. One such application of precast railings/parapets is shown in Fig. 9.43. After the erection of segments of precast railings/parapets some *cast-in-situ* concrete may be required, as per the design requirements. Until the concrete segments are fixed permanently, they need support which is also shown in Fig. 9.43.

Figure 9.42 Formwork Arrangement (Accessible from Both Sides) of Parapets for Flyover in Delhi.

Figure 9.43 Temporary Support for Precast Parapet.

The parapet walls for all the elevated corridor of the Delhi Metro (Fig. 9.44) were constructed using precast segments. Besides offering a good concrete finish, and proper alignment, precasting also saved considerable time for the different contractors engaged in the Delhi Metro project. It is possible to get different textures and designs for aesthetic appeal in precast arrangements.

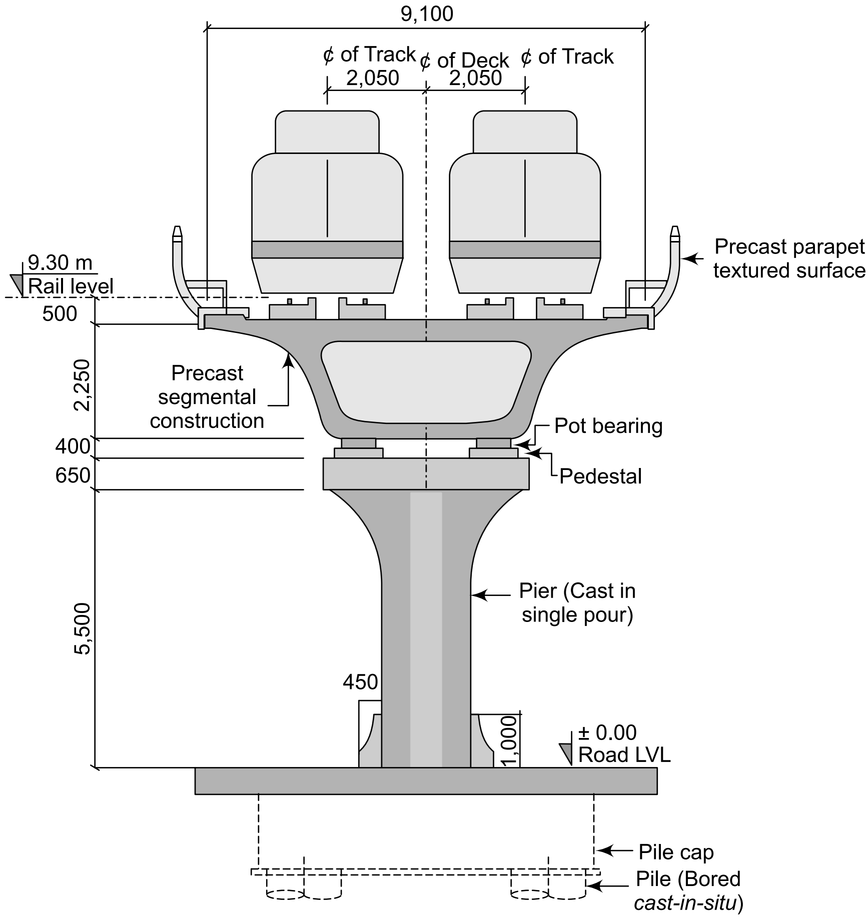


Figure 9.44 Precast Parapet Textured Surface.

9.5.4 Proprietary System

The problem with the above mentioned formwork systems is that they are 'use-and-throw' types. Different applications require different arrangements of shutters and different form designs. In a proprietary system, with changes in spacing and components, they can be used for a variety of parapet/railing dimensions.

Doka offers an efficient solution for forming the bridge edge. The system is known as bridge edge beam formwork T. The system is made up of a very small number of different parts and can be assembled quickly and dismantled manually. The system is lightweight but can carry loads of high intensity. The system can be used to form different dimensions of cantilevered parapet for different types of bridges. The system is used most efficiently on shorter superstructures, medium-length bridges with a small number of repositioning cycles, and bridges with tight radii and complicated cross-sections. A schematic diagram of the formwork system is shown in Fig. 9.45 while its field application is shown in Fig. 9.46.

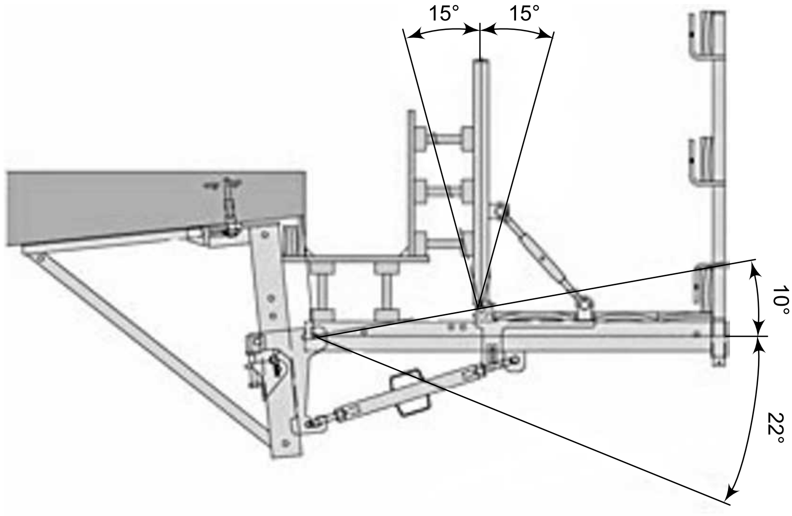


Figure 9.45 Schematic Sketch of Bridge Edge Beam Formwork T.

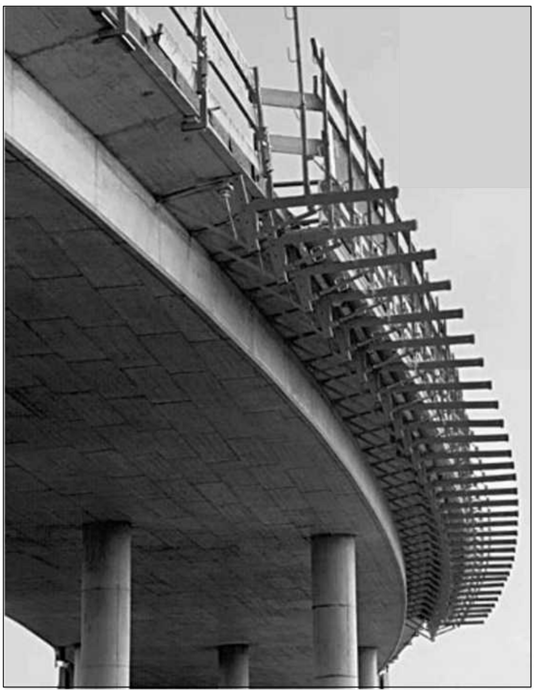


Figure 9.46 Field Application of Doka Edge Beam Formwork Solution.

9.6 CASES IN FAILURE OF TEMPORARY SUPPORT STRUCTURES OF BRIDGES

In the following sections we discuss some cases of temporary support structure failure. Though these cases may not strictly be called formwork failure, we can categorize them in the failure of temporary structures.

9.6.1 Failure of a Launching Girder at Laxmi Nagar in the Delhi Metro Project

The failure of a launching girder (Fig. 9.47) is attributed to improper fabrication of the launching girder. The holes for bolts were made with a gas cutter and therefore not drilled properly. The number of bolts was also less than that required, besides an insufficient contact area. All this led to stress concentration leading to the ultimate failure of the launching girder.

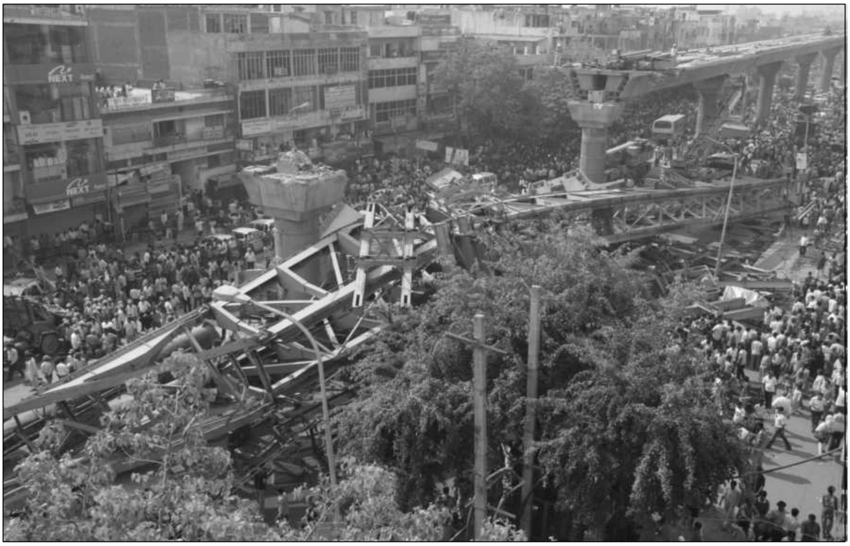


Figure 9.47 Collapse of Launching Girder in Laxmi Nagar.

9.6.2 Failure of Cantilever Portion of Pier Cap and Deck Slab Failure

The entire span between two piers for an under-construction metro bridge collapsed in July 2009 in Zamrudpur, New Delhi (see Fig. 9.48). This is a classic case in which a number of causes were responsible for the failure.

(a) The design was not correct.
(b) The reinforcement detailing had a major deficiency. The critical areas lacked the desired quantity of reinforcement
(c) The work on site proceeded based on 'advance copies' not validated by the concerned people.
(d) The implementation on site was not as per the provisions of the drawings, and several deviations were noted.
(e) Above all, the strength of the concrete failed in strength criteria by a wide margin.

The most unfortunate part in the whole episode was that a large number of cracks were noticed prior to the failure. The investigators of the said cracks did not understand the nature of the cracks and did not recommend proper, judicious corrective measures.

Figure 9.48 View of Zamrudpur Under Construction Metro Bridge Span Collapse.

9.6.3 Collapse of Prefabricated Segment in the Hyderabad Flyover

Eight pre-fabricated segments of the flyover collapsed after scaffolding caved in on September 9, 2007 (Figs. 9.49 and 9.50). The accident resulted in a large number of fatalities and injuries. The contractor erected the temporary structures (trestles) on concrete pedestals. The pedestals were partly on compacted soil and partly on not–so–properly compacted soil. The not so properly compacted soil was above the pipe line (going along the flyover) which was recently laid by the municipal authority. Heavy rains resulted in a high velocity of the flow of water, due to which the concrete pedestals were disturbed and eroded. As some of the concrete pedestals' supporting trestles were partially resting on the original hard ground and partially on a trench, differential settlement and tilting of the trestle concrete pedestals took place, which ultimately resulted in the collapse of the prefabricated segments.

9.6.4 Toppling of Prestressed Girder during Construction at a Major Bridge on Banganga River

During the construction of a major bridge on the Banganga River near Bharatpur, one prestressed girder (Girder No. 30) fell from its cast position to the ground, killing two people. One end of the Girder was resting on abutment A2 while the other end was resting on Pier 7. The author was a co-investigator of this accident.

Figure 9.49 View of the Accident Site (Photo Courtesy www.chinapost.com.tw).

Figure 9.50 Another View of the Accident Site (Photo Courtesy www.thehindubusinessline.com).

During the investigation it was found that the toppled girder was *cast-in-situ* on July 21, 2008 and the formwork was still in place. The girder was prestressed to design level on July 30, 2008 and the ducts for prestressing tendons were grouted on July 31, 2008. The F4 type of bearing (of sliding POT/PTFE type) was in locked condition on abutment A2. The F3 type of bearing (longitudinal guided sliding POT/PTFE type) was fixed in locked condition on Pier P7.

A ramp consisting of two channels was placed at some distance near the abutment A2. One end of the ramp was supported on the ground while the other end was on top of girder no. 30. The ramp was used for shifting the reinforcement for the casting of a diaphragm wall between girder nos. 29 and 30. On the day of accident, a vehicle carrying some workers hit the ramp. Immediately after this the girder no. 30 fell to the ground and the vehicle got trapped under the girder. The accident resulted into two fatal injuries: one labor working near girder no. 29 and the driver of the vehicle which hit the ramp) and a few other non–fatal but serious injuries to those who were in the vehicle.

The investigators noticed that there was no adequate provision of laterally supporting the girders prior to the casting of the diaphragm wall and deck slab, and that it was supported using different means such as wire ropes, timber packing and so on.

The girder was found to be unstable due to the following:

(a) The bottom flange of the girder is smaller than the top flange, and the girder is top heavy. As a result the centre of mass is considerably higher than the vertical axis providing a large lever arm in the case of any toppling effect. Besides, the flange width being small at the bottom, the required resisting moment against any toppling effect is also small. Generally, as a safety measure, such girders are held in position with lateral supports throughout its length, which in this case was inadequate.

(b) Proper bearing of the girders at the ends was not provided due to constructional inaccuracies. This is likely to cause eccentricity in the system resulting in destabilizing moments and thus toppling. Furthermore, the bearing plate, although fixed to the bottom of the girder, was not fastened at the Jaipur end, while the girder was not resting on bearing plates at the Pier 7 end.

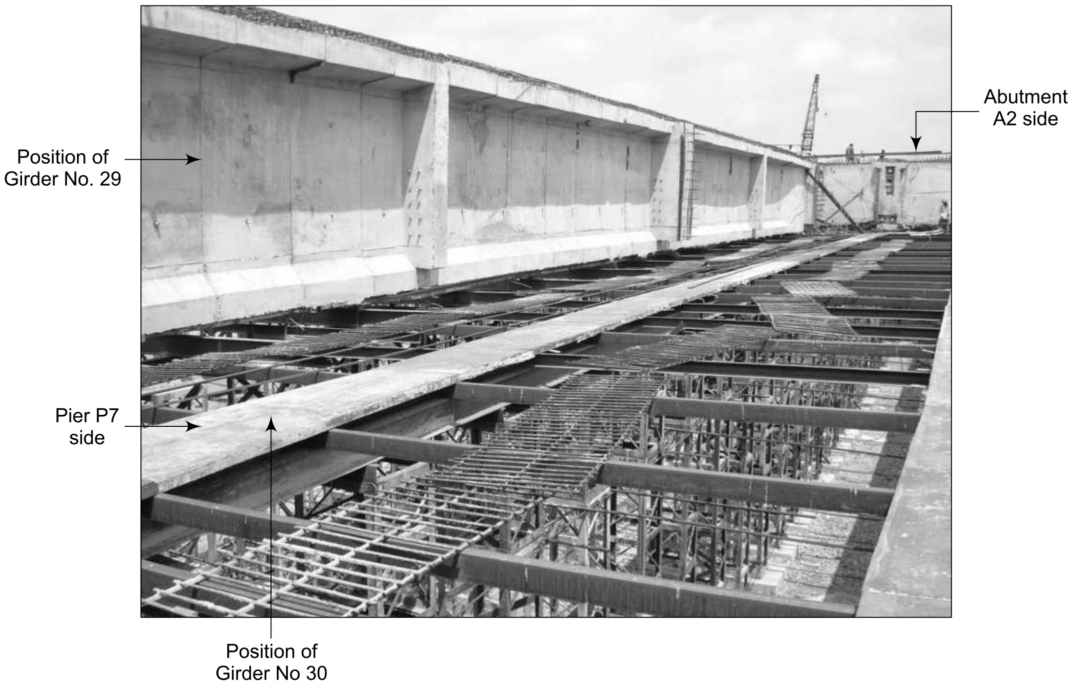


Figure 9.51 View of Dislocated Shutters.

Figure 9.52 Toppled Girder Lying on Ground.

(c) As a result of the prestressing, the central portion of the girder is lifted as a bow and therefore the contact at both the ends (supports) must have been only on the edges instead of full surface contact area—practically only providing line support.

The horizontal load, on account of the impact made on the ramp by the vehicle, caused girder no. 30 to slide. The investigators recommended the provision lateral support to the girders until the time the diaphragm walls at all locations and the deck slab are cast in a span. Some possible schematic support arrangements are shown in Figs. 9.53 and 9.54, which can be adopted in consultation with the designers.

9.6.5 Toppling of Girders in an Under Construction Viaduct Bridge at Faizabad

Three girders fell from their cast positions at Rauzgaon Viaduct Bridge in January 2010. It appears that the girders fell when the boom of a mobile crane lifting some material hit a girder. The affected girder hit the second girder which in turn hit the third girder and all three girders fell to the ground.

The investigators remarked that free standing girders is a very unstable state and prone to such accidents, even after the locking of bearings as in the present case. They suggested the following to avoid such accidents in the future.

(a) The method statement for such works should be such that the time period between the placement/construction of individual girder on bearings and the construction of the deck slab should be minimized. This involves the preferance of the precast construction of girders over *cast-in-situ* type girders, together with first stage prestressing that allows immediate casting of cross girders and the deck slab before final prestressing is done.

(b) The use of seismic arrestors on the pier cap should be preferred irrespective of the seismic zone in which the construction falls. The use of seismic arrestors will provide some additional safety measure and would prevent such a failure.

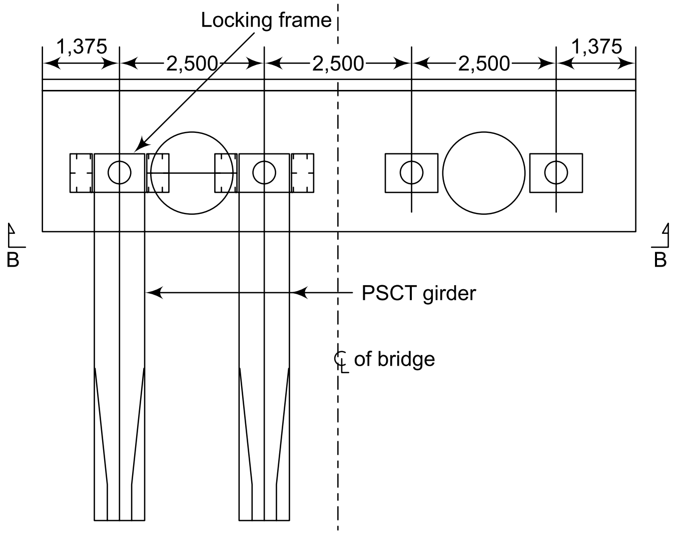


Figure 9.53 Part Plan For Girder Restraining Arrangement (Restraining arrangement at the other end of girder is same and thus not shown.)

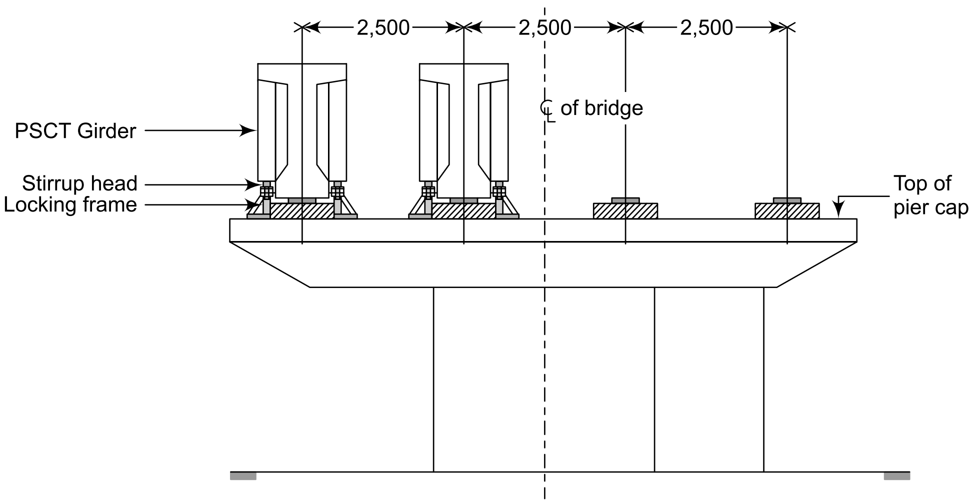


Figure 9.54 Sectional view BB of Restraining Arrangement.

To avoid such accidents in the future, the investigators suggested the following sequence of construction:

Step 1: Cast the first girder.

Step 2 : After 7 days, remove side shutter and bottom shutter around the bearing area only.

Step 3: Erect support system and lock with bearing pedestal using wooden wedges/packing.

Step 4: Lock girder with packing between support system and girder.

Step 5: Tighten stirrup head on both sides together. Stirrup heads are nominally only tightened to ensure that the stirrup heads are in proper contact with the diaphragm soffit.

Step 6: Prestress the girder to stage 1 and stage 2 and grout the cable as per specifications.

Step 7: Cast the second girder. Repeat steps 2 to 6.

Step 8: Weld top two reinforcement bars and bottom two reinforcement bars of the two girders immediately after the girder is cast.

Step 9: Similarly, complete the third and fourth girders. All four girders shall be interconnected by welding diaphragm reinforcement.

Step 10: Erect diaphragm staging and shuttering as per approved drawings.

Step 11: Concrete deck slab and diaphragm.

Step 12: Remove the supporting system after the specified period of deck concreting.

REVIEW QUESTIONS

Q1. True or False

(a) Substructure consists of the foundation of bridge which supports two or more piers joined together at the top by a pier cap.

(b) Foundation type is dependent upon the span length of bridge.

(c) Foundation type is dependent upon the location of bridge.

(d) Foundation type is dependent upon the bearing capacity of material on which the foundation rests.

(e) Caissons are hollow structures sunk to a very high depth below ground or the river bottom.

(f) The three broad parts of caissons are cutting edge, curb, and wall.

(g) Piers are important for 2 cases viz. relatively lesser heights and large heights.

(h) Slipform and climbing formwork are two major formworks for piers with large heights.

(i) Major failures of temporary support structures of bridges are: (i) improper design (ii) deficiency in reinforcement detailing (iii) concrete strength failure (iv) improper implementation at site.

Q2. Match the following:

(i) Cutting edge	(a) air jetting reduces the skin friction on the outside periphery of the wall
(ii) Curb	(b) assists the caisson in sinking below the ground or river bottom
(iii) Wall	(c) inside shutter is fixed, aligned correctly and supported by suitable props. Curb reinforcement is fixed and starter bar is welded with cutting edge.

Q3. Match the following:

(i) Short span bridges	(a) Girder and slab arrangement
(ii) Long span bridges	(b) Simple slab arrangements

Q4. Write short notes on:
- (a) Pier formwork construction for DMRC
- (b) Pier with slipform
- (c) Pier with climbing form
- (d) Formwork for pier cap.

Q5. Write short notes on:
- (a) Deck slab and girder supported by tubular steel scaffolding
- (b) Deck forms supported by steel cribs/trestles
- (c) Deck forms supported by girder

Q6. Enunciate the following in the context of formwork for bridge railings /parapets/edge beams
- (a) Forms accessible from only one side
- (b) Forms accessible from both sides
- (c) Precast railings /parapets.
- (d) Proprietary system bridge edge beam formwork.

Q7. Write short notes on:
- (a) Failure of prefabricated segment in Hyderabad flyover
- (b) Failure of launching girder of DMRC in Laxmi Nagar area
- (c) Toppling of girders in an under construction viaduct bridge at Faizabad.

Chapter

10 Flying Formwork

Contents: Introduction; Some Examples of Flying Formwork; Flying Formwork Cycle; Advantages and Limitations of Flying Forms; Design Issues in Flying Forms; Safety Issues in Flying Forms; Table Forms; Tunnel Formwork System; Column Mounted Shoring System; Gang Forms

10.1 INTRODUCTION

The flying form is a system of formwork which is assembled into form units usually on the ground and is located to form concrete elements at the site location. The form units are further relocated at a new location with virtually no disassembly of parts to form concrete elements. The term flying formwork is used because forms are flown from one story to another with the help of a crane. The flying formwork can be used to cast various concrete elements such as concrete beams, girders, slabs, shear walls, etc. The flying form is one of the most commonly used formwork system even though the initial cost of fabrication for flying form is high as compared to the hand set form. The flying forms are more common in high rise buildings for rapid cycle construction wherein large repetitions are possible, thus justifying their relatively high initial cost of fabrication. The flying form is also supposed to save material and labor costs.

A typical unit of flying formwork consists of sheathing member (mostly plywood), trusses or shores, steel or wood beams as stringers, joists; same as in the handset forms. Joists are timber member or aluminum beams connected in most cases to deep trusses. In case of flying forms, all members are connected together to allow lifting of the whole system in one piece which is in contrast to the handset forms. This also prevents the possibility of joist or stringer uplift at their supports.

There is a vast range of flying formwork systems available today such as the tunnel forms, the flying truss systems, the column mounted shoring systems by different formwork manufacturers. The various flying formwork systems are available today either on purchase or on rental basis.

In the following sections, we will discuss some of the flying formwork systems by a few manufacturers. The various steps involved in the flying formwork cycle time are described briefly. The advantages and limitations of the flying formwork system are pointed out. We will also discuss some design and safety issues involved with the flying formwork system. The mentioned systems are briefly discussed for some of the manufacturers.

The flying form modules supplied by a leading manufacturer comes in a length in excess of 30 m and width up to 15 m. In fact the actual sizes of the modules may be limited only by the capacity of the crane which will hoist a unit. By combining modules, components, accessories, and special parts, it is possible to assemble the form systems suitable for use in casting many concrete shapes. One typical large flying formwork is shown in Fig. 10.1.

Figure 10.1 Large Flying Formwork Panels.

10.2 SOME EXAMPLES OF FLYING FORMWORK

10.2.1 Interform Aluminium Flying Forming System

The system consists of light weight aluminum trusses and joists. The truss modules are delivered preassembled, which reduces on-site erection time and labor costs. The system is fully adjustable in terms of table width, length and height, and thus, is suitable for most job requirements. The system is ideal for use in high-rise construction with lots of possible typical repetitions and where a long span working platform is required. The interform aluminum flying forming system is shown in Fig. 10.2.

Figure 10.2 Interform Aluminium Flying Forming System in Use.

10.2.2 Symons Multiple-reuse Flying Truss System

This system is designed for specific project dimensions and schedules. The system consists of a series of reusable tables to support the deck. The tables are moved from one story to another during construction. The system claims substantially reduced labor costs due to the use of large table sizes and less involvement of crane time. The system is ideal for use in high-rise construction with repetitive configurations such as hotels, offices, hospitals etc. The Symons multiple-reuse flying truss system is shown in Fig. 10.3.

Figure 10.3 Symons Multiple-reuse Flying Truss System in Use.

10.2.3 Harsco Infrastructure Flying Table Form

Harsco Infrastructure also offers the flying table formwork. Figure 10.4 shows the various components used in this form system. In this system the deck is supported on ledger frames. These frames come in various sizes. Depending on the requirement, various sizes of flying form modules can be assembled and lifted through crane (see Fig. 10.5).The system has a provision of fixing the working platform and safety handrails for safe working at all heights and even on the floor edges (see Fig. 10.6).

Figure 10.4 Harsco Infrastructure Flying Table Form Being Assembled.

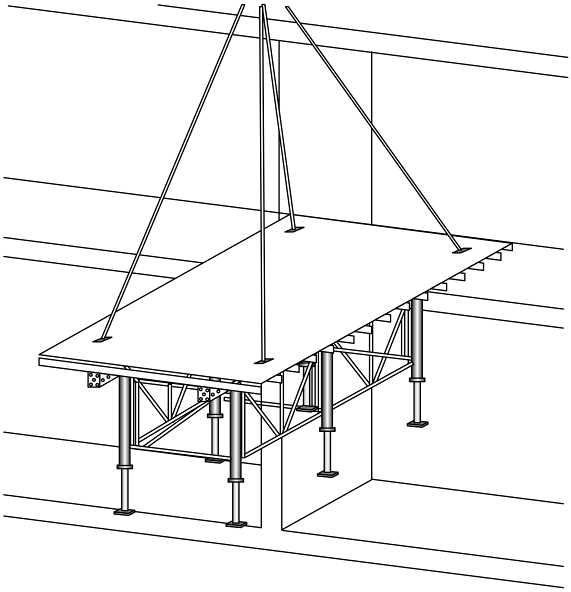


Figure 10.5 Harsco Infrastructure Flying Table Form Ready for Lifting Using Crane (Courtesy Harsco Infrastructure).

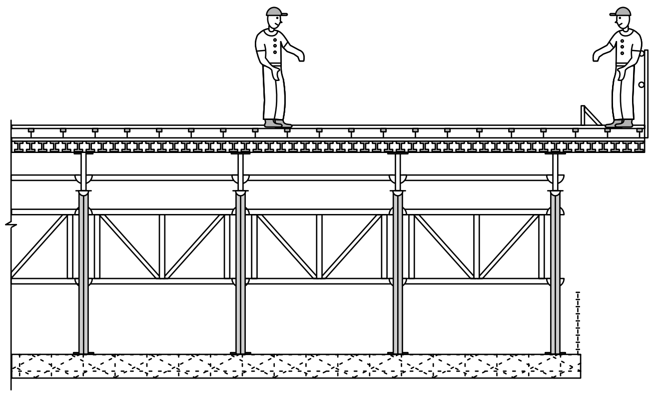


Figure 10.6 Harsco Infrastructure Flying Table Form Installed at Concreting Location (Courtesy Harsco Infrastructure).

10.3 FLYING FORMWORK CYCLE

The various steps in the flying formwork cycle are given briefly in the following paragraphs:

Step 1

The forms are preassembled into modules at the ground level (see Fig. 10.7). As mentioned earlier, the formwork system consists of the sheathing member (mostly plywood), trusses or shores, steel or wood beams as stringers, and joists. The materials for some of the flying formwork components may vary depending on the manufacturer.

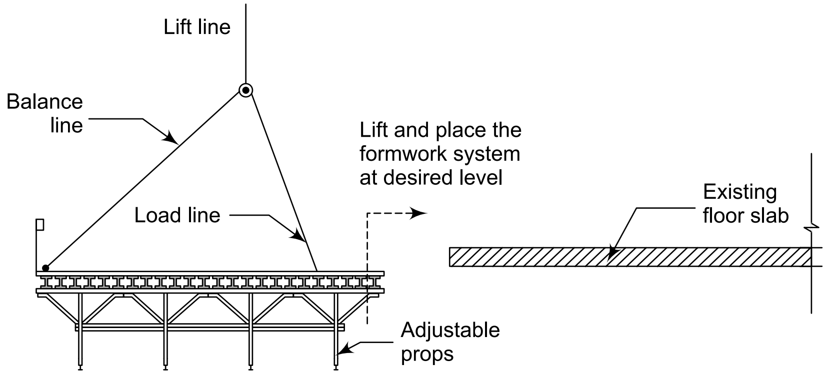


Figure 10.7 Forms Being Preassembled into Modules at Ground Level.

Step 2

The forms are moved into positions for concreting with the help of a crane. For internal shifting beyond the reach of the crane, moveable dollies are used as shown in Fig. 10.8. There may be some custom made fabrication required at the site location, especially over the concrete columns, to cover the space between the columns and the flying formwork components. The step 2 in a real project application is also shown in Fig. 10.9.

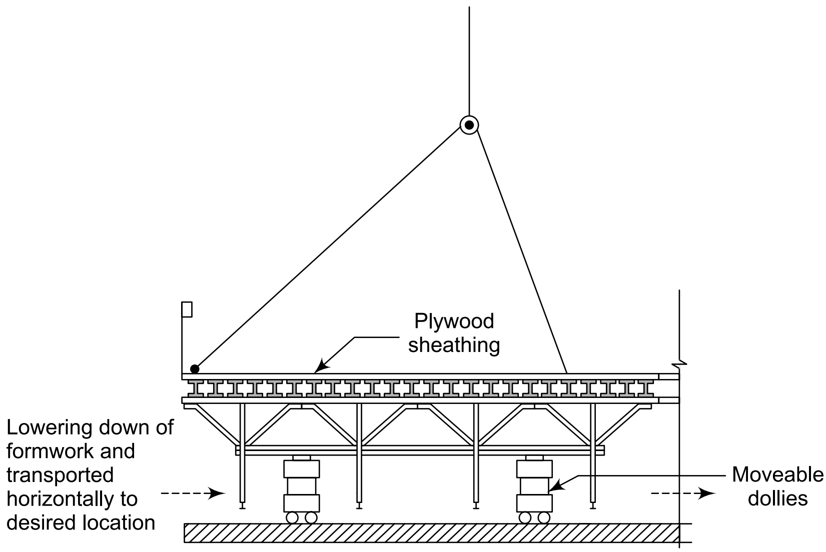


Figure 10.8 Forms Being Moved into Positions for Concreting.

Step 3

The elevation of each flying formwork module is adjusted to the correct height, and attached firmly to other modules. The reinforcing steel is fixed. The electrical, mechanical, and plumbing components, and any other services are also attended to at this stage. The concrete is placed as in case of any conventional formwork. The concreted slab is shown as the new slab in Fig. 10.10.

Figure 10.9 Positioning a Flying Form Table.

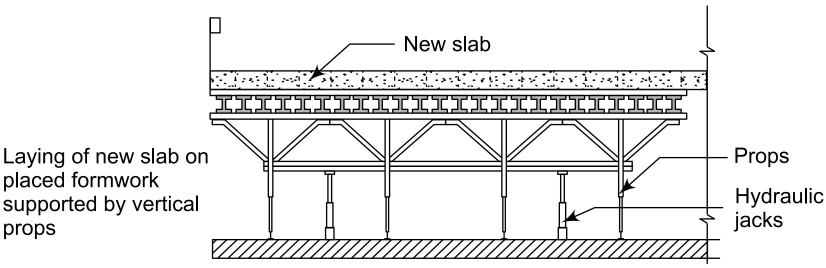


Figure 10.10 View Showing Completion of Casting While Forms in Position.

Step 4

After the concrete has attained sufficient strength, the stripping of forms begins. For this, the flying formwork is lowered by means of an adjustable jack arrangement and retracted to clear the concrete element cast recently. This is shown in Fig. 10.11.

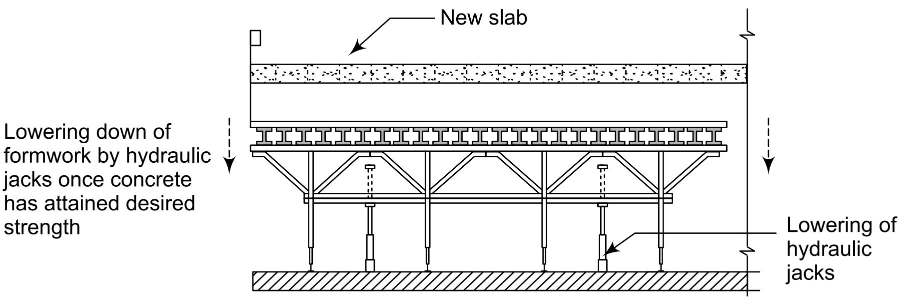


Figure 10.11 Stripping of Forms.

Step 5

The forms are moved to the edge of the supporting concrete slab with the help of transportation devices such as moveable dollies. This is shown in Fig. 10.12.

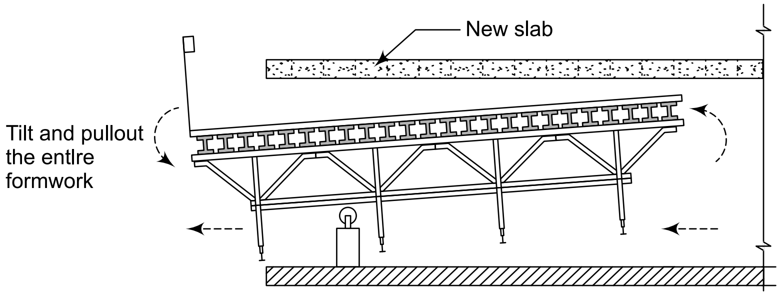


Figure 10.12 Shifting of Forms.

Step 6

The form tilts when about half the form panel is out of the floor, as shown in Fig. 10.13. At this point of time, it may even touch the roof slab cast recently. The form panels are now hoisted to a higher floor for reuse. It may be noted that the recently placed concrete slab should be reshored immediately as per the advice of the design engineer before any live load is placed on it.

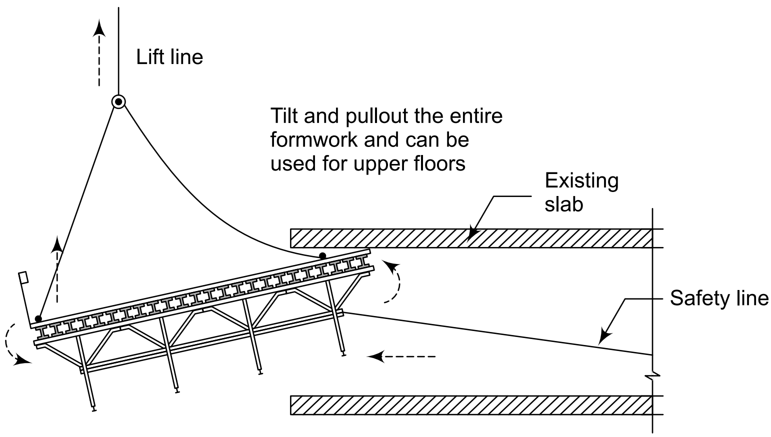


Figure 10.13 Tilting of Form When About Half the Form Panel is Out of the Floor.

Step 7

The forms are cleaned and the release agent applied. In fact, some of the practitioners point out that cleaning the flying forms between floors and applying a release agent is one of the most arduous tasks faced by construction crews in the flying formwork operation.

The steps mentioned above are repeated as per the requirement.

In some of the flying formwork systems, only the trusses are flown from one story to another and the tables are assembled on each floor separately as in the case of the conventional formwork. This situation may arise on account of site limitation and varying configuration of the bay at different floors.

10.4 ADVANTAGES AND LIMITATIONS OF FLYING FORMS

Some of the advantages of flying forms are given below:

1. Since the fabrication of the formwork takes place on the ground using labor saving equipment and methods, the productivity obtained in flying formwork fabrication is higher as compared to the fabrication of the conventional hand set formwork. Also, since the flying formwork need not be disassembled as in the case of a conventional formwork, there is a considerable saving in the stripping costs also.
2. Although there is higher initial fabrication cost involved with the flying formwork, the higher cost gets offset with the large number of repetitions associated with the flying formwork.
3. There is considerably enough working space available below the flying formwork which allows other construction activities to proceed.
4. The cycle time for the flying formwork is considerably reduced thus helping in completing the project in less time i.e. it reduces the project duration. A five day construction cycle is quite easily achievable using the flying formwork system and thus, the cost of the formwork can be reduced substantially below the cost of the conventional formworks.
5. Large area of the formwork can be handled easily in one go and thus it reduces the crane time.

Some of the limitations of flying formworks are given below:

1. Large flying formworks are difficult to handle especially in windy weather conditions.
2. The flying formworks are easy to work with, when the building has an open façade. In case there is no open façade, there is difficulty in shifting the flying formwork although some manufacturers have overcome this limitation by developing collapsible flying tables.
3. The flying formwork is difficult to work with, for flat slab with drop panels around the column locations.

10.5 DESIGN ISSUES IN FLYING FORMS

Flying forms must always be designed by a professional engineer and constructed, hoisted, moved, and set strictly according to the instructions of the designer or the manufacturer. Using forms designed for typical floors in non-typical situations has resulted in serious accidents. Before using any flying form under non-typical conditions, consult the designer or the manufacturer. Wall forms should not be extended in height or width, for instance, or slab panels cantilevered without professional consultation. Such situations usually occur with penthouses or mechanical rooms where wall and ceiling heights are greater than for typical floors.

10.6 SAFETY ISSUES IN FLYING FORMS

In Table 10.1, the various operations associated with the flying formwork are identified along with the hazards associated with those operations. The possible remedies corresponding to the given hazards are also suggested.

Table 10.1 Operations, Hazards, and Remedies for Flying Formwork

Sl. no.	Operation	Hazards	Remedies
1.	Initial fabrication of flying formwork.	Trusses and wall panels have also been blown over by wind during fabrication and dismantling. Working in cramped conditions.	Temporary bracing or temporary support by a crane may be necessary to ensure stability during certain phases of the operation. Providing adequate space for stacking materials and flying formwork components.
2.	Installing, pushing a panel out toward the slab edge, receiving a panel in from the slab edge, helping other workers attach rigging hardware such as slings, getting on and off, bolting and unbolting wall forms for exterior walls and elevator shafts, stepping onto a panel to attach slings to pick points.	Falling from height.	A fall-arrest system should be used by any worker involved with the mentioned operation. Each worker's fall-arrest system must be attached to an individual anchor independent of the flying form. Contractors can provide for anchorage by casting rebar anchors in columns or other areas to be covered over or filled in later.
3.	Hoisting and moving.	Forms are often swung out over sidewalks and streets.	The most efficient protection for workers is to rope off the area below to prevent anyone from entering the area. Pedestrian traffic on sidewalks, as well as vehicle traffic if necessary, should be detoured around the area while hoisting is under way. Direct radio communication between work crew and crane operator along with the hand signals.
4.	Concreting.		Before concrete placing begins, formwork must be inspected and signed off by the designer or a competent person to ensure that it has been constructed to provide for worker safety and to meet job specifications.
5.	Stripping—probably the most hazardous operation in concrete construction.	Hazards include – falling material – material and equipment underfoot – manual handling of heavy or awkward forms, panels, and other components – prying forms loose from concrete presents risk of overexertion, lost balance, and slips and falls.	Hazards can be reduced by – planning and providing for stripping when designing and constructing formwork – supplying facilities and equipment for removing materials as they are stripped – providing proper tools and adequate access for the stripping crew – training the personnel properly for this and the other aspects of the formwork.

Some of the do's and dont's while using flying form are given below: (Source Harsco Infrastructure)

DO's

1. A detailed method statement should be prepared while using the flying form. The site specific risks assessment involved with flying formwork operations should also be carried out.
2. It is recommended to use trained and dedicated crews for flying formwork operations. The crew members should be familiar with the method statement.
3. The safety rope must be attached between the flying form modules and suitable parts of permanent construction such as a reinforced concrete column.
4. The suspended parts of the flying form modules should be properly secured to prevent them from falling out.
5. The bolted connections wherever they are in the flying form modules, should be secured.
6. The slings of the crane must be attached to the flying form modules at correct positions specified by the manufacturer. The flying form modules should not be pushed out beyond safe distances specified by the manufacturer.
7. While lowering the flying form modules, all the legs should touch the floor simultaneously.
8. Flying form modules should be moved only on the trolley units specified by the manufacturers.

DON'Ts

1. The flying form modules should not be moved on an incline or near an exposed edge unless a Safety Line is attached.
2. Untrained crews should not be used for any operation involving striking, moving, flying and lowering of the flying form modules.
3. The flying form modules should not have any loose objects or unnecessary materials loaded on them during flying.
4. Any personnel riding on the flying form modules should not be allowed during flying.
5. The flying form modules should not be flown inclined at an angle.
6. The flying form modules should not be landing heavily onto the slab.

10.7 TABLE FORMS

Table form is another variant of flying form. The table form is also equally capable of providing very high speed of construction. The system is primarily used for multi-story building (such as residential flats, hotels, hostels, offices and commercial buildings) construction works with regular plan layouts and long repetitive structures.

The table form is a large pre-assembled formwork. It consists of a form deck of fairly large area up to about 100 m^2 (often forming a complete bay of suspended floor slab) resting on adjustable shores or props. The props could be 4, 6, or 8 legged. The commonly used sheathing material is plywood, although steel sheathing is also used. As in the case of other variants of the flying formwork, the whole table consisting of the sheathing member, primary beam etc. along with the supporting shores, is built as one unit and can be lowered or raised by means of mechanical or hydraulic jacks.

The formwork can be moved horizontally with the help of castor wheels or trolley units. Plumbing frames and concrete hardwares are used for the alignment of table forms.

The operational procedure using table form is exactly similar to what has been explained earlier. The process starts with assembling the table form at the ground level and then shifting to the location where the floor is to be cast. After aligning the formwork, the reinforcement is tied and provisions for other services are laid. Concrete is poured and allowed to gain strength. The formwork is struck with the help of jacks and without disassembly of the table form, it is rolled towards the façade (exterior) of the building. From here, the table form is picked up by a crane for locating it on the upper story.

10.7.1 L&T Table Formwork

The L&T table formwork uses the same components as used in its slab and beam formwork (known as Flex system). As in any table formwork, the formwork is not required to be dismantled after every concrete pour and a unit of the table formwork panel can be shifted at a time with the help of a crane from one location to the other. The major activities involved in the table formwork operations are: assembly of tables, shifting and erection of tables, aligning the tables, reinforcement and concreting work followed by the removal and shifting of tables to the new location. Figure 10.14 shows the table formwork being flown to its new location while Fig. 10.15 shows the table formwork being positioned. Figure 10.16 shows the transportation arrangement for table formwork. In the construction of a commercial mall at Mumbai (Fig. 10.17), the contractor used L&T table formwork extensively. The building has six floors consisting of a total slab area of 80,000 m^2. The contractor had mobilised 4,900 m^2 of table formwork. A twelve day cycle was achieved in the project. On an average, 2.25 man hours were consumed for making one m^2 of table formwork while an average 0.85 man hours were consumed in shifting and aligning the table formwork.

Figure 10.14 Table Formwork Being Flown to its New Location (Courtesy L&T Formwork).

Figure 10.15 Table Formwork Being Positioned (Courtesy L&T Formwork).

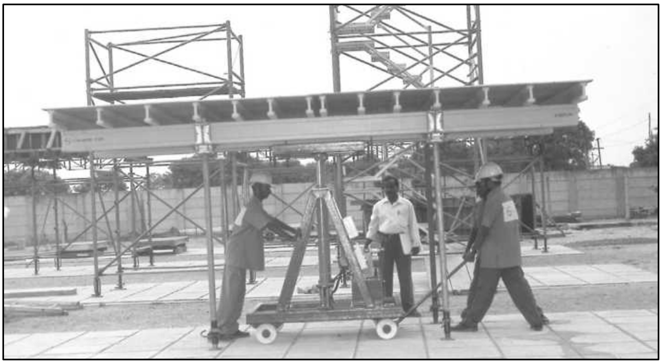


Figure 10.16 Transportation Arrangement for Table Formwork (Courtesy L&T Formwork).

Figure 10.17 Extensive Use of Tableform at Oberoi Mall Project Site Mumbai (Courtesy L&T Formwork).

10.7.2 PERI Table Formwork

PERI offers modular tables which are pre-assembled for immediate use. These modules are claimed to be cost effective for even small number of applications in a project. The modules are available in four standard sizes which allow optimal adjustment to the building. The sizes are:

Table Module VT 200/215 × 400
Table Module VT 250/265 × 400
Table Module VT 200/215 × 500
Table Module VT 250/265 × 500

With a special device called table swivel head, the props used for assembling the table modules can be folded in two directions (see Fig. 10.18). This makes the table modules easier to negotiate the parapets and guard rails (see Fig. 10.19). Besides, the table formwork can also be moved under down stand beams. The device also ensures low stacking and storage heights during transportation of the table formwork. Using the lifting fork, the table formwork can be quite easily shifted with the help of a crane (see Fig. 10.20). The table trolley is used to shift the table formwork on the base floor to the desired location (see Fig. 10.21).

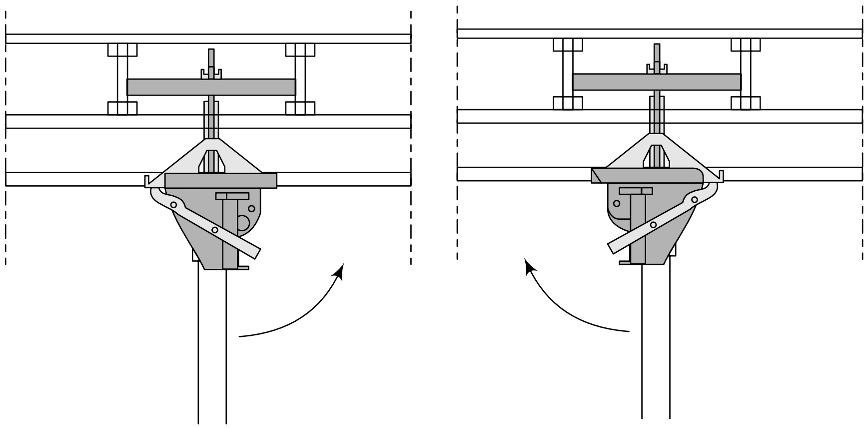


Figure 10.18 Table Swivel Head Used in Props (Courtesy PERI).

The Uniportal table formwork (see Fig. 10.22) of PERI is suited for projects with a large number of similar applications and open facades. In this system of formwork, the design of the table is customized to suit a particular structure such as a trapezoidal shape, circular shape etc. As discussed earlier, in this system also, there is a provision of foldable slab props.

PERI offers another table form system known as Skytable table forms. The system uses two trusses to support the table form components (see Fig. 10.23). The system uses multiprop slab props which allow a large lowering range so that the forms can be moved even under the beams. The system can be used both for large and small floor heights. The modules can accommodate slab area of up to 150 m^2.

Figure 10.19 Table Module Being Shifted to New Location (Note the Props in Folded Position) (Courtesy PERI).

Figure 10.20 Table Module Being Transported Using Crane (Note the Lifting Fork) (Courtesy PERI).

Figure 10.21 Table Form Being Shifted on Trolley to the Desired Location (Courtesy PERI).

Figure 10.22 PERI Uniportal Table Form (Courtesy PERI).

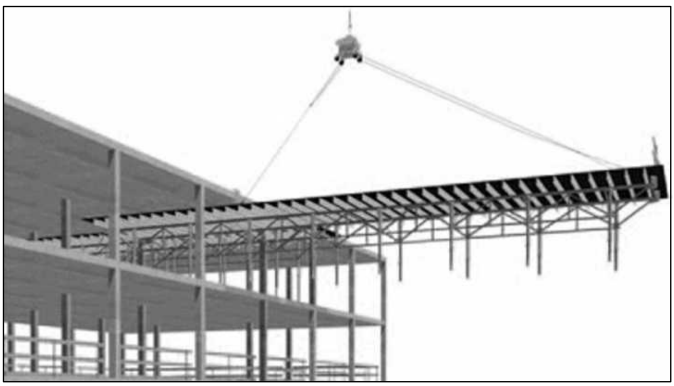


Figure 10.23 Skytable Form (See the Trusses) (Courtesy PERI).

10.7.3 The Advantages of Table Formwork

1. The speed of construction is very high for large floor layouts.
2. The productivity of the assembly of the tunnel formwork is high as it is carried out at ground location where the availability of the most suitable tools and tackles can be ensured.
3. At only a few locations, there is the need for infill areas (i.e. very less area of formwork is required to be fixed locally at site location) since the table form creates large bay sizes.
4. The transportation of large table form panels can be affected in one go rather than in piece mill and thus it ensures less crane time.
5. The quality of the finish achieved with table form is good when appropriate quality control is done.
6. Table form includes the arrangement for guard rails and edge protection.
7. The table formwork generates very less waste compared to the conventional formwork as large number of reuses are possible without the need of the table form being dismantled.

10.7.4 Limitations

1. There is a requirement of a large open space around the site so that the table forms can be lifted and transported easily.
2. A crane of adequate capacity is required for lifting the assembled table formwork. Also, there is a requirement of an open façade for easy transportation of the table formwork from one floor to another floor. These limitations however can be addressed by using a smaller table formwork which can be lifted even without the use of a crane and can be handled even in limited space availability.
3. The table formwork system requires detailed planning and is not suited when the site is expected to face high wind.

10.8 TUNNEL FORMWORK SYSTEM

Tunnel form is a formwork system through which it is possible to cast the walls and slabs in one operation on a daily cycle. The tunnel formwork is used to form the repetitive cellular structures such as hotels, residential buildings, hostels etc. It is possible to achieve speed without compromising on the quality. The system combines accuracy as in the case of a factory/off-site production with the flexibility and economy of *in-situ* construction.

The tunnel formwork system proves to be very economical for projects having 100 or more cellular units. The system requires great amount of planning and coordination with an architect, especially in the initial stages.

The tunnel formwork uses steel for all the components. The formwork system consists of the sheathing of a thick steel plate both for casting the wall and ceiling, waler, and the diagonal strut assembly. The tunnel formwork system is available either module wide (full-tunnel) or half-module wide (half-tunnel). The system is fitted with wheels to be rolled up and placed on the next story after the hardening of the concrete floor and walls. This system requires a high degree of repetition of standard types of spans, although many construction companies have been very inventive in providing some flexibility.

The two variants of tunnel formwork by Symons Corp. are shown in Figs. 10.24 and 10.25.

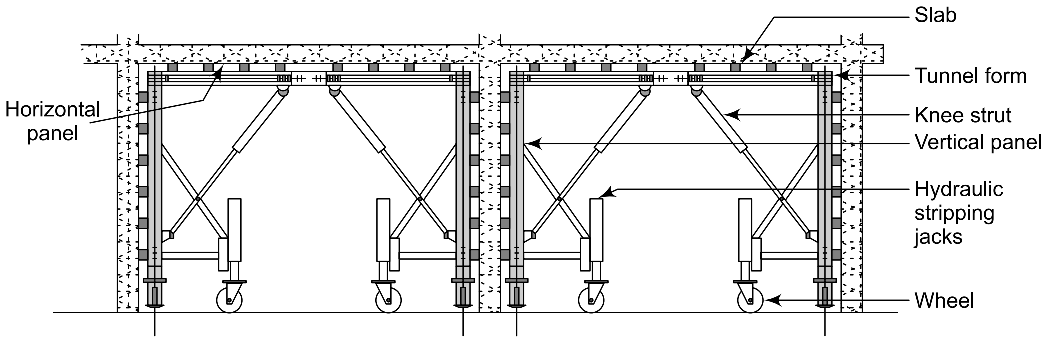


Figure 10.24 Tunnel Formwork System (Courtesy of Symons Corp. Available at http://www.symons.com).

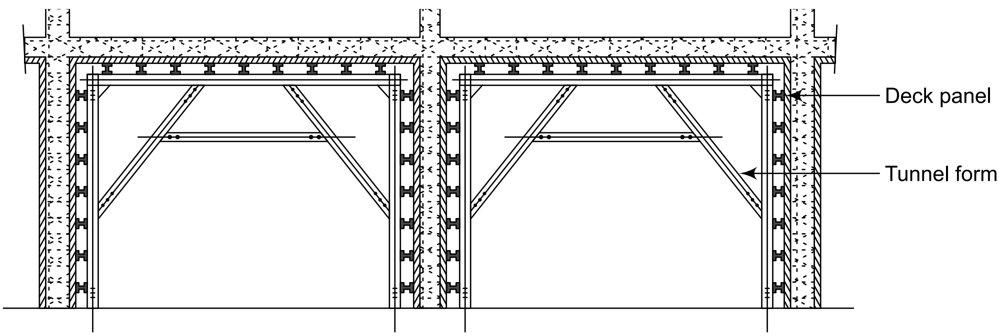


Figure 10.25 Tunnel Formwork System (Courtesy of Symons Corp. Available at http://www.symons.com).

The tunnel formwork system creates an efficient load-bearing structure for use in a wide variety of applications. It is particularly effective in projects suited for repetitive cellular construction such as the construction of residential blocks, hotels, student accommodation, barracks and prisons. The solid, strong monolithic structure can be 40 or more stories in height and the accuracy of the system suits the installation of prefabricated elements such as cladding panels and bathroom pods.

The steel tunnel forms create spaces spanning 2.4 to 6.6 m. These can easily be subdivided to create smaller rooms. Where longer spans (up to 11 m) are required, the tunnel form is extended using a mid-span section.

Tunnel form is a fast-track method of construction that is well suited for repetitive cellular projects such as hotels, apartment blocks and student accommodation. It is claimed that the cost and time can be reduced by 15% and 25% respectively using the tunnel form.

During the tunnel form construction process, a structural tunnel is created by pouring concrete into the steel formwork to make the floor and walls. Every 24 hours, the formwork is moved, so that another tunnel can be formed. When a story has been completed, the process is repeated on the next floor.

As long as the architect has chosen, or is prepared to work within the constraints of regular wall alignments, the tunnel form is an excellent structural solution.

The smooth face of the formwork results in a high quality finish that can be decorated directly. This reduces the need for finishing trades, thereby providing additional cost savings and speeding of the entire process.

The formwork is available to the contractor for purchase or rent and can be reused on other projects.

The tunnel form suppliers/contractors provide full design and technical support to ensure that the engineers, architects and site staff are all familiar with the system and its application, as the project starts, thus enabling time and cost savings to be achieved. If the site staff is inexperienced on tunnel form construction, the supplier's site training quickly brings them up to speed.

The different components of a typical tunnel formwork system are shown in Fig. 10.26.

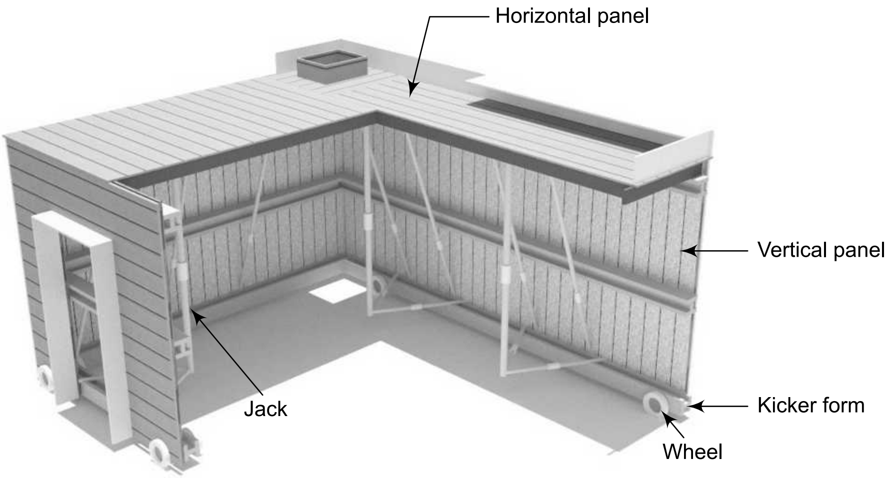


Figure 10.26 Tunnel Formwork System.

10.8.1 Advantages of Tunnel Form

Some of the advantages associated with tunnel formwork are given below:

1. **Saving in time and costs:** It is claimed that the application of tunnel form results in an average saving of about 25% in time and about 15% in costs.
2. **Productivity and control:** There is less reliance on skilled labor in the tunnel form construction. According to a leading manufacturer, using tunnel form, on an average, a team of nine workers plus a crane driver can strike and fix 300 m^2 of formwork each day, including the placing of approximately 35 m^3 of ready-mixed concrete: typically 2.5 cells.

   The repetitive and predictable nature of the tasks involved encourages familiarity with operations, and once training is complete, productivity improves as construction progresses.

3. **Work possible in all weather:** The work can continue in every weather except high winds, and heaters can be used to accelerate the concrete curing process.
4. **Quality:** Despite the high speed of construction, quality is never compromised in tunnel form construction. The steel sheathing of the tunnel formwork creates a smooth and high quality finish. The surface finish obtained, virtually reduces the requirement for finishing trades. For the finishing, only a minimum level of preparation (a skim coat may be required) is needed. This results in further savings in cost. Wallpaper can be directly applied or a skim coat may be used to provide a perfect plastered finish.
5. **Design:** The tunnel form provides great flexibility in design. It allows high degree of freedom in the final appearance. For example, it can provide for cantilevered balconies and a number of facade options such as: insulated framed infill panels, curtain walling, and so on. It is possible to achieve high level of dimensional accuracy and superior load distribution.
6. **Safety:** The tunnel formwork is provided with integral working platforms, guard rails, and edge protection systems. There is minimal requirement for tools and equipment when moving the tunnel form. This further helps reduce the risk of accidents. Also, since the work to be performed at the site is minimum due to the preassembled formwork, the chances of accidents are reduced.
7. **Ease of service installation:** Tunnel forms can facilitate pre-installation of service runs before the concrete is poured.

The advantages of the tunnel formwork system are summarized in Box 10.1

BOX 10.1 Advantages of Tunnel Formwork

- Fast formwork installation and concrete laying
- Economic feasibility through short construction periods, substantial savings in costs and labor, quicker return on investment
- A monolithical main frame that is formed by simultaneous laying of the concrete of curtain and floor together; buildings that are resistant to earthquake, design flexibility
- Clean concrete surface that does not require plastering
- Prefabrication and standardization of the complementary construction elements, 50% reduction in the construction period by the tunnel formwork when compared with traditional methods. In one day, the formwork of about one or two apartments can be erected and the laying of the concrete is easily completed by 10 workers
- The inner wall separations and materials such as window and floor frames, which require accuracy in concrete manufacturing, can be installed immediately after the moulding of the materials
- Enhanced safety

10.8.2 Disadvantages/Limitations of Tunnel Formwork

Some of the disadvantages/limitations associated with tunnel formwork are given below:

- High initial cost and requirement of extensive planning of architectural details are the major limitations. The high initial cost may get offset only by large number of repetitions, say of the order of 100 or more.
- Accuracy in fabrication and assembling calls for a high degree of supervision.
- Operations are crane dependent and require considerable crane time.

  The system requires a few weeks to understand and become familiar. Thus in the initial periods, productivity may be on a lower side.
- Requires good co-ordination between the structural designer and the formwork engineer.
- It is difficult to form large and different shaped structures using tunnel form. The system is mostly suited for buildings with repetitive configurations.

10.8.3 Case Study in Tunnel Formwork— South City Project at Bangalore

The tunnel formwork system was identified as the most economical system for the South City Project located in Bangalore. The project consisted of constructing 19 story residential building. Each story consisted of 8 flats of approximately 100 m^2 each. The building was symmetrical in two perpendicular axes and there was a repeating module of quarter portion in the building.

The builder for the project used two independent sets of tunnel form. While the majority of the area was catered using the tunnel formwork system, the conventional formwork was used for the non-repeating and sunken portions in the toilets and corridors of the building. The tunnel formwork system was shifted with the help of a tower crane employed by the builder.

The accelerated curing was carried out using hot air blowers. At peak speed, the 8 pours of tunnel form in a floor were completed in 9 days time, which is almost 1 flat per day construction speed.

10.8.4 Tunnel Formwork Cycle

Step 1

The first step is to cast the slab including the starter walls (kickers). The starter is usually of 75 mm to 100 mm depth and is used to position the tunnel forms (see Fig. 10.27). The starter walls should be carefully positioned and their vertical alignment must be accurate.

Step 2

After the starter wall forms have been stripped, it is time to tie the reinforcement which can be tied in either of the two possible ways: (1) placement of reinforcement before the tunnel formwork is placed and (2) placement of reinforcement after the tunnel formwork is placed. In the latter case, the reinforcement is in the form of pre-assembled sections. Tunnel forms are placed by crane between the rows of reinforcement, butted against the starter walls for alignment, and leveled with screw jacks (see Fig. 10.28). Box-outs for door, windows, electrical conduit, and mechanical block outs are also installed at this stage (see Fig. 10.29).

Figure 10.27 Construction of Starter Wall for the Tunnel Formwork System.

Figure 10.28 Placement of Tunnel Form in Progress.

Step 3

Forms for the other sides of each wall (which may also be tunnel forms) are then placed. Spacers are used to center the wire mesh between the forms. The ceiling panel is positioned at proper heights by adjusting the inclined struts. The struts transfer the load of the concrete to the base of the tunnel forms.

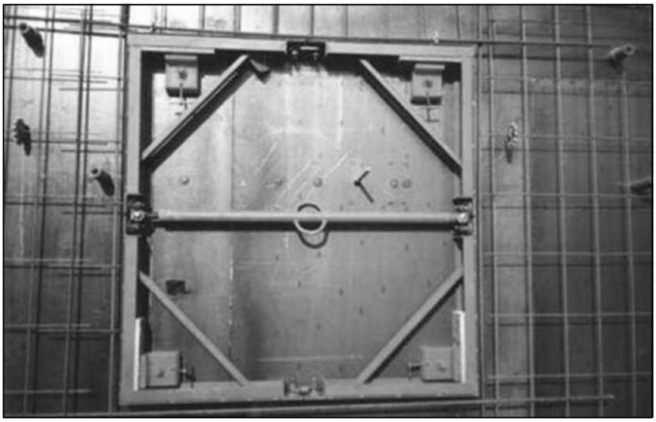


Figure 10.29 Tunnel Form (Window).

The tunnel form system has heating equipment (see Fig. 10.30) for accelerating the hardening of the concrete thus enabling a complete cycle of work to be performed every day. Some contractors even claim that 2 concreting operations are possible in a 24-hour period for the slabs of short span.

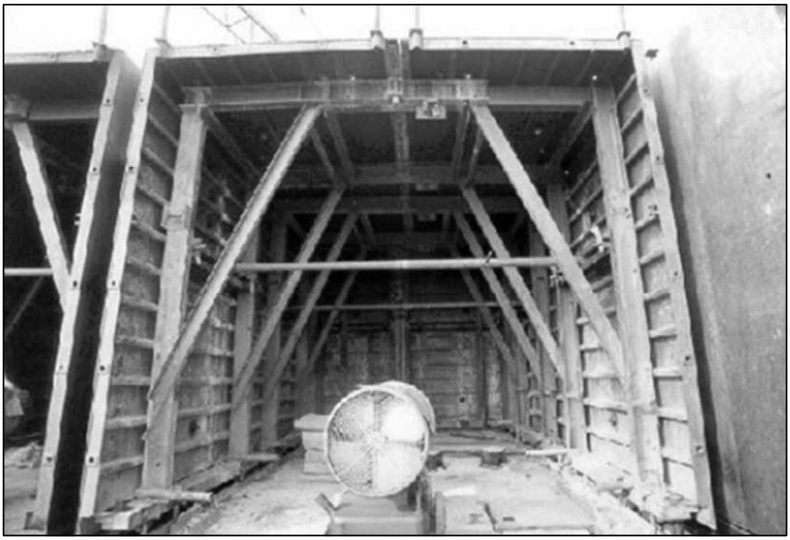
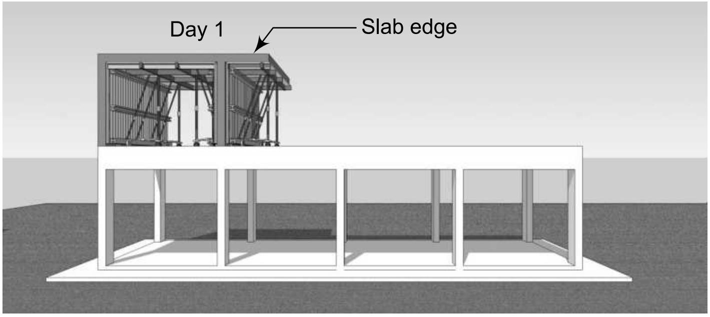


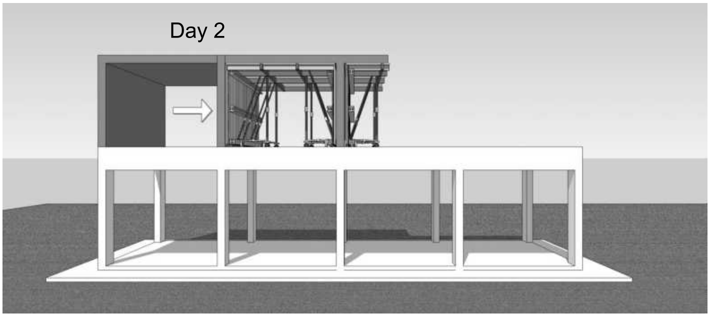
Figure 10.30 Heating Equipment in Position for the Tunnel Formwork.

The whole construction cycle thus consists of positioning the forms, fixing the steel reinforcement, pouring the concrete, curing the concrete, lowering the form and moving it to the new location and as mentioned, could be as short as 24 hours. It may be noted that different types of tunnel formworks have been shown in Figs. 10.28, 10.29, and 10.30 to illustrate the tunnel formwork cycle.

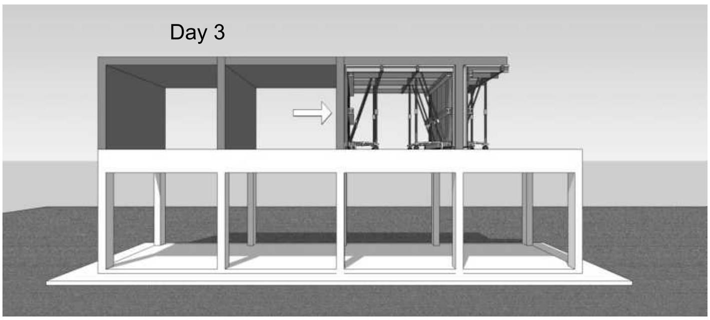
The day wise cycle for the tunnel formwork operation is shown schematically in Fig. 10.31.



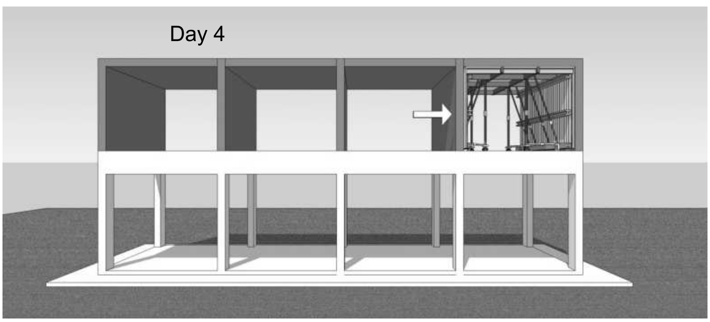
(a)



(b)



(c)



(d)

Figure 10.31 Typical Tunnel Formwork Cycle.

10.9 COLUMN MOUNTED SHORING SYSTEM

In conventional methods, the scaffolding or staging from the floor below is erected to support the formwork for slabs and beams. On the contrary, in the column mounted shoring system, the strength of the existing RC columns is utilized to support the formwork arrangement for the slabs and beams to be cast. Thus, the need of shores and reshores are completely eliminated in this system. The absence of shores means the availability of sufficient space on the floor below which can enable other works to be taken up in parallel. This ensures faster construction. According to a leading manufacturer, concrete frames can be completed about 30-40% faster with column mounted shoring system.

The system, though not very common, can be applied in the construction of high rise buildings such as hotels and residential apartments. A typical column mounted shoring system utilizing lightweight steel trusses supported on bracket jacks is shown in Fig. 10.32. The system is being used for the construction of a high rise building.

Figure 10.32 (a) View of Column Mounted Shoring System.

Figure 10.32 (b) Another View of Column Mounted Shoring System (Courtesy Formwork–Shoreall Available at http://www.formwork-exchange.com).

As can be observed, the column mounted shoring system essentially consists of sheathing, joists, stringers, supports for stringers, and bracket jacks. In the column mounted shoring system the formwork consists of plywood as the sheathing member and timber joists. This is similar to any other formwork system. Joists are supported on truss members acting as stringers. For smaller span, the stringer could be made up of structural steel sections such as single channels, box made up of two channel sections, etc.

The truss members are supported on steel I beams spanning perpendicular to the truss member. The I beams are supported on bracket jacks. The bracket jacks consist of a double steel roller at the top of the jacks, an adjustable screw to adjust the jacks, and a steel plate attachment. The rollers help ease the horizontal movement of the formwork panel. The formwork panel can slide in and out without much of an effort. The adjustable screw of the bracket jacks control the vertical movement (height adjustment) of the formwork panels. The jacks are available to suit different requirements of vertical height adjustments. The steel plate attachment is in contact with the concrete wall or column with the help of thorough bolts.

The above mentioned components are shown schematically in Fig. 10.33.

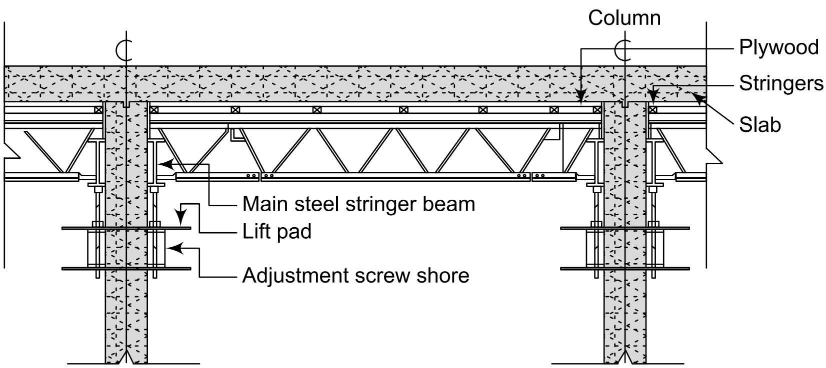


Figure 10.33 Schematic Sketches for Column Mounted Shoring System.

Advantages and Limitations of Column Mounted Shoring System

The advantages and limitations of column mounted shoring system are given below:

1. *Columns and walls support the deck form. Virtually eliminates reshoring.* Shores and reshores are not required in the column mounted shoring system. The existing concrete columns and walls support the load of the formwork, the concrete, and the crew working on the form.
2. *Clear span under forms to permit other construction activities.* Other construction activities on the floor can be taken up concurrently with the forming operation of the concrete slab. This is unlike other formwork systems where one has to wait till the shores are removed.
3. *Height of floors not a constraint.* In the conventional and some of the proprietary systems, the floor to floor height is a deciding factor in choosing the type of shores. For example, shores could be tubular props if the floor to floor height is less than about 4.5 m, whereas the tower

type shores or cribs are preferred for higher floor heights. In case of column mounted shoring system, height of floors is not a governing criterion.

4. *Large investment required initially.* The column mounted shoring system requires large capital investment initially, which can be justified when a large number of reuses are envisaged.
5. *Crane needed.* The system requires crane for various formwork operations.

10.10 GANG FORMS

The gang forming systems employ larger sized standard forms. In this system of formwork, large sections of high walls can be formed at a time at a given location. The whole formwork system is then moved by a crane to the next location without being dismantled. The system offers great opportunity of automation of different formwork operations. Besides this, there is a great amount of planning and scheduling effort required for the system. This is because of the requirement of interaction among the gang forming crew and the crane required for the various operations of formwork.

One typical gang formwork is shown in Fig. 10.34 wherein the application of the gang form has been shown for forming the wall of the track hopper at Anpara super thermal power station. The tower crane is also visible in the figure which is used to transport the forms from one location to another.

As in any other formwork system, the success of gang forming system also hinges upon the number of reuses of the gang form either in one project or in several projects. The success of gang forming depends on the availability of similar configuration of formwork elements at the same level and at different levels. For this, the designer of the facility should be taken into confidence.

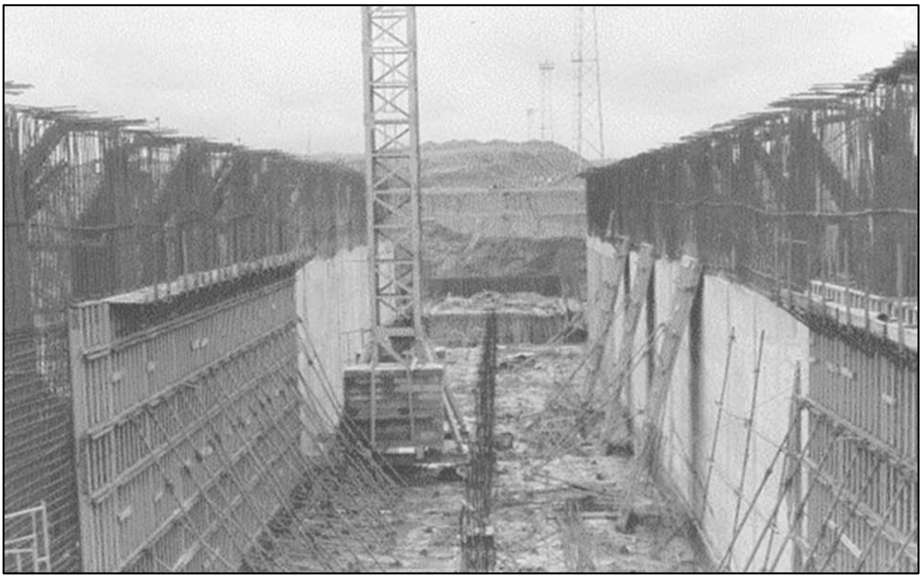


Figure 10.34 Construction of Track Hopper at Anpara Super Thermal Power Station Project Executed by L&T- ECC Construction Group.

10.10.1 Gang Forming Operation

A typical process chart for a gang forming operation is shown in Fig. 10.35. For the erection of each type of building component such as column, wall, beam and slab, the work tasks include the following in general:

1. tying reinforcement,
2. plumbing if any,
3. form lifting and erection,
4. concrete pouring,
5. curing, and
6. form stripping

The above work tasks are performed sequentially for different building components. The steel reinforcement could be prefabricated off site or it could be tied on site depending on the requirement and nature of the work. Before each form fixing, the forms have to be cleaned and oiled. The work tasks mentioned above require various resources involving complex interactions for their completion. Thus the availability of each resource type would affect the other resources which in turn would affect the overall progress of the work. Some of the resources required are:

- The various form components (e.g., columns, walls, beams or slabs)
- The cranes needed for form and steel lifting, installation and stripping of forms, concrete pouring, etc.
- The labor crews

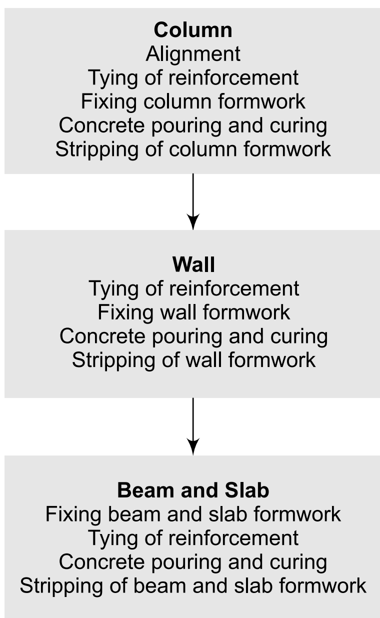


Figure 10.35 Schematic Sketch Showing Workflow for the Gang Forming Operation.

10.10.2 Reuse Schemes for Gang Forming Construction

Some of the factors affecting the choice of reuse scheme for gang forming construction are:

- The number of regions each building floor is divided into—the region is defined as the area which is covered by full sets of formwork for different building components such as column, wall, beam and slab.
- The number and the load carrying capacity of the cranes.
- The number of horizontal and vertical form sets.
- The number of crews, and the sequence of activities.

The detailed discussion on the above factors is done in Chapter 15.

Some of the reuse schemes practiced in the industry are explained briefly with the help of schematic diagrams.

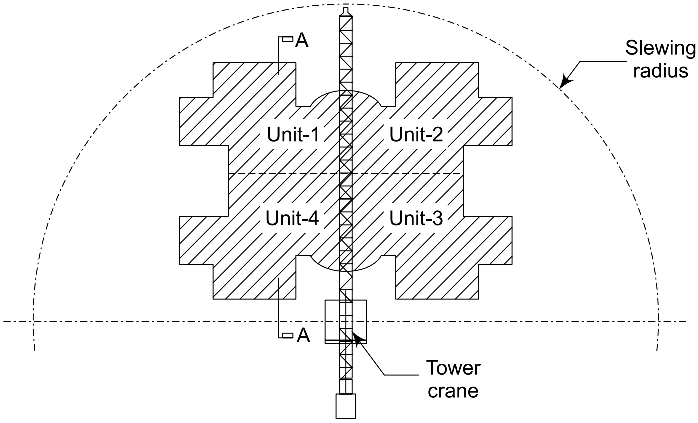
10.10.2.1 Single region, single crane, and form sets not shared in a floor

In this scheme, the building has only one region in a floor and the form sets are mobilized for one complete region. Thus only vertical movement of forms is possible. A single crane is assigned to the region for formwork operations. This type of scheme is useful for tall buildings (having number of floors) with small floor areas in a region. The site layout plan and the elevation of a building are shown in Figs. 10.36 (a) and (b). The building has been divided into four units and the formwork for all the concrete elements for the four units are mobilized simultaneously. Thus the four units constitute one region according to our definition. From Fig. 10.36 (b), it can be noted that forms are moved only in the upward direction, i.e., from the lower floors to the upper floors. The location of the crane is also shown in the figure.

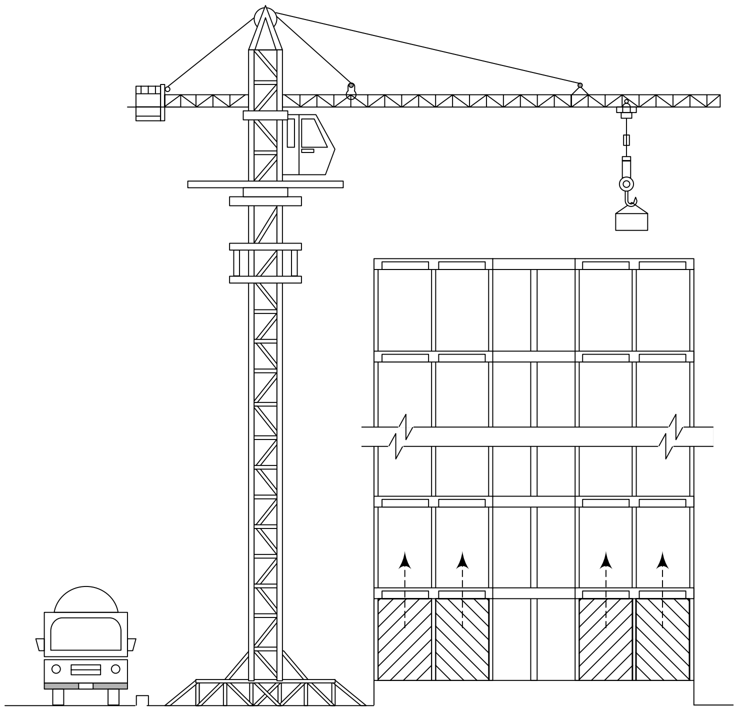
10.10.2.2 Multiple regions, single crane, and form sets not shared in a floor

In this scheme, the building has a number of regions in a floor. The form sets are not shared in a floor. In other words, form sets move only vertically from the lower floors to the upper floors. As a result, the formwork cost is going to be more. There is a single crane which caters to the needs of multiple regions. Thus, depending on the availability of crane formwork operations, all the regions in a floor can be taken up simultaneously. This type of scheme is suitable for projects having a tight schedule.

We take the previous building (referred in previous section) for illustrating the concept. The building has four units. Now the two units, unit 1 and unit 4 constitute one region. The form for columns, walls, beams, and slab for unit 1 and unit 4 are mobilized as one set (see Fig. 10.37(a)). Thus the plan of the building has two regions. Region 1 consists of units 1 and 4 while region 2 consists of units 2 and 3. Since time is a constraint here, the formwork operation is in progress for both the regions depending on the availability of the only crane that is available. Thus, in this case also, the forms for regions can not be moved within the floor and only their vertical movement is possible as shown in Fig. 10.37(b).



(a) Site layout plan



(b) Elevation A-A

Figure 10.36 Illustration of Single Region and Single Crane Scheme, Form Sets not Shared in a Floor.

10.10.2.3 Multiple regions, single crane, and form sets shared in a floor

In this scheme, the building has a number of regions on a floor. The form sets are shared in a floor. In other words, the form sets move horizontally first in a floor, and then vertically from the lower floors to the upper floors. This would result in lower formwork cost. There is a single crane which caters to the needs of the multiple regions. Since the formwork has to move horizontally also from one region to another region in a floor, this type of reuse scheme is suitable where the project duration is not too tight. Work can take place at a relaxed pace. Needless to say, in order to reduce the time, the form sets and the associated crews have to be increased.

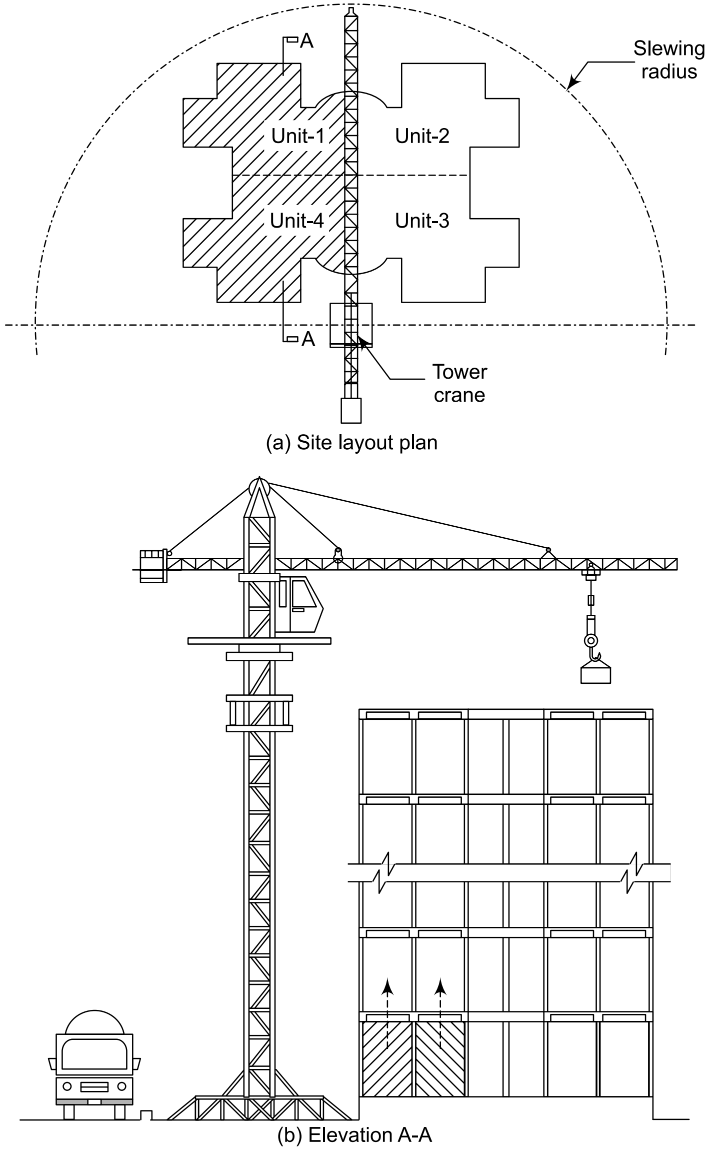


Figure 10.37 Illustration of Multiple Regions and Single Crane Scheme, Form Sets Not Shared in a Floor.

In Fig. 10.38 (a), the building has been divided into four units. The formwork for columns, walls, beams and slab for unit 1 has been mobilized fully. Thus, according to our definition, unit 1 consists of one region. Thus we have four regions in the building. The form for region 1 moves to other regions on the same floor first and then it is taken to the upper floors. The movement is shown in the elevation Fig. 10.38 (b). Obviously, the forms have more opportunity to be reused in this scheme. The location of the crane is also marked in the figures.

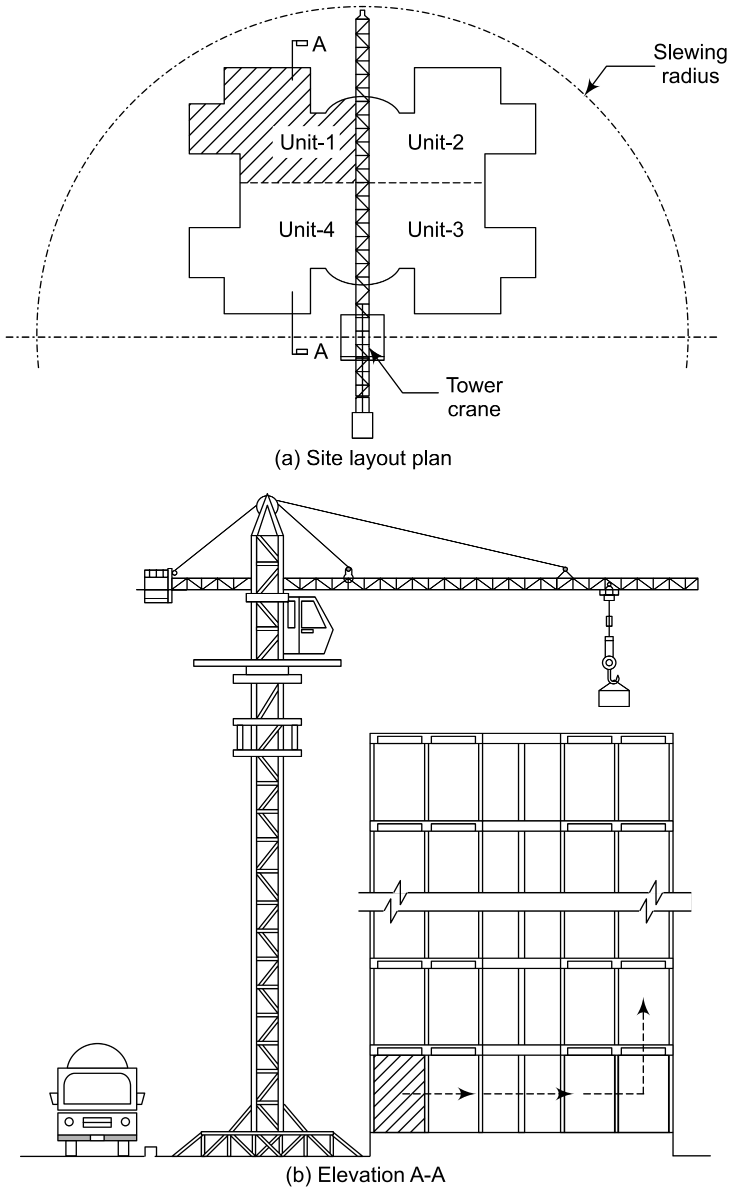


Figure 10.38 Illustration of Multiple Regions, Single Crane and Form Sets Shared in a Floor.

10.10.2.4 Multiple regions, multiple cranes, and form sets shared in a floor

In this scheme, the building has a number of regions on a floor. There are multiple cranes available. Each region is serviced by a single crane for formwork operations. Obviously such schemes would be preferable when there is a lot of work to be performed by the crane in a region besides the formwork activities. The form sets are shared in a floor. In other words, the form sets move horizontally first in a floor and then vertically from the lower floors to the upper floors.

In Fig. 10.39 (a), the building has been divided into 4 units and each unit is considered as a region. There are two cranes employed for the job. The forms for the region are shared within the floors first and then are moved to the upper floors.

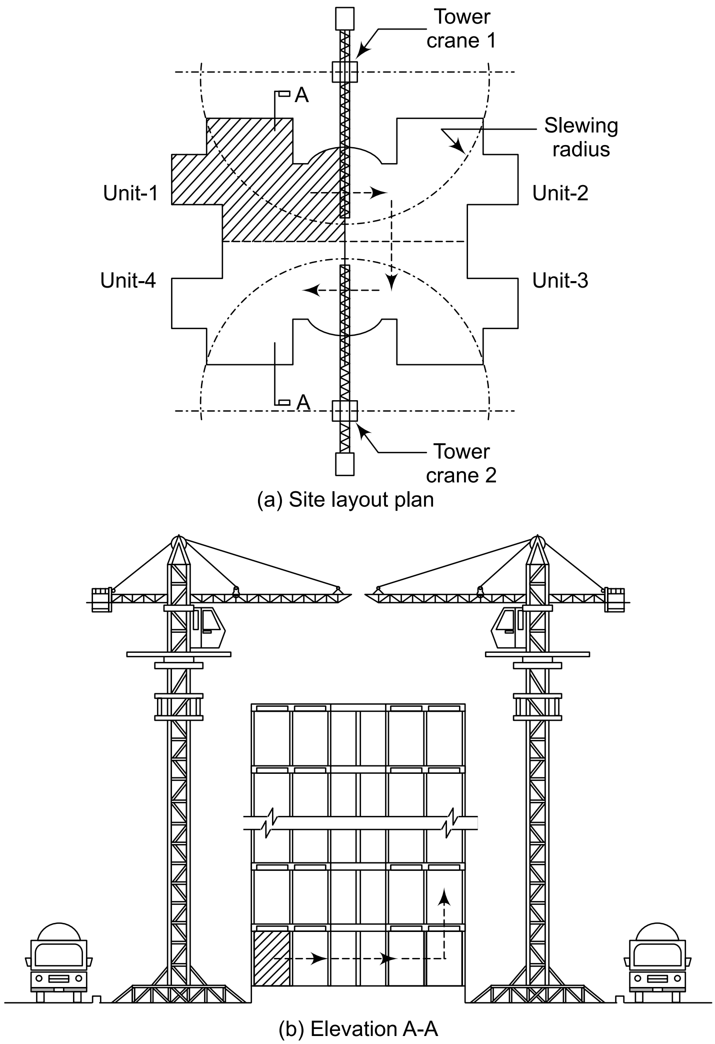


Figure 10.39 Illustration of Multiple Regions, Multiple Cranes and Form Sets Shared in a Floor.

10.10.2.5 Multiple regions, multiple cranes, and forms not being shared in a floor

In this scheme, the building has a number of regions on a floor. The form sets are not shared in a floor. In other words, the form sets move only vertically the from lower floors to the upper floors and as a result, the formwork cost increases. There are multiple cranes available. Each region is serviced by a single crane for the formwork operations. Obviously such schemes would be preferable when there is extreme urgency to complete the project. The formwork cost is going to be extremely high in such a situation.

In Fig. 10.40 (a), the building has been divided into four units. The formwork for columns, walls, beams and slabs of each of the four units has been mobilized. And thus according to our definition, the formwork for all the four regions have been mobilized. The forms thus move only in the vertical directions i.e., from lower floors to upper floors. Obviously such schemes can prove to be a costly option. Each region has a dedicated crane for formwork and other operations. The locations of each of these cranes are shown in the plan and elevation both.

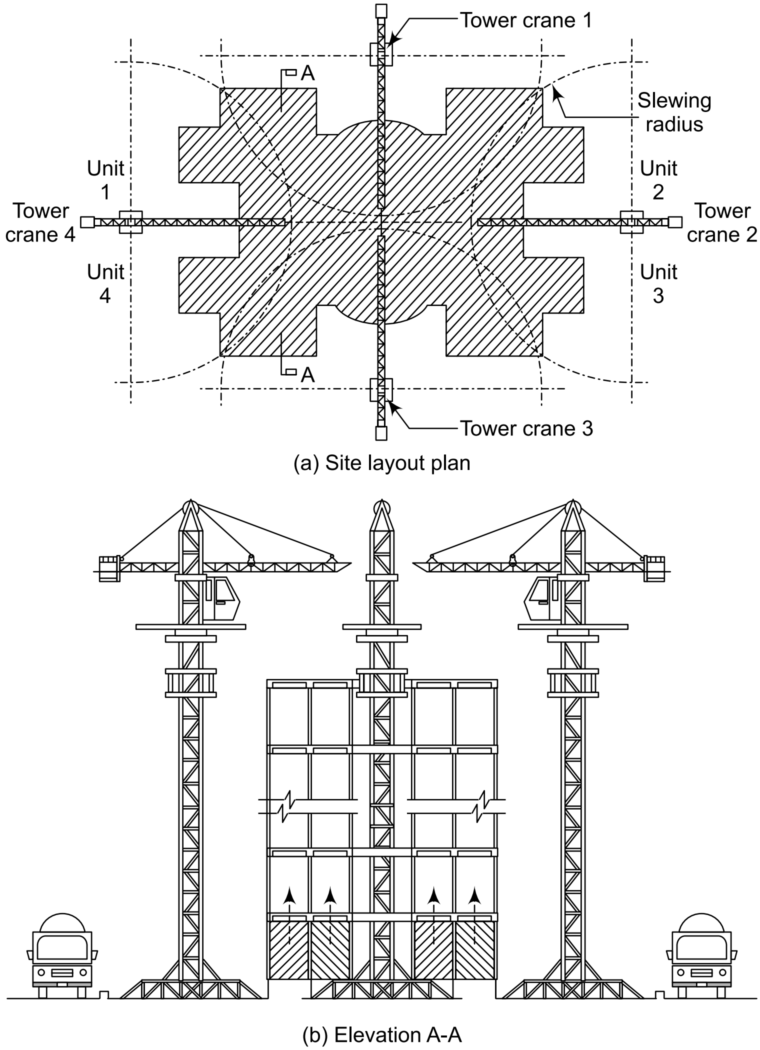


Figure 10.40 Illustration of Multiple Regions, Multiple Cranes and Forms Not Being Shared in a Floor.

REVIEW QUESTIONS

Q1. True or False

(a) The flying form is a system of formwork which is assembled into form units usually on the ground and is located to form concrete elements at the site location.

(b) The term flying formwork is used because forms are flown from one story to another story with the help of a crane.

(c) The flying forms are more common in high rise buildings for rapid cycle construction wherein large repetitions are possible thus justifying their relatively low initial cost of fabrication.

(d) The flying form also inflates the material and labor cost.

(e) A typical unit of flying formwork consists of sheathing member, trusses or shores, steel or wood beams as stringers, joists.

(f) Tunnel forms, flying truss systems, and column mounted shoring systems are different types of flying formwork system.

(g) Table form and flying from are two different entities.

(h) In table form, the speed of construction is very slow.

(i) In table form, large area formwork is pre assembled at the ground usually.

(j) In tunnel forms, it is possible to cast walls and slabs in one operation in a one day cycle.

(k) Tunnel forms are used to form repetitive cellular structures such as hotels, residential buildings etc.

(l) Tunnel forms provide accuracy, flexibility, and economy of in-situ construction.

(m) Tunnel forms are very economical for projects having 100 or more cellular units.

(n) Tunnel form uses steel for all components.

(o) Tunnel form consists of a sheathing of thick steel plate both for casting wall and ceiling, waler and diagonal strut assembly.

(p) Gang forming operations include—tying reinforcement, plumbing, form lifting and erection, concrete pouring, curing and form stripping in general.

(q) Gang forming systems employ larger sized standard forms.

Q2. Match the following

(i) Conventional method of scaffolding	(a) strength of existing RC columns is utilized to support the formwork
(ii) Column mounted shoring system	(b) scaffolding or staging from the floor below is erected to support the formwork for slabs and beams.

Q3. Write short notes on

(a) Interform Aluminum Flying Forming System

(b) Symons Multiple Reuse Flying Truss System

(c) Harsco Infrastructure Flying Table Form

Q4. Enunciate the different steps involved in the flying formwork cycle.

Q5. List out the advantages and limitations of the flying formwork.

Q6. List out the design issues, safety issues, operations hazards, and remedies for the flying formwork.

Q7. Write the do's and don'ts involved in the safety operations for the flying formwork.

Q8. Write short notes on:
   (a) L&T table formwork
   (b) PERI table formwork

Q9. List out the advantages and limitations of table formwork.

Q10. Prepare short notes for the two variants of tunnel form by Symons Corporation.

Q11. List out the advantages, disadvantages, and limitations of the tunnel form.

Q12. List out the different steps involved in the tunnel formwork cycle.

Q13. Prepare short notes on column mounted shoring system.

Q14. List out the advantages and limitations of the column mounted shoring system.

Q15. Prepare short notes on Gang forming system.

Q16. List out and describe different reuse schemes for Gang forming construction.

Chapter 11

Slipform

Contents: Introduction; Vertical Slipform; Horizontal Slipform; Types of Slipform; Functions of Various Slipform Components; Assembly, Sliding, and Dismantling of Slipform; Slipform Design Issues; Some Cases in Slipform; Safety Operations During Slipform Erection; Productivity Issues in Slipform Construction

11.1 INTRODUCTION

Slipform construction, also known as sliding form construction, is similar to extrusion process in which the wet concrete is extruded rather than retained in the forms until it has hardened. The slipform construction method is used to build high-rise structures quickly. In this method, the concrete is placed at a pre-determined rate on top of a travelling form, which emerges in a hardened state from the bottom. Concrete is shaped in the desired profile during the travel of the form. In slipform, the form moves semi continuously with respect to the concrete surface. The movement is at such a regulated speed that the concrete when exposed, has already become strong enough to retain its shape, and support its own weight besides being capable of supporting the vertical pressure from the concrete still in the form, as well as to withstand the lateral pressure caused by the wind, etc. It is different from other forms in a sense that form ties are not used in slipform.

In the initial days of slipforming, wooden forms in combination with wooden screw jacks and wooden yokes were used. These were replaced by metallic forms, yokes, and hydraulic jacks. The slipform construction technology was used to construct one of the notable structures— the 345 m high concrete shaft of CN Tower in Toronto in 1974 (see Fig. 11.1).

In India also, a large number of structures have been constructed using slipform. In one of the early constructions in 1988, slipform was used to construct 235 m high TV tower in Pitampura, New Delhi (see Fig. 11.2).

The slipform construction method can be applied with great advantages to many construction projects such as chimneys, silos, water towers, telecommunication towers, bridge piers, pylons, stair and elevator shaft cores, shaft linings, heavy concrete offshore platforms, and oil platforms etc. Slipforming is resorted to where continuous concreting is possible and where monolithic structure is desired.

Figure 11.1 CN Tower at Toronto (One of the Early Slipformed Structures).

Figure 11.2 Pitampura TV Tower in New Delhi, 235 m Height Constructed Using Slipform.

Although the initial investment required in slipform construction is high, in the long run, the technology proves to be economical due to the reduction in construction time and labor cost compared to other formworks. High slipping speed and accurate alignment of the concrete structures are possible now with the application of large yoke capacities and better laser guidance. Some other advantages offered by slipform construction over other formworks are: high quality finished surfaces and continuous moving monolithic structure.

Slipform construction can be used both for vertical and horizontal structures. Vertical slipforming is used for silos, bins, shafts, cores, caissons, bridge piers, etc. Horizontal slipforming is used for canal lining, tunnel inverts, highway pavements, water conduits etc.

11.2 VERTICAL SLIPFORM

The vertical slipform system essentially comprises of form panel (made of timber or steel), yoke assembly, jack rod, mechanical or hydraulic jack, working platform, finisher's platform or scaffold etc. The form panels are supported by yoke assembly which is usually placed at about 1.5 m to 2.0 m intervals. The formwork moves with the help of mechanical or hydraulic jacks. The jacks climb either on a pipe or a solid rod embedded in the concrete. These pipes/rods are also known as climbing rods or jack rods. Loads from shuttering, working platform, finisher's platform/scaffold etc. get transferred to the jack rod through the jacks. The jacks have gripping devices which could either be tooth type or ball type. The sliding speed of the form depends primarily on the setting time of the concrete, the ambient temperature of the concrete, and the type of cement.

During the initial days, the vertical slipforming was used primarily for structures with uniform cross sections. However, with recent developments, it is possible to slipform continuous vertical flutings and varying cross sections. It is not possible to form the projections in the structure, for example, walkways within the forms during the actual slide. For constructing projections, dowels are bent to fit within the forms which are straightened and cast later. The openings for doors and windows at different locations are also normally not attended to at the time of sliding. Block-outs for doors and windows, inserts or pockets to take beams etc. can be placed inside the form as the slide progresses, which can be attended later.

11.3 HORIZONTAL SLIPFORM

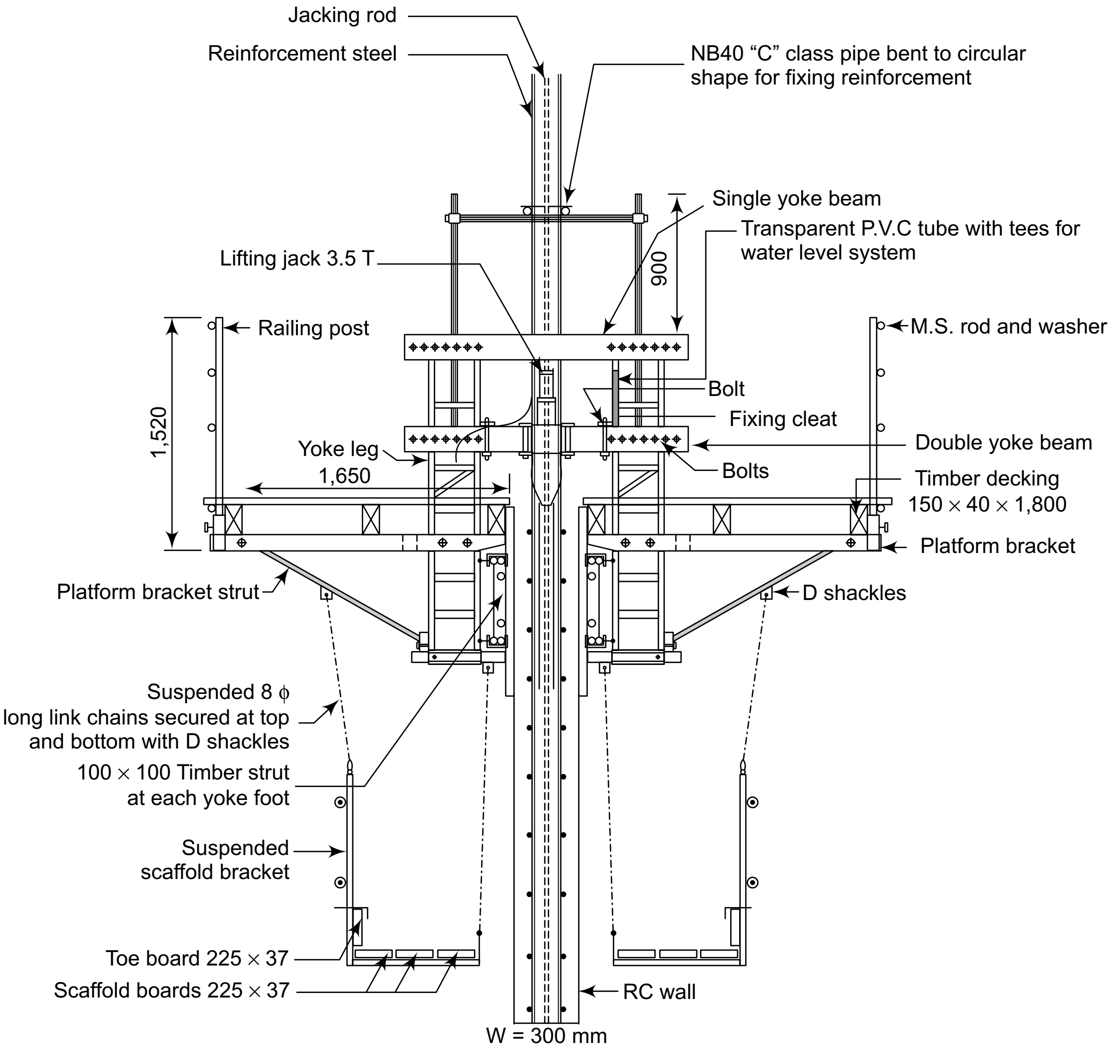
Horizontal slipforms generally move on a rail system. A shaped beam can also be used in place of a rail system. The arrangements for the working deck, the finisher's platform or the scaffold are common in both the vertical and horizontal slipforms. These attachments also get moved along with the moving formwork.

11.4 TYPES OF SLIPFORM

The slipform can be classified as the straight slipform, the tapering slipform, and the slipform for special applications.

11.4.1 Straight Slipform

Some typical structures constructed using the straight slipform technique are: silos, straight chimneys, water towers, columns etc. For silos, tanks, bridge columns, nuclear power stations and industrial buildings, slipform construction provides a sound economic solution for the project. Other structures suitable for slipforming are: chimneys, radio towers, water towers, caissons, stair and lift shaft cores shaft linings, tanks etc. To achieve the maximum benefit from the slipform method, planning is essential at an early stage. The schematic diagrams for straight slipform are shown in Figs. 11.3 and 11.4. Figure 11.3 shows the sectional view while Fig. 11.4 shows the part plan. The various components used are marked on the figures.



(a) Schematic view

(b) Actual photograph

Figure 11.3 Sectional View of Straight Slipform.

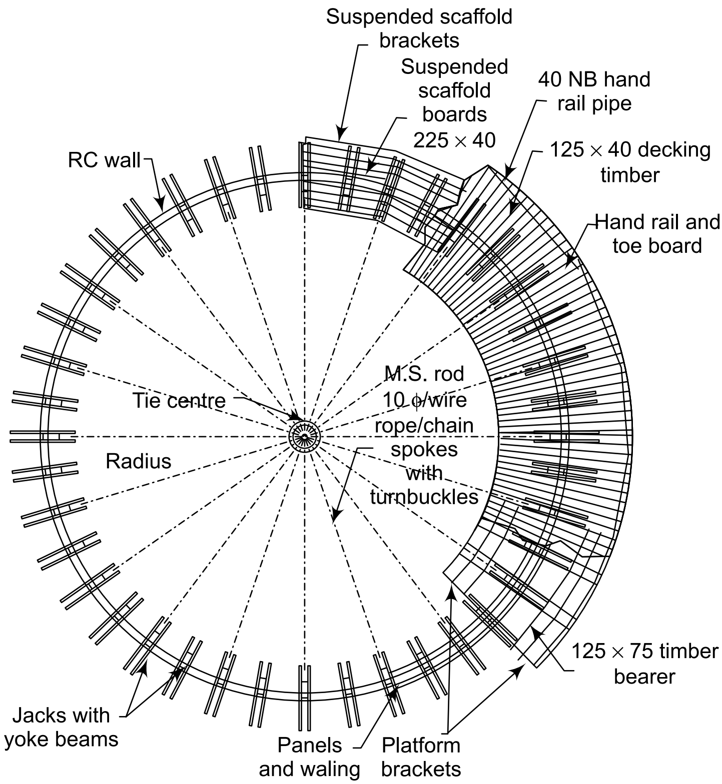


Figure 11.4 Part Plan for Straight Slipform.

11.4.2 Tapered Slipform

It is now possible to customize the slipform system depending on the type of structures. For example, it is possible now to slipform the structures which have changes (decrease/increase) in the wall inclination and variation in the wall thickness. For this purpose, special yokes are used which enable horizontal movement for changing radius. The tapered structures such as tapering chimneys and ventilators, bridge piers, tapered towers for offshore structures, and silos are quite easily slipformed in modern times. A typical tapered slipform assembly is shown in Fig. 11.5.

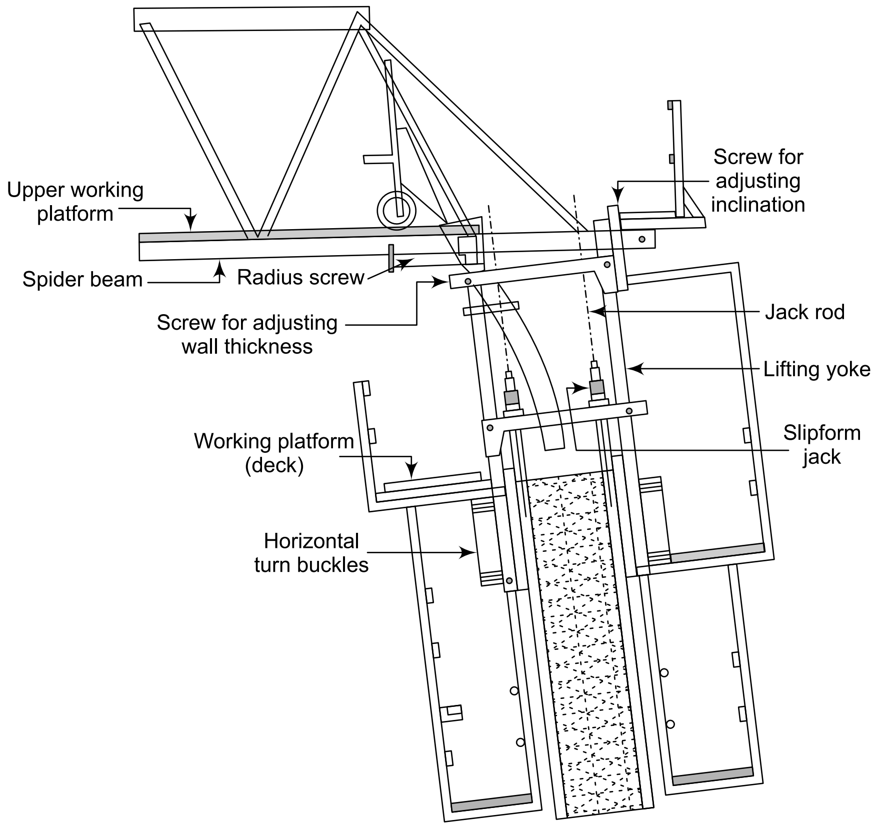


Figure 11.5 Slipforming for Tapered Structures.

Unlike straight slipform, the design of tapering slipform system is more complicated. The components are assembled in such a way that as and when the slipform moves up, the reduction in dimensions (for example diameter and wall thickness) of the structure and the inclination of wall is taken care of. Most of the components are fixed on one end and are free to move on the other end to cater to the changing profile of tapered structure. The reduction in circumference is facilitated by a set of hydraulic cylinders using spindles with left hand and right hand threads.

11.4.3 Inclined Slipform

Inclined structures, such as bridge-piers for suspended bridges and different types of walls and shafts can be slipformed using special forms. Special yokes are designed and fabricated to take care of the inclination and tapering of structures. Some examples of the inclined slipform construction are: the tower of a bridge, and control tower of an airport.

A concrete lining project, 100 m long inclined mountain shaft leaning 43° from the ground plane, was slipformed in 1959. The form was guided by rails earlier used when blasting the shaft and by the finished concrete surface (see Fig. 11.6).

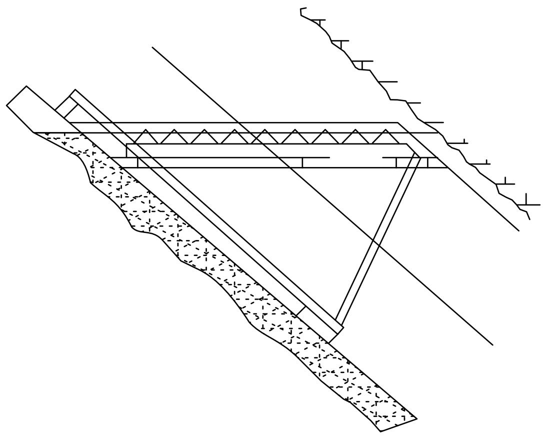


Figure 11.6 Inclined Slipform.

11.5 FUNCTIONS OF VARIOUS SLIPFORM COMPONENTS

Before elaborating the assembly and dismantling techniques of slipform, it is essential to know the functions of the various slipform components in the assembly. Following are some of the major components used in slipform operations.

- Sheathing
- Wales (also referred as 'walers' by some manufacturers)
- Yokes
- Jacks
- Jack rods
- Working or storage decks
- Walkway brackets (inside and outside)
- Hydraulic pump, and
- Inside and outside mason's (hanging) scaffold

The function and the commonly used materials for the above components are discussed below briefly:

11.5.1 Sheathing

The function of sheathing is to contain and provide shape to the concrete. It also helps maintain the correct profile of the structure to be slipformed. The sheathing along with the wales, resist the concrete pressure. The sheathing can be made out of timber, plywood, metal, glass fiber, reinforced plastic or a combination of these materials.

11.5.2 Wales

The wales are used to support and hold the sheathing in place. They transmit the lifting force from the yokes to the sheathing and to the other elements of the form. Wales also provide support for various platforms and the scaffolding.

11.5.3 Yokes

Yokes support the wales at regular intervals with their legs, transmit the lifting forces from the jacks to the wales, and resist the lateral force of the plastic concrete within the form.

Yoke Legs

The function of yoke leg is to lift the slipform structure as one integral unit. It transfers the lifting reactions to the jacks. Yoke legs also act as the main connecting member for walkway platforms, mason's scaffold, yoke beams, top platforms, etc.

Yoke Beams

Yoke beam is the main connecting member between the inside and outside yoke legs. Two yoke beams are connected at the bottom portion of the yoke legs and a single yoke beam is connected at the top portion. Jacks are mounted over the yoke beams. The yoke beam transfers the lifting forces of the jacks to the yoke legs.

11.5.4 Jacks

The jacks, installed on the yoke beams, climb up the jack rods and provide the force needed to raise the entire slipform system. In one patented arrangement by a leading manufacturer, the jack grips the jack rod with two ballgrips. Jacks are to be suitably located preferably at equal intervals to enable lifting the slipform as one integral unit. The capacity of the jacks is decided depending upon the reactions at the point of lifting.

One of the leading manufacturers of jacks produces these jacks with a working capacity of 3, 6 and 12 ton (Fig. 11.7). The 6 ton jack is normally used for high-rise tapered structures and when the required lifting load is heavy. The 3 ton slipform hydraulic jack is normally used for straight structures and when the load is normal. The stroke for each lift can be adjusted individually making it possible to operate the slipform with the desired accuracy.

11.5.5 Jack Rods or Climbing Rods

Jack rods are also known as climbing rods. These rods are normally located centrally in the wall to be cast or equidistant in the yoke beams depending upon the number of jacks. The jack rods can be 48 mm, 32 mm, or 25 mm diameter depending on the capacity of jacks. The lifting jack climbs over the jack rod. The entire load of the slipform assembly is transferred to the jack rod when the jacks are energized.

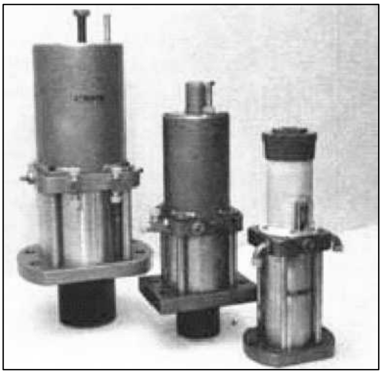


Figure 11.7 The Interform Hydraulic Slipform Jacks with 3, 6, and 12 Ton Lifting Capacity.

In order to salvage (extract) the jack rods, a thin pipe or tapered tube sleeve is usually placed around the jack rods for about one meter depth from the concrete surface and attached to the yoke. This way, the sleeve forms a hole for the jack rod to stand freely as the form moves. After the completion of the slide, the jack rod is pulled out.

11.5.6 Working or Storage Decks

Various platforms, decks, and scaffolding complete the slipform system. They provide a space for the storage of concrete, reinforcing steel, and embedments, as well as serve as a working area for placing and finishing.

11.5.7 Walkway Brackets (Inside and Outside)

Walkway brackets (inside and outside) facilitate the placing of concrete, the tying of reinforcement, vibration, fixing inserts, block outs, pockets, etc. Inside and outside brackets are connected with the respective yoke legs with the help of a pin for easy erection and dismantling along with a pipe strut to support the cantilever portion.

11.5.8 Hydraulic Pump

The function of the hydraulic pump is to circulate the required quantity of hydraulic oil at the required pressure for energizing the jacks to lift the assembly. The pumps also facilitate uniform lifting of the assembly. The hydraulic pump by a leading manufacturer is shown in Fig. 11.8.

11.5.9 Inside and Outside Mason's (Hanging) Scaffold

The inside and outside mason's (hanging) scaffold are used for curing, finishing the exposed concrete, and carrying out necessary repairs and treatments etc., if required. They are also used for the exposing of inserts and dowels etc., if required.

11.6 ASSEMBLY, SLIDING, AND DISMANTLING OF SLIPFORM

After the construction of the foundation of the structure to be slipformed, the inner and outer forms are assembled. The form consists of the sheathing member, the support member, and the bracing

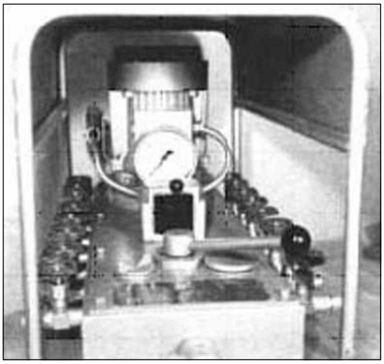


Figure 11.8 The Interform Slipform Pump Unit, PA-4.

members to ensure that the required shape is maintained during the slipforming operation. The gap between the inner and outer forms creates the void thickness of wall in which the vertical and horizontal reinforcements are tied. The forms are connected to jack rods with hydraulic jacks which move the form. The slipform assembly is an extremely complicated activity which has to be done in a systematic manner. It requires an expert's supervision at different stages.

11.6.1 Assembly of Straight Slipform

The various steps involved in the assembly of straight slipforms are given below:

Step 1

A kicker or starter (150 mm deep) is cast over the base slab or the foundation. The vertical and horizontal reinforcements are positioned properly to ensure the correct cover (Fig. 11.9). Reinforcement starter bars must be between a minimum of 1.2 m and a maximum of 1.8 m in height. Lacing bars must be fitted before the erection of the slipform rig can begin. In Fig. 11.9, kicker is not shown for the sake of clarity.

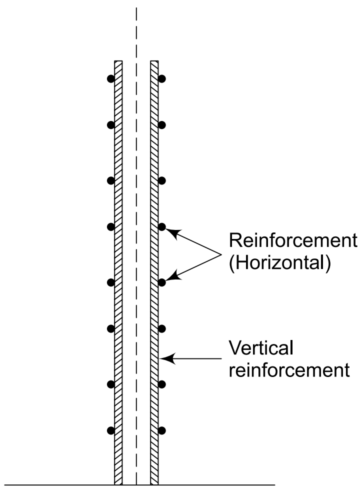


Figure 11.9 Step 1 of Slipform Construction Process.

Step 2

The panels are clipped together and positioned according to the wall lines on the base slab. This is shown in Fig. 11.10.

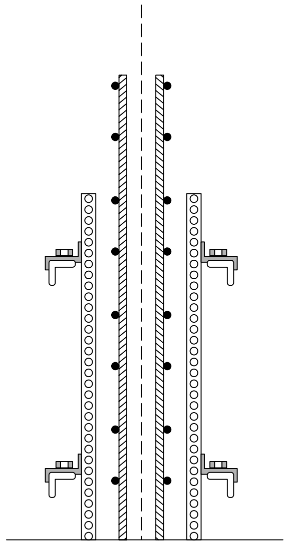


Figure 11.10 Step 2 of Slipform Construction Process.

Step 3

The top frames are added and can be used to give taper to both sides of the shutter, the panels are then braced, fixing them in position. The corresponding arrangement is shown in Fig. 11.11.

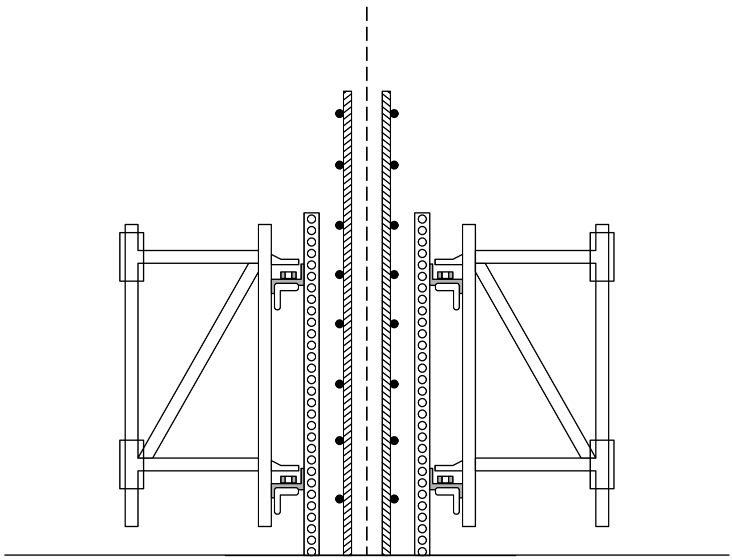


Figure 11.11 Step 3 of Slipform Construction Process.

Step 4

The pre-assembled yoke frames are positioned and fixed to the sides of the shutter at both the waling levels. This is shown in Fig. 11.12.

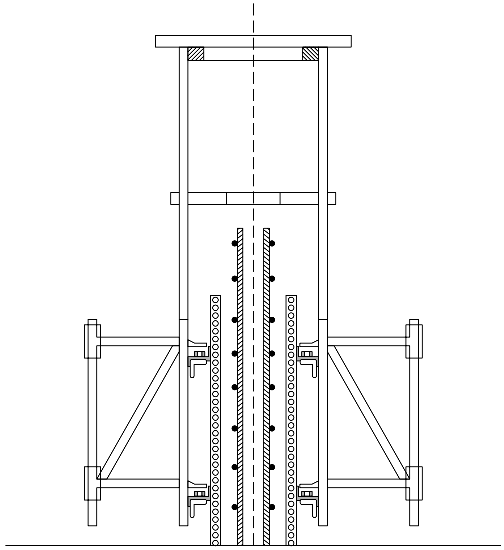


Figure 11.12 Step 4 of Slipform Construction Process.

Step 5

The working deck is made up of 100 mm × 100 mm timbers fixed into the tops of the top frames and 225 mm × 50 mm boards spanning between the top frames. This is shown in Fig. 11.13.

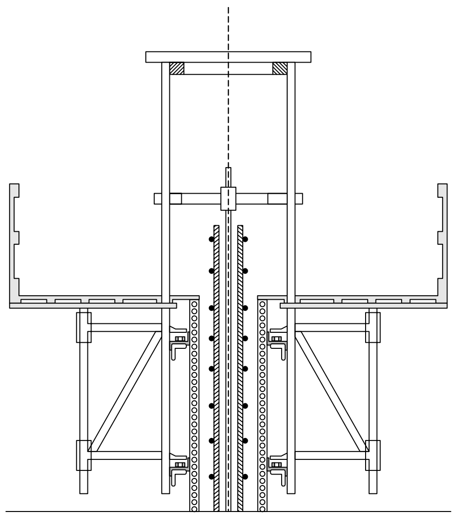


Figure 11.13 Step 5 of Slipform Construction Process.

Step 6

With the working deck fitted, the top deck can be assembled. The top deck is made up of primary and secondary 125 mm × 100 mm timber beams and 225 mm × 50 mm boards (see Fig. 11.14).

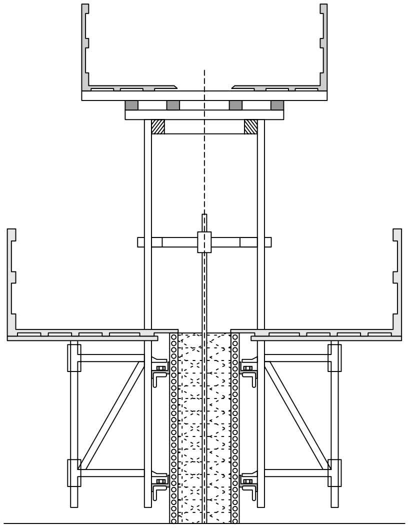


Figure 11.14 Step 6 of Slipform Construction Process.

Step 7

The hydraulic jacking units are fitted at every lifting point and the climbing tubes lowered down through them from the top deck. Once the hydraulics have been tested, the rig is ready to slide. The corresponding arrangement is shown in Fig. 11.15.

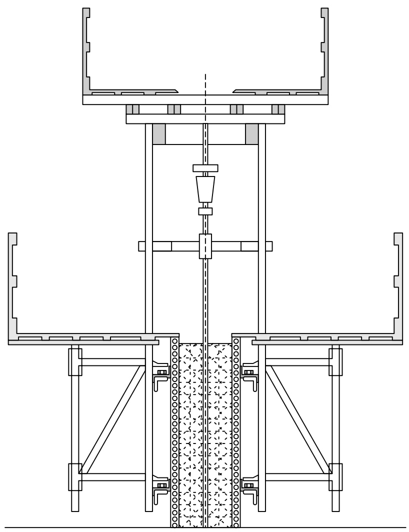


Figure 11.15 Step 7 of Slipform Construction Process.

Step 8

Once the slide has reached a sufficient height, the hanging scaffold frames are connected and decked out. The corresponding arrangement is shown in Fig. 11.16.

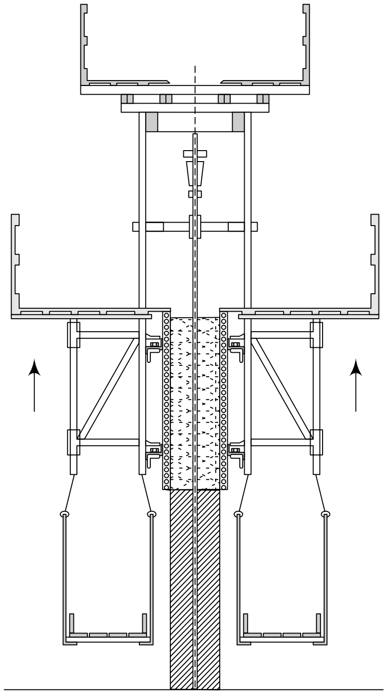


Figure 11.16 Step 8 of Slipform Construction Process.

11.6.2 Sliding Operation

The technique is based on movable forms which are gradually lifted by hydraulic jacks. The jacks climb on special steel rods which are extended as the concrete structure grows. It is a continuous process where wet concrete is added to wet concrete. The reinforcing steel and/or post-tensioned cables are continuously fixed as the form climbs. The concrete surface that appears below the form is usually treated by floating and brushing before it hardens. Normal slipping speed is three to six meters per 24 hours. In Figs. 11.17–11.19, the different stages of slipform operation are schematically explained.

The movement of slipforms should be carried out safely and accurately. The slipform movement is a continuous operation till the structure is completed or the structure has reached a height where a stoppage was planned. While deciding the rate of movement, the quality of concrete and the construction conditions prevailing at the given point of time should be considered. The supervisor should ensure that the form leaves the formed concrete only after it has attained the desired strength, and is capable to resist the loads exerted on it. Also, it should be ensured that the maximum rate of movement should be less than or equal to the rate of movement for which the forms are designed. In addition, both the maximum and minimum rates of slide should be determined by an experienced slipform supervisor to accommodate the changes in weather, concrete slump, initial set of concrete, workability, and the many exigencies that arise during a slide and cannot be accurately predicted beforehand. A person experienced in slipform construction should be present on the deck at all times during slipform movement.

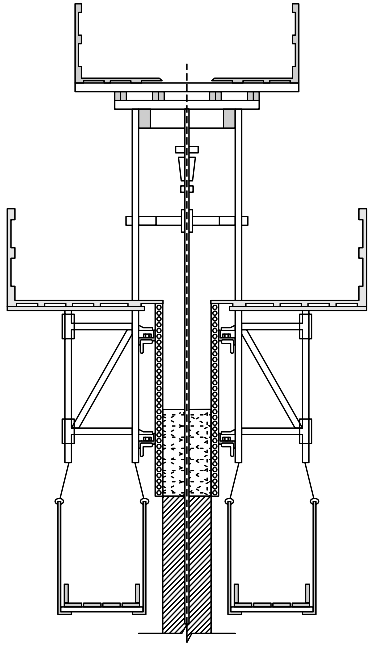


Figure 11.17 Stage 1 of Slipping.

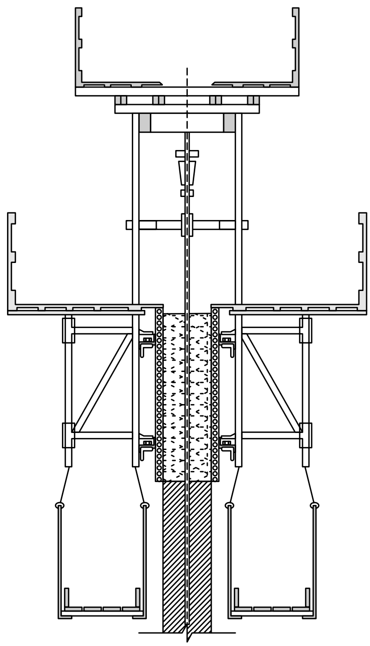


Figure 11.18 Stage 2 of Slipping.

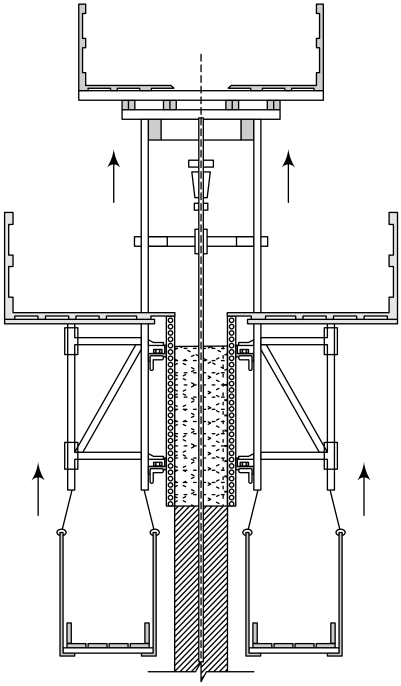


Figure 11.19 Stage 3 of Slipping.

The following issues based on the recommendations of ACI 347 need attention and should be closely observed during the slipform operation.

1. **Leveling of forms and concrete:** Forms should be leveled before they are filled. The use of a probe to establish safe lifting rates is recommended. The level of the hardened concrete in the form should be checked frequently by the probe.
2. **Drifting from plumb or twisting:** It is essential to prevent the form from drifting from the plumb or twisting due to torsional movement during the slipform operation. The plumb of the structure can be checked by a plumb bob/optical plummet/laser beams, and also by the vertical tape measures at fixed points. It is recommended that the formwork should be adequately braced and the platform should be kept leveled at all times. The level of the platform can be maintained by checking constantly with reference to the marks on the jack rods or by the water level systems.
3. **Frequency of check for alignment and plumbness:** The alignment and plumbness of a structure should be checked with respect to the alignment or the designed dimensions at least once during every four hours that the slide is in operation and preferably every two hours according to the recommendations of ACI 347. Where the stoppages are planned in slipform operation, alignment and plumbness should be checked at the beginning of each slipform operation. For tall structures with relatively small plan dimensions, ACI 347 recommends frequent readings so that the condition of being out of plumb and any twist can be detected early and remedial measures can be taken. It is also recommended to keep the records of both the vertical and lateral form movements throughout the slipform operation.

4. **Planning and coordination:** Slipforming involves a large crew taking charge of the different activities in a round the clock operation. Unlike *cast-in-situ* structures, formwork, reinforcement, and concreting are simultaneously done in slipform. Activities like fixing inserts, pockets for permanent and temporary arrangements are also done along with the concrete, requiring meticulous planning of activities and resources. It is obvious that thorough planning and coordination between the various crews are essential in ensuring the success of the slipform operation. It is advisable to prepare a chart/checklist in advance showing reinforcements, inserts, pockets, openings, coil nuts, sleeves, etc.
5. **Constant supervision:** ACI 347 recommends that the construction of the slipform and slipping should be carried out under the immediate supervision of a person experienced in slipform work.
6. **Regular examination of all ancillaries:** All ancillaries on the slipform should be examined and regularly inspected after erection and during the operation period to ensure that sufficient safety and capacity are achieved.

11.6.3 Dismantling of Slipform

After the completion of the slipping operation, the dismantling of the slipform components begin. Dismantling of the slipform is as important as the assembly and sliding operation. Utmost care is required during dismantling and a systematic process has to be followed for dismantling. Throwing of even the smallest components from the top can have disastrous consequences. Some steps involved in the dismantling process are given below:

- The coil nuts/pipe sleeves are provided in the final layer of concrete.
- The staging brackets are fixed along with the hand rails and walkway planks.
- The load of the slipform is transferred to the brackets.
- The inside and outside hanging scaffolds are removed.
- The flying tie assembly is lowered.
- The jacks are removed.
- The yoke assembly consisting of the yoke legs, the yoke beams, and the waler shoes are removed.
- The inside and outside walers are removed. The form panels are also removed in parallel.
- The jack rods are extracted and holes are grouted.

Sometimes, hoists may be erected from the completed concrete walls to lower the slipform materials. The dismantling usually starts from the centre and proceeds in both the directions towards the access tower from where the slipform dismantling crew can come down.

11.7 SLIPFORM DESIGN ISSUES

Slipform design should ensure sufficient strength of form to withstand all the loads, and the safety of the personnel and the materials involved. The design should ensure sufficient rigidity. The forms and the various other ancillaries should have the required capacity and appropriate factor of safety.

It should be designed for all the special loading conditions including frictional forces between the concrete and the forms. Where loads from the slipforms are to be supported by the existing structures, the safety of such structures to all the load conditions should be taken into consideration.

The slipform design should be such that the forms do not deform the concrete harmfully after erection and during the operation. The design should also consider providing appropriate camber to the forms.

ACI 347 recommends that slipforms should be designed by experienced, competent engineers familiar with slipform construction. It also recommends that the drawings for slipform construction should be prepared by a slipform engineer employed by the contractor and it must show the jack layout, formwork, working decks, and scaffolds. The engineer should be familiar with the exact brand of the slipform equipment proposed to be used for slipform operation. The engineer should also prepare a developed elevation of the structure showing the location of all the openings and embedments.

The dead loads are computed as in any other formwork. When it comes to estimating live load, ACI 347 recommends a minimum design live load of 3.6 kN/m^2, or concentrated buggy wheel load whichever is greater, for the design of the sheathing and joists. The minimum design live load for designing beams, trusses, and wales used in slipform is assumed to be 2.4 kN/m^2, whereas the minimum design live load for designing light-duty finishers' scaffolding is recommended to be 1.2 kN/m^2.

The lateral pressure of fresh concrete to be used in designing the forms, bracing, and wales can be calculated according to the recommendation of ACI 347 as given below:

$$P_{\max} = c_1 + \frac{524 \times R}{T + 17.8} \tag{11.1}$$

where

$P_{\max}$ is the lateral pressure, kN/m^2;

c_1 = 4.8 kN/m^2 (For some applications, such as gastight or containment structures, the value of c_1 should be increased to 7.2 kN/m^2);

R is the rate of concrete placement, m per h; and

T is the temperature of concrete in the forms, °C.

11.8 SOME CASES IN SLIPFORM

Major contracting firms in India have successfully built many structures using both indigenous and imported slipform equipment. There are companies/organizations specialized in this construction method, and are equipped with the best hydraulic jack and yoke-system on the market today. These organizations have engineers and technicians who are experienced at finding economy in a construction project. They plan, design and construct the forms. Then they supply the equipment and the expertise required to assemble and lift the form at the site. The jacks with capacities such as three ton, six ton and twelve ton with climbing strokes adjustable from 20 mm to 60 mm allow varying form loads to be raised evenly. Jacks and yokes are designed to allow maximum flexibility in working deck and hanging scaffolds. This is crucial in designing the most efficient and economical slipform system for each particular project.

With their expertise, they inform the owner how slipform concreting will shorten the construction periods and affect the economies.

11.8.1 Chimneys

Tall chimneys are required to be built to disperse flue gases mixed with other pollutants at high altitudes. Single chimney with multiple flues is also designed these days to disperse pollutants from different boiler units. With stringent anti pollution regulations, the required heights of the chimneys are increasing day by day. It is common to have a 275 m high chimney for boilers of 500 MW or more. A number of such chimneys have been constructed in the country. A number of contractors are capable of constructing chimneys of such height using slipform.

A chimney of 275 m height, 30 m base diameter, and 20.8 m top diameter was built by using the slipform method at Anpara in U.P. For this, a massive structural frame consisting of an upper frame, a lower frame, bracings, main trusses, spider beams, and rig beams was erected (Fig. 11.20). The load of the concrete and the reinforcement to be lifted each day for such chimneys is considerable. Four rope guided hoists were installed on the slipform structural member to take care of the lifting of the top concrete, reinforcement, and access to the platform. The slipform system was designed to provide a lifting force of 336 tonnes to cater to live load, dead load, and friction load. Figure 11.21 shows the nearly completed view of the chimney.

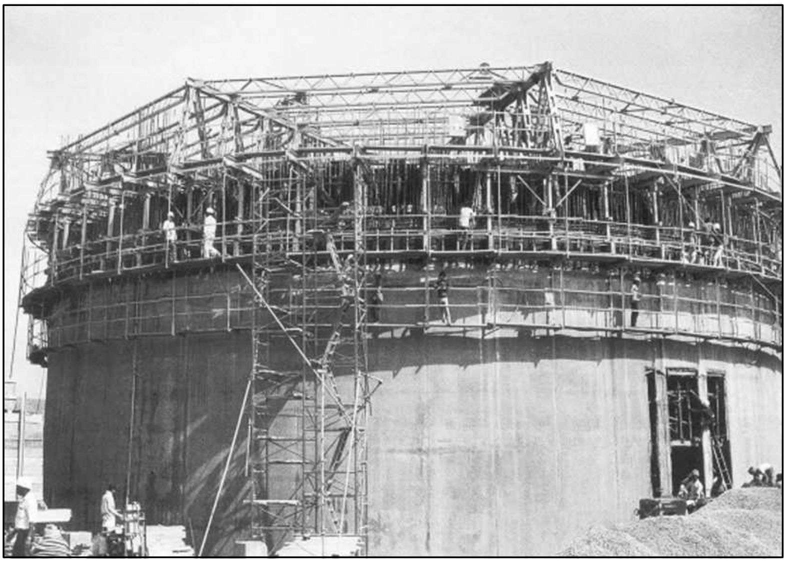


Figure 11.20 Slipform in Progress for the Chimney.

In order to accommodate the reduction in the diameter of the chimney with increasing heights, the bolted connections were designed which helped in dismantling some of the slipform panels during the slipping operation.

The contractor employed 8 main yokes and 40 normal yokes, 16 twelve-ton capacity slipform lifting jacks, 40 six-ton capacity slipform lifting jacks, and 1 hydraulic power pack for the slipform jack. Laser beams were used for vertical alignment. The contractor used 4 passenger/material hoists and two Space Clima for emergency access from the outside. For reinforcement lifting, 2 two-ton capacity gear winch were used. The contractor installed batching plant at the site to produce concrete which was transported to the chimney location in transit mixers.

Figure 11.21 275 m High RCC Slipformed Tapered Chimney at 2 × 500 MW Anpara Thermal Power Station in U.P.

11.8.2 Tall Bridge Piers

Slipform construction has been used quite successfully in the construction of tall bridge piers in the country. The cross section and heights of these piers vary depending on the requirement. Some of the construction of bridge piers has taken place in extremely difficult terrains.

Concrete pier of 80 m height was constructed using slipform for the Jammu Udhampur Railway bridge project. The slipform was carried out in extreme climatic conditions and in a hostile terrain. It was difficult to employ the equipments for transporting bulk materials such as aggregate and sand for producing concrete. The materials were transported using ponies. The pier structure was located near the river where flash floods and landslides were common. The pier in advanced construction stage is shown in Fig. 11.22.

Figure 11.23 illustrates the use of slipforms to construct the tallest viaduct in Asia across Panvel river in Ratnagiri, Maharashtra for the Konkan Railway Corporation. The cross section of the bridge piers was of hollow octagon shape, with reducing plan dimensions along the height of the pier. During the construction, a number of problems such as twisting of the slipform, improper cover for reinforcement and improper collapsing of form panels, etc. were faced by the contractor (Larsen and Toubro Limited, ECC - Construction Division). These problems were addressed appropriately and the completed structure was adjudged the 'most outstanding concrete structure' by the American

Concrete Institute (Maharashtra India Chapter). On the best day, in 24 hours, the crew completed 9.4 m of vertical height on the bridge pier.

Figure 11.22 Construction of Pier (Jammu Udhampur Raillink Project).

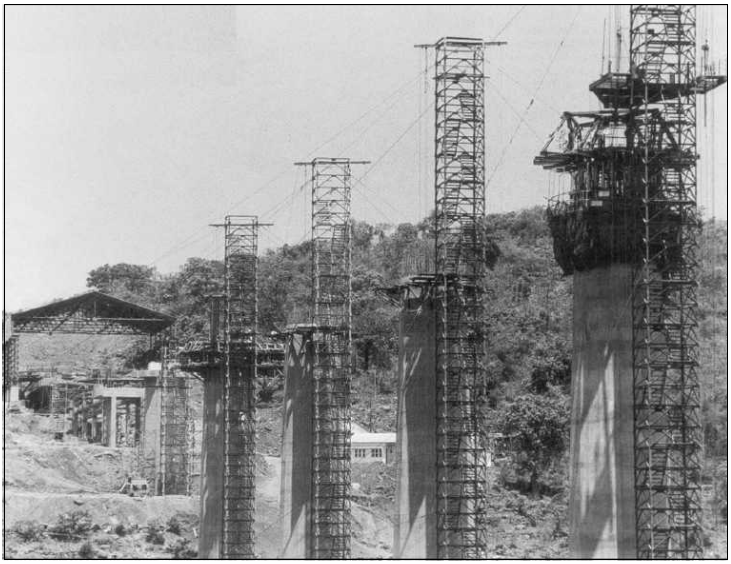


Figure 11.23 Slipforming of Octagonal Shaped RCC Bridge Piers in Progress for the Panvel River Viaduct at Ratnagiri.

11.8.3 Columns, Pylons, and Towers

An interesting work using slipform was performed for the construction of the Concrete Columns Piazza at Noida in U.P. The concrete column has a circular cross section and its diameter and thickness varies along the height of the column. For the first 2,955 mm, the column has 1,858 mm outer diameter and 550 mm thickness. Between 2,955 mm and 10,620 mm, the column has 1,650 mm diameter and 450 mm thickness. Between 10,620 mm and 18,832 mm, the column has 1,424 mm diameter and 350 mm thickness. Between 18,832 mm and 27,044 mm, the column has 1,200 mm diameter and 230 mm thickness.

The cross section and elevation of the column is shown in Fig. 11.24.

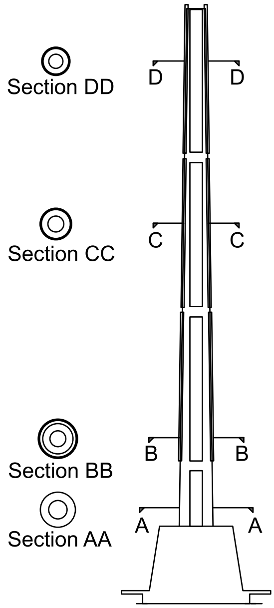


Figure 11.24 Cross-section and Elevation of Column.

The slipform arrangement adopted for the column construction is shown schematically in Fig. 11.25.

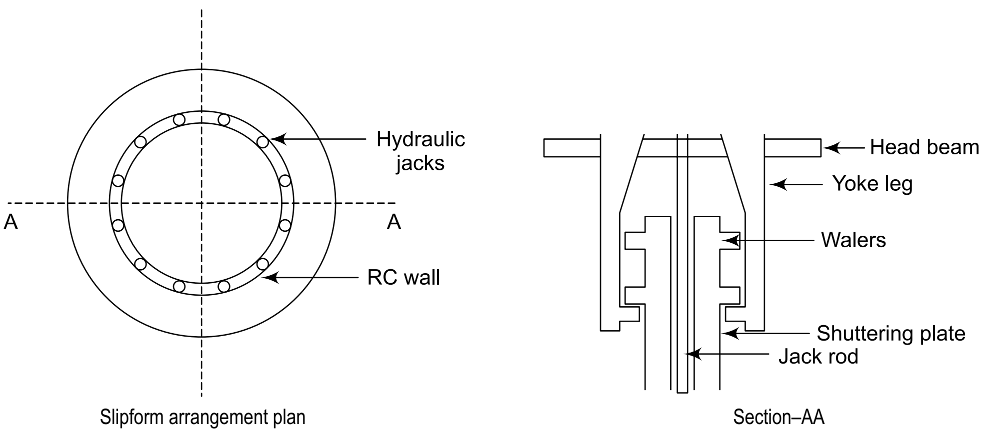


Figure 11.25 Plan and Section for Slipform.

The slipform was planned with a total of three intermittent stoppages before reaching to the completed height. The scheduled stoppages were at 2,955 mm, 10,620 mm, and 18,832 mm. At each of these stoppages, the slipform assembly was dismantled and re-erected to suit the changed column thickness. The project was executed by Shilpa Associates Pvt. Ltd. for the Noida Authority. The contractor used a set of eight hydraulic jacks of 3 ton capacity. The yoke legs were made out of ISMC 125 section while the head beam was made of ISMB 100 section. The suspended scaffold was fabricated using ISA 75 × 75 × 6 sections. The form plates were made of 5 mm thick mild steel plates stiffened with ISA 50 × 50 × 6 sections. A mild steel pipe 20 NB of 'C' class was used to serve as the jack rod. A mechanical winch supported on mild steel access tower was used to transport the concrete for the column construction.

The construction started with the casting of the concrete starters followed by the assembly of the slipform. As mentioned, the slipform operation had three planned stoppages at the changing thickness locations. At each of these locations, the slipform assembly was dismantled and reassembled to suit the changed thicknesses and the diameter of the column. Concreting was done in a layer of 400 mm and sliding was done in an increment of 10 mm stroke. Verticality of the structure was measured at every half meter height by suspending 10kg —4 Plumb bob and the correction on the jack movement was done by level controllers fixed at the top of the hydraulic jack .The total concrete quantity used for the project was 45 m^3 and the total reinforcement steel was 3.05 ton. M30 grade concrete had been used in the construction. The vertical reinforcement bars were of 25 mm diameter while for circumferential reinforcement, 10 mm diameter bars had been used. The reinforcing bars were of Stainless Steel (SS).

The different stages of construction of the column piazza are shown in Fig. 11.26.

Concrete pylons for boiler supports for a height of approximately 80 m were slipformed by Larsen and Toubro Limited, ECC - Construction Division (see Fig. 11.27). The span between the pylons was around 22 m which were constructed using a walkway bridge. Pylons were constructed together with an accuracy of plus or minus 10 mm to accommodate the supporting girder with 52 holes to match with the bolts emanating from the pylon. Also, precast staircase was erected in a short span of 15 days. Slipping of the pylons resulted in an aesthetic surface finish and a clean, maintenance free concrete pylon.

11.8.4 Elevator and Stair Core

The elevator and stair core of the high-rise buildings is one of the most important elements. Fast and efficient construction of the concrete core of a high-rise building is essential to maintain phased progress in other parts of the building. Different formwork arrangements are possible to construct these high rise cores. These are: slipform, jump form, climb-form, super-shafter, and conventional forms. Slipforms have been used extensively in the past few decades for the construction of the elevator and stair core of high rise structures. According to Jaafari *et al*. (1989), for building cores of less than 15 stories high, none of the alternative methods can compete with the conventional forms. They further stated that for tall structures (greater than 30 stories), the alternative methods could potentially reduce the cost of the conventional method by up to 30-40%. The slipforms showed cost advantages for more than 20 stories and for area larger than 600 m^2 formed area per floor.

Figure 11.26 Different Stages of Construction of Column Piazza.

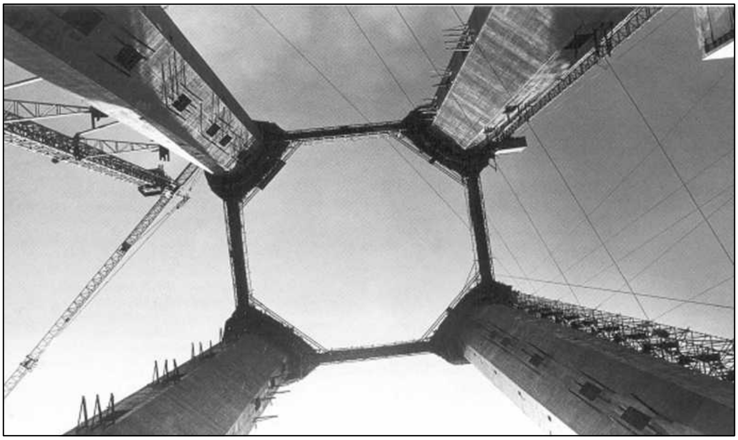


Figure 11.27 Simultaneous Slipforming of Four Pylons at Vijayawada Thermal Power Station.

In one of the slipform methods of core construction, the slipping is not done on a continuous 24 hour per day basis. Rather, the slipforms are started and stopped each day. The slipform operation remains suspended on holidays. Specific formworks for the openings of elevators—mechanical, and electrical are constructed in the duration of the stoppage. Usually at this stage, vertical reinforcements are installed with the appropriate lengths to cover one floor.

In another method of slipform construction of the core, the slipping is done on a continuous basis. For connection at different floor levels, dowel bars or insert plates are left inside the concrete during slipping. These dowel bars or inserts are exposed later on the completion of the slipping operation and suitable connections from the beams and slabs at different floors are made with the core wall.

Slipform was used to construct the 14 elevator and stair core of the Bharat Hotels (Hilton), New Delhi in 1982. The height of slipforming was 54.13 m and the inside plan dimension of the core measured 2.5 m × 2.4 m. The wall thickness of the core varied from 230 mm to 180 mm. Insert plates were provided on the face of the elevator core wall at respective levels. These plates were later used to connect the end of the floor beams with walls after the completion of the slipform. Similar arrangements were made for the stair core also. Temporary block-outs were provided during slipform concreting for elevator door openings. Figure 11.28 shows the construction of the core using slipform.

11.8.5 Silos

Silos are constructed for storing grains, and the construction and the industrial materials. Being circular in shape, generally these silos represent the ideal cases for the slipform technique of continuous construction, especially when the height of the silo wall is very high. According to Jaafari *et al*. (1989), the slipform method is the most economical and time saving technique for constructing silos higher than 15 m.

Construction of circular silos is a common feature in cement plants. These are built to store clinker, cement, etc. Indian contractors have slipformed silos of different diameters and heights in different conditions successfully. Silos of fluted designs have also been built by the slipform method.

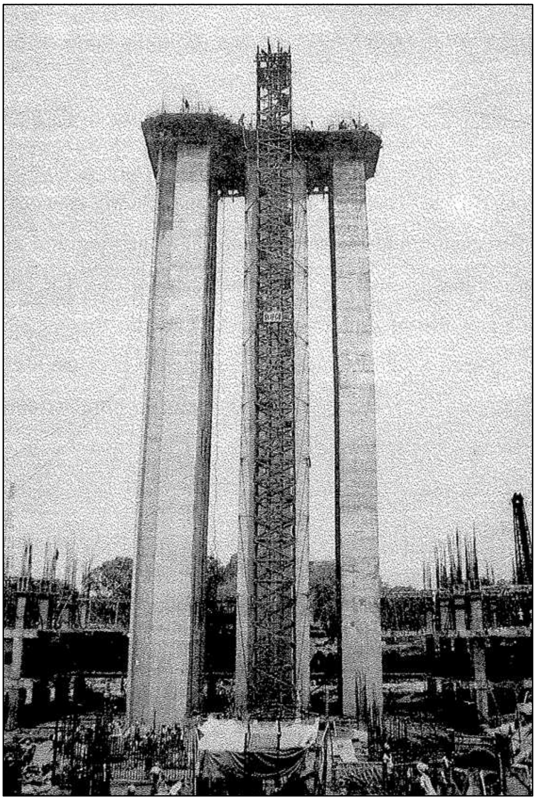


Figure 11.28 Bharat Hotels Elevator Core Under Construction Using Slipform.

A number of silos with architectural grooves resulting in aesthetic surface finish have been constructed using slipforms at Tadapatri, Nimhada, and abroad in Oman by the Indian contractor Larsen and Toubro Limited, ECC - Construction Division. One such silo constructed at Tadapatri Cement Plant is shown in Fig. 11.29.

A 65 m diameter silo to store clinker was constructed using slipform technique for the Andhra Pradesh Cement Plant in Tadipatri. Such large diameter silos were constructed using the climbing formwork technology earlier. Slipform structure was specially designed to cater to the large diameter. The total height of slipping was 25.5 m.

The typical slipform arrangement for constructing silos of low to medium diameter (say up to 36 m) consists of centrally connected spider assembly (yokes) and central spokes made up of 12 mm M.S. round bars. One end of the spoke rods are connected to a central ring while the other end is connected to the waler pipes of the slipform shutters with an adjustable spindle. During the slipform operation, these spokes are kept intact through turnbuckles and they maintain the circular shape. This arrangement would become very heavy and expensive if used for slipforming large diameter silos.

The arrangement of slipform for constructing 65 m diameter silo consisted of the turnbuckle assembly and avoided the usual central spokes mentioned above. The turnbuckle assembly is similar to a horizontal frame running parallel to the structural wall in the plan on any or both sides of the yokes. The turnbuckle arrangement provides structural integrity to the entire slipform assembly and provides resistance to the differences in yoke levels. The arrangement prevented local buckling of walers and form panels.

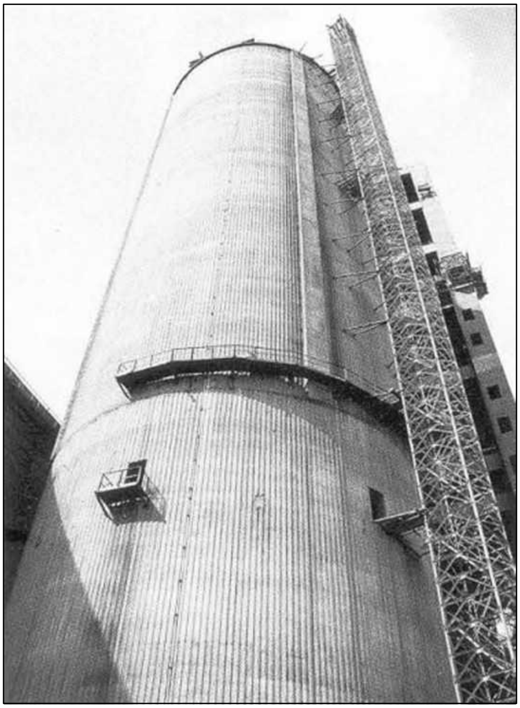


Figure 11.29 Slipformed Grooved Silo at Tadipatri Cement Plant.

Pre-stressing cables weighing up to 1 ton were placed in the shell, while slipping. The slipform assembly consisted of 154 yoke elements and 166 slipform hydraulic jacks each of 3.5 ton capacity. The construction involved 2,260 m^3 of concrete pouring, 10,000 m^2 of slipformed area, and tying 650 tonnes of reinforcement and 20,000 m of pre-stressing cables. In each shift of 12 hours, the contractor mobilized 165 workers for the concreting work, 35 workers for slipform shuttering operations, 70 workers for the reinforcement works, and 80 workers for pre-stressing, 18 mates, and 5 engineers. The structure rose to 25.5 m slipping height in 15 days. The contractor had mobilized 3 access cum concrete lifting towers, 3 winches for lifting reinforcement, 1 winch for lifting the pre-stressing cables, and 6 concrete mixers. This has resulted in a direct saving of 2-2.5 months in construction time and also considerable cost.

The pre-stressing and reinforcement tying operations are critical in constructing silos of this nature. The lifting arrangement of the pre-stressing cables, placing pre-stressing sheathing pipes during concreting, and post threading the pre-stressing cables after the completion of the slipform, need careful consideration. Mock assembly of cables installed before commencing the slipform operation helps in foreseeing the problem. Specially designed concrete mixes are required to cater to the speed of slipping. In this case, the contractor had designed M-40 concrete mix to have initial setting time between 2 to 2.5 hours and final setting time between 6 to 7 hours.

Coordinating such a large crew dealing in different activities is also a tedious task. It requires detailed planning and monitoring. Efficient communication devices such as intercoms and walkie talkies (used earlier), and mobile phones prove helpful in coordinating with different crew leaders. The completed silo is shown in Fig. 11.30.

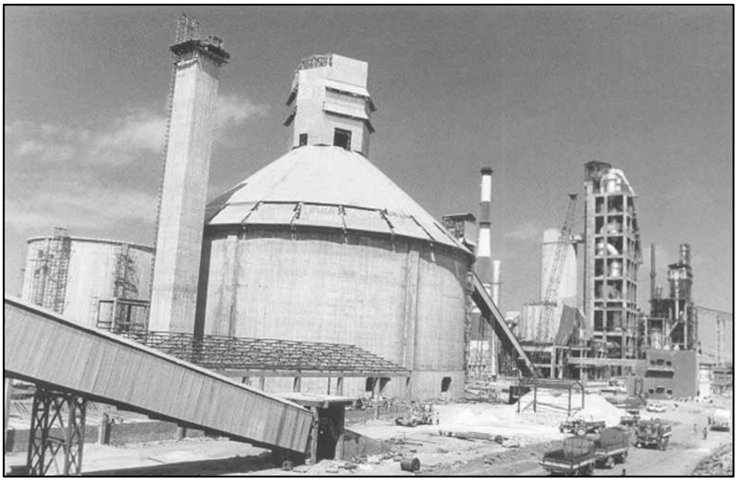


Figure 11.30 Slipformed Silo (65 m diameter) at Tadipatri Cement Plant.

11.8.6 Slipform for RCC Framed Structures

Though slipform for RCC framed structures are not very common, they have been employed by Indian contractors to construct pipe racks and pre-heater buildings of cement plants.

RCC framed structure with columns and tie beams were slipformed together for the construction of pipe racks at RPL Jamnagar. The pipe racks consisted of shear bays which were rectangular in plan. The shear bays had a set of 4 columns with tie beams in between. In comparison to the traditional methods of formwork used for constructing pipe racks, the contractor claims to have saved between 4 to 5 months in construction time besides a cost saving of about Rs. 40 lakhs.

A 61 m high pre-heater building was constructed using the slipform technique at the Awarpur Cement Plant in Maharashtra by Larsen and Toubro Limited, ECC - construction division. The building was designed with load bearing walls to accommodate construction using slipform technique. The contractor could complete the slipform despite working in peak summer when the temperature reached nearly 48°C. Due to the high temperature, the slipform jacks imported for the slipform started leaking. This problem was however overcome by replacing the hydraulic oil and keeping the cylinder portion of the jack in hydraulic cool condition by spraying water. Another problem faced by the contractor was the jamming of the shutter due to excessive frictional force, which was also overcome.

After the construction of the pre-heater structure at Awarpur Cement, a number of similar structures were taken up for slipforming in the country. Figure 11.31 shows the slipformed rectangular pre-heater building with window openings for Madras Cements at Jayanthipuram.

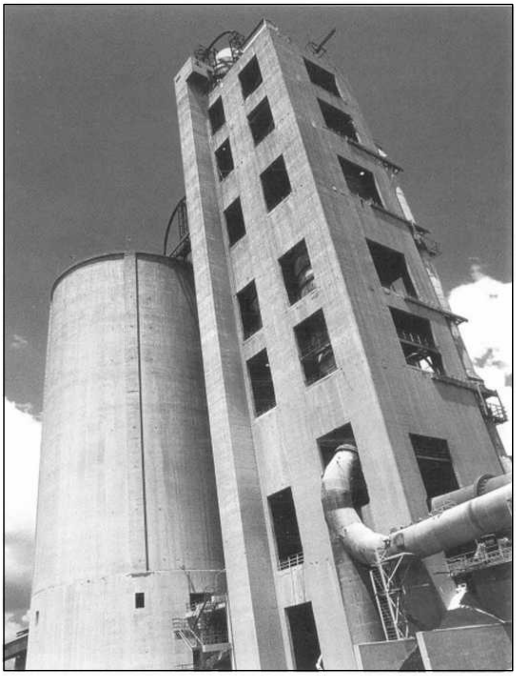


Figure 11.31 Slipformed CF Silo and Pre-heater at Madras Cements at Jayanthipuram.

11.9 SAFETY OPERATIONS DURING SLIPFORM ERECTION

11.9.1 General

(a) Wearing safety helmet (hard hat) by all is imperative.
(b) It is preferable to have one entrance to enter the chimney. One security guard should be posted to prohibit anyone entering without the helmet and also the trespassers.
(c) The entrance should be sheltered to a length of 15 m outside the chimney and inside up to the R.G hoist; so that the entrance is saved from the falling material.
(d) Barricading is required around the chimney circumference at a distance of 15 m.
(e) Inside the chimney, at the locations where perpendicularity is checked by laser beam, should be sheltered with a small hole.
(f) Deploy minimum number of workmen on the platform, and as far as possible have them scattered.
(g) Smoking should be prohibited on the platform.
(h) Fire extinguishers should be kept on the platform. The distance to reach a fire extinguisher should be maximum 25 m.
(i) Safety posters and caution boards should be displayed on the platform and at the ground level.
(j) Intercom for communications is very much required.

(k) It is important to give a safety talk to the workers by the concerned engineer on the procedures before the slipform start.

11.9.2 Platform

(a) The platform at all levels should be checked by a competent person for its reliability on bolting, plank fixing, etc. A periodic check at a frequency of once a month is a must.
(b) The platform should have a handrail and toe board.
(c) All planks should be checked. The recommended size is 1,800 mm × 250 mm width × 50 mm thickness —preferable variety is hardwood.
(d) The external and internal platform bottom should be covered with safety nets.
(e) Access ladder is required to climb down from the platform to the bottom platform.

11.9.3 Electrical

(a) All electrical equipments should be earthed.
(b) Earth leakage circuit breaker (ELCB) should be fixed to portable equipment and lightings.
(c) The electrical cable should be neatly traced along the platform and handrail. Avoid placing them on the platform or routing in a haphazard way.
(d) Provide lightning arrester as the slipform proceeds connecting other earths to the lightning arrestor earth is prescribed.

11.9.4 Rope Guided (R.G.) Hoist

(a) R.G Hoist should be operated after fixing the safety devices.
(b) Two days are required for its trial run with and without load, to set the limit switches.
(c) The hoist should never be loaded more than the prescribed safe load. A board shall be displayed indicating the maximum number of persons allowed to travel.
(d) Deploy trained personnel as the operator and signal man.
(e) The path of the main hoist's winch rope shall be barricaded on either side. Ramp should be provided for cross over.
(f) Strict schedule of maintenance should be kept for hoist deflection pulleys, wire ropes and winches.

11.9.5 Reinforcement Crew

(a) Deploy, experienced and designated operator for winch operation.
(b) The access to pick up the reinforcement rods from the winch bucket should be safe.
(c) Stacking of reinforcement rods should be uniformly distributed on the platform, and they should be placed on the sleepers such that the load is transferred to the spider beam and not to the planks.
(d) The quantum of reinforcement rods stacked should not be more than that required for a shift.

11.9.6 Miscellaneous

(a) It is easier and less risky to do corbel casting during the slipform operation. At times, the platform is moved above the casting and workers are sent down to remove the shutters and to do the finishing work. These workers should be equipped with a safety belt of sufficient length and a kit bag to keep the spanners, bolts and nuts. It is also recommended to have one additional level of suspended platform to carry out such special activities.

(b) Outside platforms should be cast while doing the slipform. The workers deployed should have the required personnel protective equipment (PPE).

(c) Housekeeping on the platform should be maintained stringently. No material should be dropped from a height.

(d) The shutter cleaning gang should be instructed to clear the platform at the end of the shift and the collection debris should be brought down through the hoist.

(e) The bit rod cut by the gas cutter or electrode bits generated by the welders should be collected by the respective helpers and brought down at the end of each shift.

11.10 PRODUCTIVITY ISSUES IN SLIPFORM CONSTRUCTION

Zayed *et al*. (2008) studied the productivity, slipping speed, and the resource combinations for the slipform construction of silos and elevated core construction. They used simulation and sensitivity analysis in their study. The discussion in the following sections is limited to productivity issues in the construction of silos. For productivity issues in core construction using slipform, the readers can refer to Zayed *et al*. (2008).

The model takes into account the productivity affecting factors such as: stoppage times, jacking rates, silo diameter, placing method, cross section area represented by diameter of the silo and thickness of the walls, and concrete setting time. The jacking rates depend mainly on the various concrete setting times and the power of the jacks. Activity durations for silo construction were estimated based upon: maximum possible jacking rate according to the mechanical capability of the form, mix design and concrete properties, concrete placing method, rebar installation, and material lifting technique. Rebar installation time is determined based on a crew of eight rebar workers (rodmen). Concrete placing time is determined based on the crane and bucket rates. Material lifting time is based on the average crane lifting speed.

The study resulted in a set of charts which can be used by the planners to prepare the schedules and find out the resources required for slipform application in silo construction. They can also be used by the estimators in the bid estimation process.

Figures 11.32 and 11.33 can be used to determine the productivity for a given silo diameter and thickness, and jacking rate. The productivity is obtained in m/h. Figure 11.32 is applicable for concrete placing using bucket and crane. It can be seen that for a silo of 16 m diameter and 0.5 m thickness, the slipform productivity is about 0.15 m/h at 30 cm/h jacking rate. It is assumed that work takes place 24 hours a day.

Figure 11.33 can be used for finding slipform productivity for different silo diameter and thickness, and jacking rates. The charts shown in Fig. 11.33 are to be used when it is planned to place the concrete using the concrete pump. It can be observed that for the silo diameter of 16 m and 0.5 m thickness, the slipform productivity is about 0.214 m/h at a jacking rate of 30 cm/h.

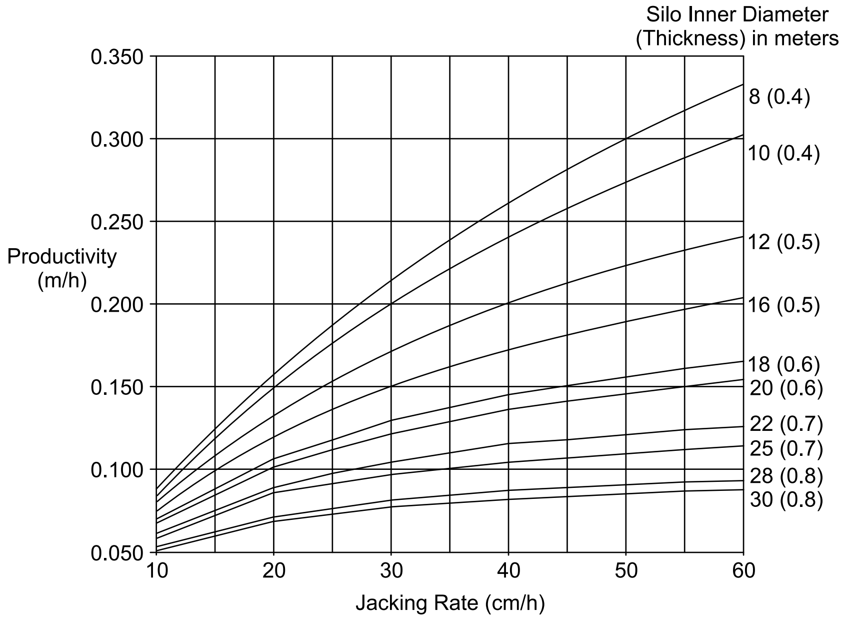


Figure 11.32 Slip-forming Productivity Versus Silo Diameter Using Bucket and Crane in Concrete Placing (24 working h/day) Courtesy American Society of Civil Engineers. License No. 2885870881477.

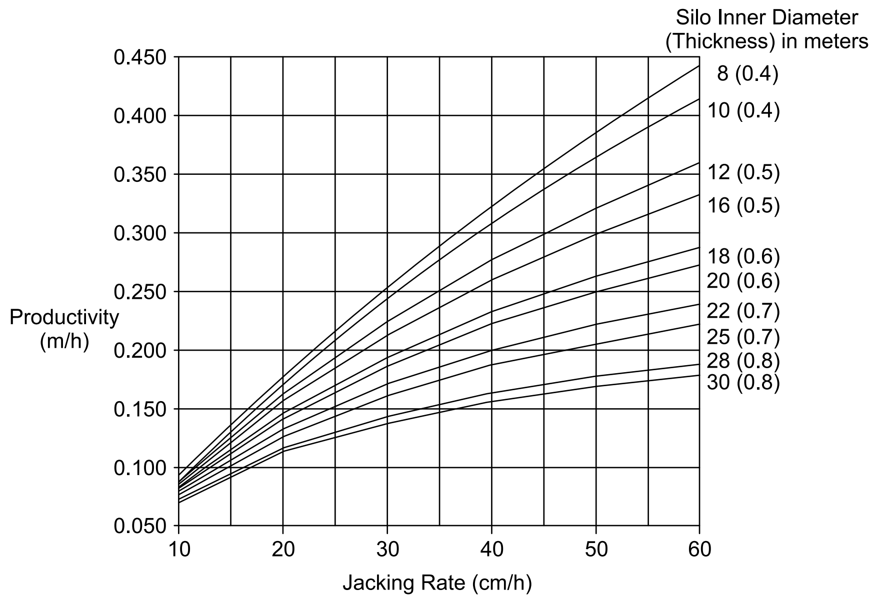


Figure 11.33 Slip-forming Productivity Versus Silo Diameter Using Pump in Concrete Placing (24 working h/day) Courtesy American Society of Civil Engineers. License No. 2885870881477.

The productivity of different silo projects, using the crane and bucket arrangement or pump to place the concrete can be predicted easily using Figs. 11.32 and 11.33. This is shown with the help of the following small illustration.

Example: It is desired to estimate the duration for a project which involves slipform construction of twelve silos of 22 m diameter and 0.7 m thickness, and eight silos of 12 m inner diameter and 0.5 m thickness. Each of the twenty silos is of 55 m height. It is proposed to use a jacking rate of 50 cm/h and concrete is planned to be placed using the crane and bucket arrangement.

Since crane and bucket arrangement is proposed to be used, Fig. 11.32 needs to be referred for estimating the project duration. From Fig. 11.32, the slipform productivity for a silo of 22 m diameter and 0.7 m thickness is 0.121 m/h at 50 cm/h jacking rate.

Thus the time to construct twelve silos = 12 × time to construct one silo

$$= 12 \times \frac{\text{Silo Height}}{\text{Slipform Productivity}} = 12 \times \frac{55}{0.121} \text{ hours} = \frac{5{,}454.54}{24} \text{ days} = 227.27 \text{ days}$$

The slipform productivity for a silo of 12 m diameter and 0.5 m thickness is 0.223 m/h at 50 cm/h jacking rate.

$$= 8 \times \frac{\text{Silo Height}}{\text{Slipform Productivity}} = 8 \times \frac{55}{0.223} \text{ hours} = \frac{1{,}973.09}{24} \text{ days} = 82.21 \text{ days}$$

Thus, the total project duration for construction of 20 silos = 227.27 + 82.21 = 309.48 days. Some contingencies may have to be added in this value for duration estimation. It may be noted that only one set of slipform equipment and crew has been considered for the above computation. The duration would decrease proportionately if more slipform equipments are mobilised. For example, if the slipform equipments and crews are doubled, the duration would become half of what has been estimated.

For computing the man hours of different crew members required for a few activities in slipform, the following productivity values can be taken for planning purpose.

1. **Assembly and dismantling of slipform:** The assembly of straight slipform can take about 75 man hours per yoke set. The dismantling of straight slipform can take on an average 55 man hours per yoke set. Thus, if the slipform assembly consists of 24 yoke sets, the total man hours required for assembly and dismantling would be on an average 75 × 24 + 55 × 24 = 3,120 man hours.
2. **Slipping operation:** For straight slipform, the man hour requirement comes to about 1.25 man hours per m^2. It can go to 2.50 man hours per m^2 for tapering slipform.
3. **Concreting activities:** For planning purpose the man hour requirement for straight slipform concreting can be estimated based on a productivity value of about 22 man hours per m^3. It can be slightly increased (about 25 man hours per m^3) in case of tapering slipform.
4. **Reinforcement:** For straight slipform, the productivity is about 65 man hours per m. It can increase to 90 man hours per m for tapering slipform.

REVIEW QUESTIONS

Q1. True or False

(a) Slipform construction is similar to the extrusion process in which the wet concrete is extruded rather than retained in forms until it is hardened.

(b) Slipform construction method is used to build high rise structures quickly.

(c) In slipform, the form moves semi continually with respect to the concrete surface.

(d) Slipforming is resorted to where continuous concreting is possible and where monolithic structure is desired.

(e) Slipform reduces construction time and labor cost compared to other formworks.

(f) Vertical slipform system essentially comprises of form panel, yoke assembly, jack rod, mechanical or hydraulic jack.

(g) Horizontal slipform generally moves on a rail system.

(h) Slipform can be classified as— straight slipform, tapering slipform, and special application slipform.

(i) Sheathing is used to contain and provide shape to concrete.

(j) Wales are used to support and hold the sheathing in place.

(k) Yokes support the wales at regular intervals with their legs, transmit lifting forces from jacks to wales, and resist lateral forces of plastic concrete within form.

(l) Yoke leg is used to lift the slipform structure as one integral unit.

(m) Yoke beam is the main connecting member between the inside and outside yoke legs.

(n) Jacks provide the force needed to raise the entire the slipform system and climb up the jack rods.

(o) Slipform design should ensure sufficient strength of form to withstand all the loads and the safety of the personnel and the materials involved.

(p) Slipform should be designed for all special loading conditions including frictional forces between the concrete and forms.

(q) Slipform design should be such that the forms do not deform the concrete harmfully after erection and during operation.

(r) For slipform design, as per ACI 347: maximum design live load = 3.6 kN/m^2 and minimum design live load = 2.4 kN/m^2.

(s) For slipform design, lateral pressure is given by the expression: $P_{\max} = \dfrac{524 \times R}{T + 17.8}$

Q2. Match the following

(i) Vertical slipform is used for	(a) canal lining, tunnel inverts, highway projects, water conduits.
(ii) Horizontal slipform is used for	(b) silos, bins, shafts, cores, caissons, bridge piers, etc.

Q3. Prepare short notes on

(a) Straight slipform

(b) Tapering slipform

(c) Special applications slipform

(d) Function of various components of slipform

Q4. List out the various steps involved in assembly, sliding, and dismantling of slipform.

Q5. What are the important issues of concern as per ACI 347 during slipform operation?

Q6. Prepare short notes on

(a) Chimney construction using slipform
(b) Tall bridge pier construction using slipform
(c) Columns, pylons, and tower construction using slipform
(d) Elevator and stair core construction using slipform
(e) Silos construction using slipform
(f) Construction of RCC framed structures using slipform

Q7. List out the safety issues involved in slipform erection.

Q8. Chalk out the productivity issues in man-hours with respect to

(a) Assembly and dismantling of slipform
(b) Slipping operation
(c) Concreting activities
(d) Reinforcement

Q9. Why is the pressure on side forms less in slipform compared to the pressure on wall form in a conventional construction?

Q10. Why is batter provided in constructing form for the slipform?

Q11. Calculate the schedule (completion time) and manpower requirement for constructing a 20 m diameter and 56 m high silo using slipform.

Q12. Give briefly the functions of following slipform components:

(a) Yoke legs
(b) Yoke beams
(c) Climbing rod
(d) Walkway brackets (inside and outside)

Q13. Estimate the project duration for a project which involves slipform construction of 15 silos of 18 m inner diameter. The height of silos is 65 m and wall thickness 0.6 m. You may assume a jacking rate of 30 cm/h. The concrete is planned to be placed using concrete pump. There is one set of slipform equipment and crew.

Q14. Estimate the project duration for a project which involves slipform construction of ten silos of 28 m inner diameter and 0.8 m wall thickness. The height of each silo is 60 m. The jacking rate is estimated to be 30 cm/h. The concrete is planned to be placed using crane and bucket arrangement. The contractor has plans to mobilize two sets of slipform equipment and crew.

Chapter

12 Formwork Supports

Contents: Introduction; Shores/Props and Dropheads; Multi-legged Shoring Towers; Design of Vertical Supports for Formwork; Work Input for Shoring Towers; Shoring Towers Reuse and Erection Sequence; Recommendations; Horizontal Supports

12.1 INTRODUCTION

In this chapter, vertical supports and horizontal supports for formwork are discussed. The vertical supports could be either single-leg type or multi-leg type. The single-leg type is also known as shore or prop. They are generally tubular and telescopic type. They also come with height adjustment features for adjustment in heights. The shores/props are available in different sizes to cater to different ranges of heights and different load capacities. Examples of multi-leg type supports are frame or tripod or trestle. In horizontal supports, the light weight trusses are discussed. The detailed discussion on shores/props, frames, trestles, etc. follows.

12.2 SHORES/PROPS AND DROPHEADS

12.2.1 Shores/Props

In chapter 2, the application of timber as vertical shores/posts was illustrated. The timber shores/props are also commonly used in low rise construction where floor to floor height may be between 3-4 m. It was emphasized that the *l*/*d* ratio is an important design parameter for timber shores and thus while using timber shores, sufficient thickness of timber should be ensured to safeguard them against buckling. Bracing of shores should also be ensured. A typical arrangement for timber shores/props is given in Fig. 12.1. Note the arrangement of jointing of shores/props to achieve additional heights (Fig. 12.2). The design aspects of timber shores/props are covered in Chapter 7.

The shores/props made up of steel are very common these days. In the early development of props, they were usually made up of mild steel tubes. Modern props are made with high tensile steel and tubes. They are of light weight yet offer high strength. The props are usually used for supporting the formwork in low clearance construction, up to heights of about 5 m. Beyond this height also, they can be used. However, in these cases, they have to be used in tiers and need to be properly tied and braced so as to form a rigid structure.

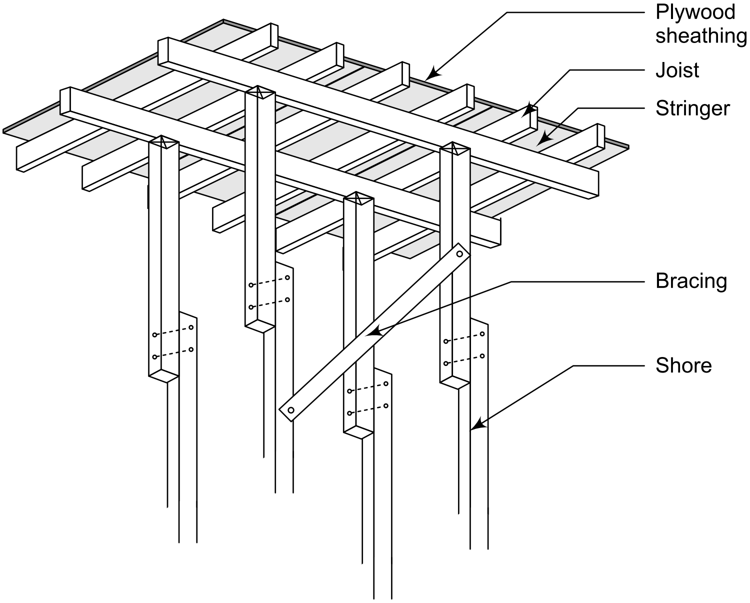


Figure 12.1 Timber Shore Arrangement (Note the Bracing of Shores).

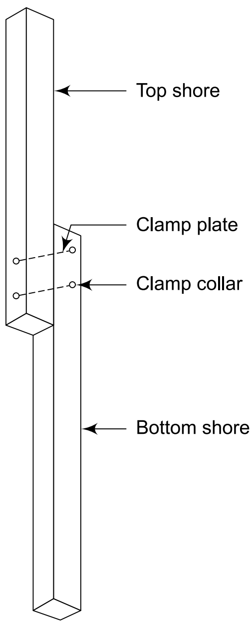


Figure 12.2 Arrangement of Jointing of Shores/Props.

As in the case of timber shores, bracings are also important in steel shores. Bracings are provided by means of tubes and clamps. There are a large number of manufacturers who produce steel props. These props come in different designs and have different load carrying capacities. The manufacturer's advice must be taken into consideration when using these shores. The design data provided by them prove to be valuable for the design. Their advice on the erection procedures of the shores and the safety precautions in their application must be adhered to. The details of some commonly available steel props by one leading manufacturer, L&T, are given in Table 12.1.

Table 12.1 Details of Props Manufactured by L&T

Prop name	Minimum (m)	Maximum (m)	Weight (kg)
CT 250	1.41	2.5	14.13
CT 300	1.71	3.0	16.35
CT 340	1.91	3.4	17.81
CT 410	2.31	4.1	21.00

12.2.2 Dropheads

Dropheads are devices which are fitted on the top of props or supports to support the slab while the remaining form for the decking could be struck for reuse. The dropheads remain in contact with the underside of the slab at all times for the full curing period, while the majority of the formwork materials, including plywood, can be removed as early as 3-4 days after pouring of the concrete.

This results in great economy as for as the cost of the formwork is concerned. Dropheads are a comparatively recent development and different manufacturers have come up with various proprietary designs of dropheads which essentially serve the same purpose. One such typical drophead by a reputed manufacturer is shown in Fig. 12.3.The arrangement of the drophead before and after striking the forms is also shown in Figs. 12.4 and 12.5, respectively. The dropheads are suited primarily for flat slabs and multi-story construction.

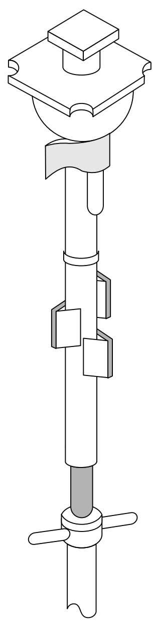


Figure 12.3 A Drophead.

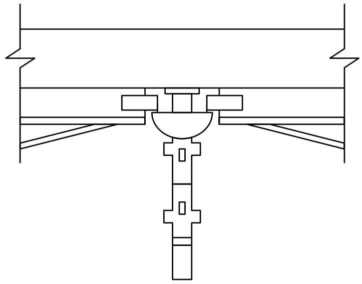


Figure 12.4 Drophead Supporting Beams in Raised Position.

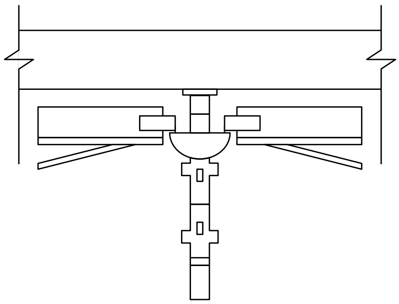


Figure 12.5 Drophead in Struck Position.

12.3 MULTI-LEGGED SHORING TOWERS

The commonly used multi-legged shoring systems for high clearance construction (floor to floor heights are considerable) are the 'frame based system', the 'tube and coupler system', and the 'trestle system'. Among the multi-legged support systems, the common ones are the multitier shoring towers, also known as the load towers or the support towers. As an alternative to the shoring towers, some other systems such as the 'drawer' or the column mounted formwork system have also been developed. These have been discussed elsewhere in the text.

The 'Frame based system' relies on pre-fabricated tubular frames which come in a variety of shapes and modular sizes which can be assembled one over the other to get the required heights. They can be spaced suitably depending upon the loads to be carried. The frames are usually braced together by means of ledgers and cross braces to form a rigid structure. In order to achieve varying heights, i.e. for adjustments in height, accessories such as screw jacks (tower spindle) are provided either at the top or the bottom or both.

Whether it is a commercial, a residential, or an industrial building construction, shoring towers find their application in almost all the types of construction. In addition to their primary function of providing support to the formwork, shoring towers are also used for the erection of precast elements, and as access scaffolds for providing access to the workers, tools, and the materials. A large number of proprietary towers, mostly of steel or aluminum with different configurations and classifications are available. There are a large numbers of companies manufacturing these shoring towers. These towers differ from each other on account of (1) tower configuration, (2) frame configuration, (3) dimensional variation of frames, (4) load carrying capacity, and (5) the way in which they are assembled.

Hurd (1963) classified the shoring towers based on the safe working load of each leg of the tower. She termed the towers as standard, heavy duty, and extra heavy duty. For a standard tower, the safe working load of each leg is 27 kN; for heavy duty tower, the safe working load of each leg is 45 kN; and for extra heavy duty tower, the safe working load of each leg is 180 kN. With the introduction of aluminum towers and towers with ultrahigh-load carrying capacity, the safe working load of each leg has increased up to 450 kN and the classification of standard, heavy duty and extra heavy duty may look obsolete in today's context. Further, the service lifetime of the shoring tower material has increased to 15-20 years now.

The classification of shoring towers adopted by Hurd (1963) and Shapira and Raj (2005) is shown in Fig. 12.6 and discussed below:

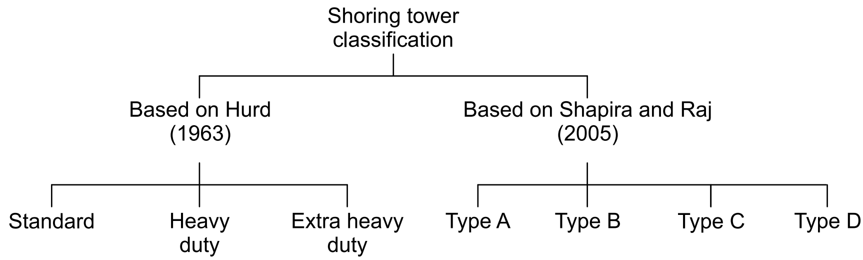


Figure 12.6 Classifications of Shoring Towers.

Shapira and Raj (2005) classified shoring towers in the following four types:

Type A

Type 'A' tower is shown in Fig. 12.7. The basic configuration of the frame for Type 'A' towers is shown in Fig. 12.8. Such towers can be assembled in square and rectangular shapes. Each tier is made up of two parallel frames which are connected by two pairs of cross braces. The cross braces are used to interconnect two separate towers or to make a larger tower.

The towers of type 'A' are manufactured by Acrow, Doka, Symons, and so on. Depending on the manufacturer, the height of each tier in such towers may vary from 910 mm to 2,100 mm. The working load per leg of the tower may vary from 45 to 80 kN. Such towers are available in aluminum and painted steel. The length of the frame (tower width) varies from 610 mm to 1,830 mm. There is an option of the whole tower section getting lifted as in the Gang form. The towers cannot be assembled in the triangular tower form.

Type B

Type 'B' tower is shown in Fig. 12.9. The basic configuration of the frame for Type 'B' towers is shown in Fig. 12.10. Each tower section is made up of four telescopic props. The props are connected by sets of four ledger frames. The props may also be used separately as single post shores. The ledger frames are used to interconnect two separate towers or to make a larger tower. The towers of type B are manufactured by Meva, PERI, and so on.

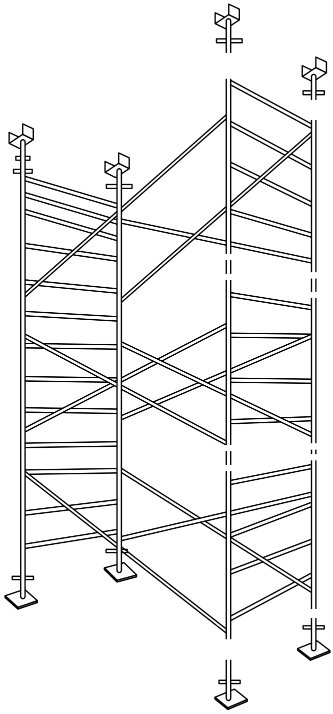


Figure 12.7 Type 'A' Tower.

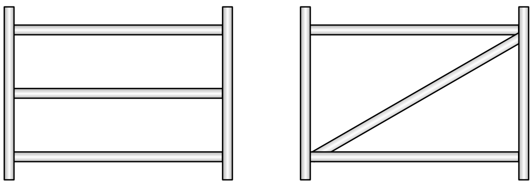


Figure 12.8 Configuration of Frame for Type 'A' Tower.

Depending on the manufacturer, the height of each tier in such towers may vary from as low as 800 mm to as high as 6,250 mm. The working load per leg of the tower may vary from 45 to 60 kN. Such towers are available in aluminum and painted steel. The length of the frame (tower width) varies from 550 mm to 3,800 mm. There is an option of the whole tower section getting lifted as in the Gang form. The towers cannot be assembled in the triangular tower form.

Type C

Type 'C' tower is shown in Fig. 12.11. The basic configuration of the frame for Type 'C' towers is shown in Fig. 12.12. These towers come in square configuration in the plan. As in the case of Type 'A' towers, each tier is made up of two parallel frames. In Type 'C' towers, the tiers are turned 90° in relation to each other. The ledger frames are used to interconnect two separate towers or to make a larger tower. The towers of type 'C' are manufactured by Doka, PERI, and so on.

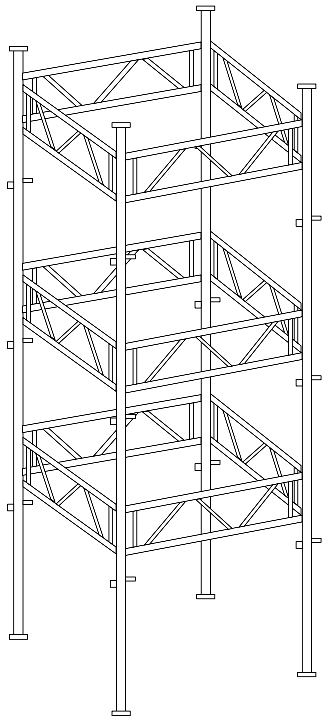


Figure 12.9 Type 'B' Tower.

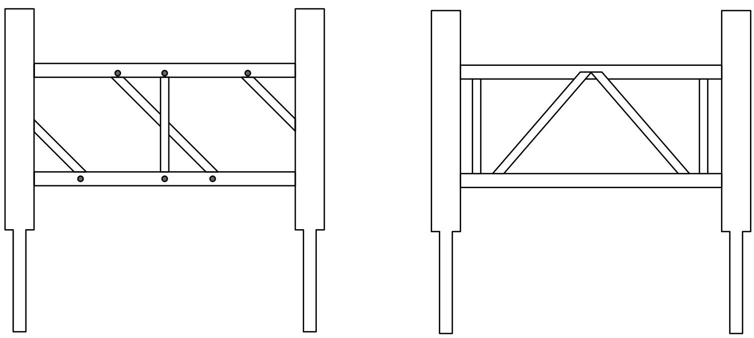


Figure 12.10 Configuration of Frame for Type 'B' Tower.

Depending on the manufacturer, the height of each tier in such towers may vary from a minimum of 500 mm to a maximum of 1,800 mm. The working load per leg of the tower may vary from 50 to 60 kN. Such towers are also available in galvanized steel. The length of the frame (tower width) varies from 1,000 mm to 1,520 mm. There is an option of the whole tower section getting lifted as in the Gang form for the towers manufactured by Doka. The towers of type 'C' usually cannot be assembled in triangular tower form; however some manufacturers can have the type 'C' tower assembled in the triangular tower form.

Type 'D'

Type 'D' tower is shown in Fig. 12.13. The basic configuration of the frame for Type 'D' towers is shown in Fig. 12.14. Such towers can be assembled in triangular, square, and rectangular shapes. Each tier is made up of four frames connected to each other. As in the other types of towers, these can also be interconnected to make a larger tower. The towers of type 'D' are manufactured by Kabir of Israel, Pal of Switzerland, and so on.

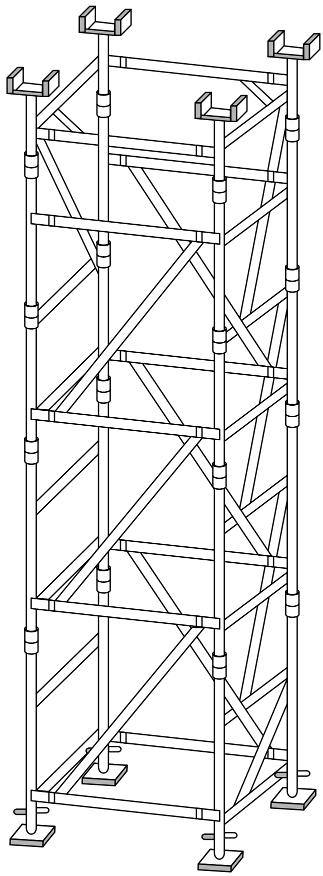


Figure 12.11 Type 'C' Tower.

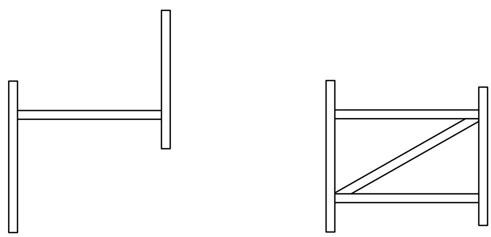


Figure 12.12 Configuration of Frame for Type 'C' Tower.

Depending on the manufacturer, the height of each tier in such towers may vary from a minimum of 500 mm to a maximum of 1,500 mm. The working load per leg of the tower may vary from 50 to 70 kN. Such towers are available in painted steel as well as galvanized steel. The length of the frame (tower width) varies from 1,000 mm to 2,000 mm. The gang form application (whole tower section getting lifted simultaneously) for such towers is not possible. The towers of type 'C' have the option of assembling them in the triangular tower form.

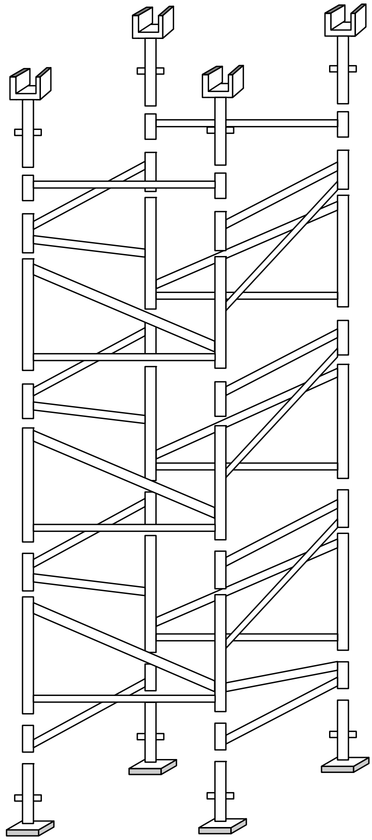


Figure 12.13 Type 'D' Tower.

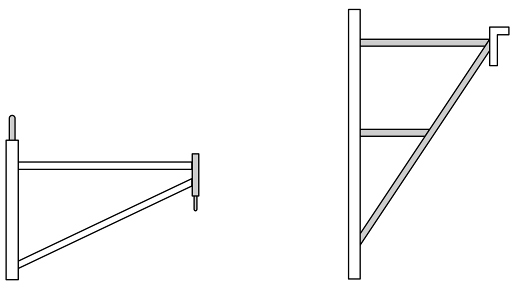


Figure 12.14 Configuration of Frame for Type 'D' Tower.

There is no tower which is ideally suited for each application and depending on the requirement of the constructor, a particular type of shoring system can be selected. There can always be more than one solution for the specific requirements of each project and the company.

12.3.1 Trestle (Crib) Shoring

The trestle (also known as crib) is also used to act as a shoring tower for heavy construction such as the bridge girders, slabs and culverts (Figs. 12.15 and 12.16). The trestle is usually made up of angle sections (ISA) and is braced appropriately. The angle sections are arranged in different patterns to suit the different requirements of carrying various loads. Some of the arrangements of angle sections to get the box shape are shown in Fig. 12.17. The boxes are of varying dimensions and they use varying number of angle sections. The load carrying capacity of each of the box sections varies. The angle sections are laced with each other to form the box section. The designer also needs to adhere to the codal provisions for doing the lacing design. The independent trestles (box sections) are also braced with each other as per the requirement. In both Figs. 12.15 and 12.16 however, these braces have not been shown for the sake of clarity.

The trestle is composed of box shaped members of some standard height which can be connected to each other with the help of bolting.

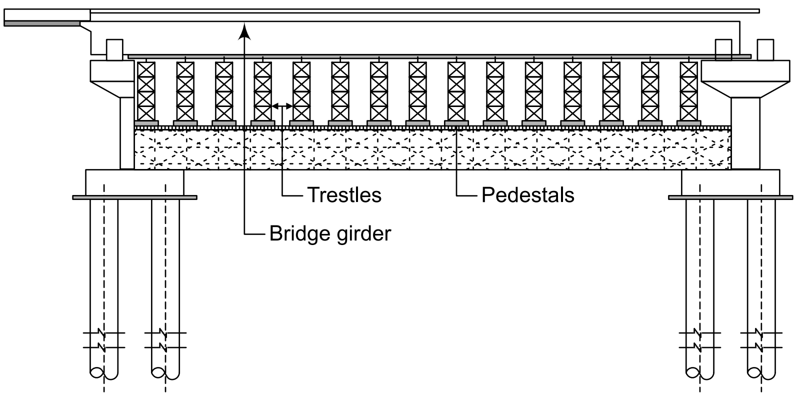


Figure 12.15 Trestles or Cribs as Shoring for Bridge Girder.

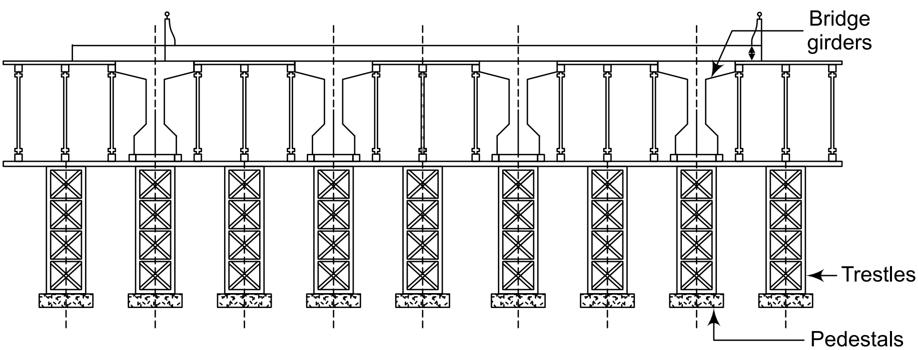


Figure 12.16 Another Arrangement of Trestles to Support Bridge Girders and Deck Slab.

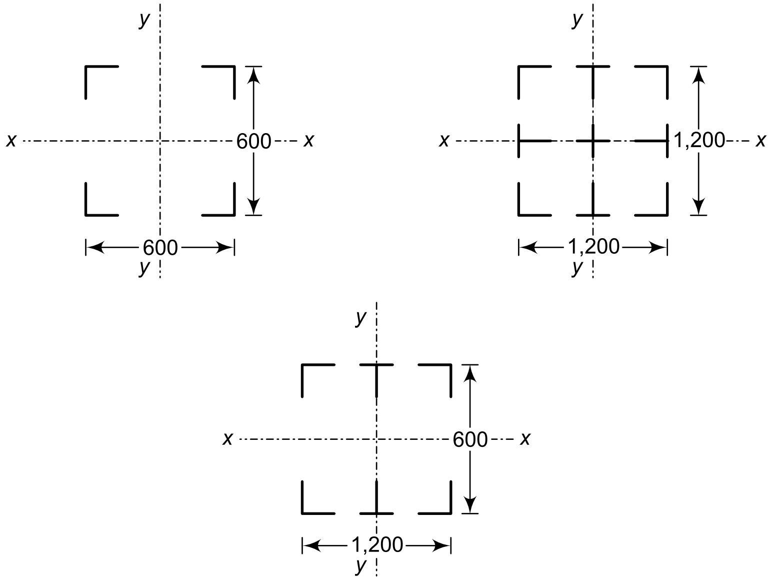


Figure 12.17 Different Arrangements of Angle Sections to Form Cribs.

One typical arrangement of trestles adopted at a project site is shown in Fig. 12.18.

Figure 12.18 Typical Arrangements of Trestles.

12.4 DESIGN OF VERTICAL SUPPORTS FOR FORMWORK

The design aspect of the vertical supports for formwork is discussed in this section. The readers have already been made familiar with the design of single legged shores in Chapter 7. The design of multi-legged shores (shoring towers) and trestles are briefly described here. The load calculation is similar to what has already been explained in earlier chapters (Chapter 7 in particular). Thus this aspect is not mentioned in the subsequent discussions.

The design of multi-legged shoring towers has received very little attention in the past. Traditionally, the design is left to the site functionary such as the project engineer, the superintendent, or the foreman, depending on the regularity of the element to be framed. Needless to say, this results mostly in poorly designed formwork, sometimes uneconomical, and can sometimes lead to physical failures also.

Similar to the design of the other formwork components, shoring tower design is concerned with finding the appropriate distances between the joists and stringers, and distances between towers the in two directions. Also the stability aspect of the tower needs careful consideration.

The distances between the joists are governed by the maximum allowable span of the sheathing elements. For safety reasons, the joist's spacing, both on and between the tower rows, is determined in such a way that the sheathing element is supported with no cantilevers on its two sides.

The distances between the tower rows are similarly governed by the maximum allowable span of the joists. The distances between the towers within the rows are dictated by the maximum allowable span of the stringers, but may be further constrained by the carrying capacity of the towers. For practical considerations, a tower placed next to a wall should be no closer than a minimum distance from the wall (around 200 mm, measured from the tower leg's axis) so as to leave room for the tower footings and to allow manipulation of the tower's screw jacks (tower spindle).

Shapira *et al*. (2001) reported the design and construction of the high multitier shoring towers which may be of interest to the practitioners. Although the area of the slab cast was small in the case project studied by them, the height of the shoring towers (about 60 m) required to be erected for the casting of the slab may be of interest to the readers, as there is limited literature available on the subject. The building in the case project was 25-story high.

Shapira *et al*. (2001) explored a number of solutions and preferred the option of erecting the multitier shoring tower for casting the slab for economic reasons. For this option also, they considered two alternative shoring towers: steel towers with a working load of 45 kN per tower leg and aluminum towers, with a working load capacity of 80 kN per tower leg, and the latter option was adopted for economic reasons. Some of the recommendations arising out of the study with regards to the design are given below:

- It is preferable to have a layout with a minimal number of towers under the given load and stability requirements;
- The option of getting uniform tower-leg loading should be adopted;
- The bracings against tower buckling, lateral forces, and wind effects are a must;
- The highest available frames must be used to minimize the number of tiers;
- The self-weight of the towers in such cases could be as high as 10% of the combined weight of the concrete plus the live load.

12.4.1 Design of Proprietary Shoring Towers

The manufacturers of the shoring towers provide literature on the safe load carrying capacity of a shoring tower for different heights. They also provide the connection details for joining different frames to assemble a shoring tower. The heights of the frames are usually available in such a combination that virtually any height of shoring tower is possible to attain in practice. The manufacturers also provide guidelines on bracing different independent shoring towers. The claim of the manufacturers regarding the load carrying capacity of the towers must be cross-checked with the test reports provided by them. It is important to stick to the guidelines provided by the manufacturer.

Now given the load carrying capacity of the towers, or the frames, or the each leg of the frame, one can easily decide the spacing of the towers in both the directions for a given design load. This is explained below.

Let's design the shoring towers for supporting the formwork for a RC roof slab of 1,200 mm thickness. The overall intensity of vertical load is the sum of the dead load, the live load, and the self weight of the formwork.

$$\begin{aligned}\text{Dead load} &= 26 \times 1.2 = 31.20\ \text{kN/m}^2\\ \text{Live load} &= 3.0\ \text{kN/m}^2\\ \text{Self weight of formwork} &= 0.8\ \text{kN/m}^2\ \text{(assume)}\\ \text{Thus total design load } w_d &= 35.0\ \text{kN/m}^2\end{aligned}$$

Let's arrange the shoring towers along the X-direction at 750 mm clear distance and along the Y-direction at 1,500 mm clear distance. The dimension of the shoring tower selected is 1,524 mm × 1,524 mm (L&T Heavy Duty Tower). The load carrying capacity of the L&T Heavy Duty Tower (plan dimension 1,524 mm × 1,524 mm) is 250 kN for a height of 6.0 m. Beyond this height also, the manufacturers claim that the capacity does not come down, provided the individual towers are braced to each other as per the manufacturers' guidelines.

In our case, let's assume that the required height of the shoring tower is 5.0 m. Now we simply need to check the actual load coming on one tower. For this we need to find out the area covered by one shoring tower.

The area covered by one shoring tower a_{xy} = distance covered in X-direction, d_x × distance covered in Y-direction, d_y

$$\begin{aligned}&= (0.750/2 + 1.524 + 0.750/2) \times (1.5/2 + 1.524 + 1.5/2)\\ &= 2.274\ \text{m} \times 3.024\ \text{m} = 6.876\ \text{m}^2\end{aligned}$$

Thus the maximum load exerted on one shoring tower = $w_d \times d_x \times d_y = 35 \times 2.274 \times 3.024 = 240.1$ kN which is less than 250 kN and is thus safe.

In case the load on one shoring tower would have exceeded the prescribed load carrying capacity of the shoring tower, one needs to appropriately reduce d_x, d_y, or both.

In Figs. 12.19-12.23, different possibilities of arrangement of shoring towers in the slab and beam formwork are shown. From the load intensity, the spacing of the shoring towers can be decided appropriately.

12.4.2 Design of Trestles/Cribs

The design of the cribs is explained in the following section. Let's assume that the maximum height of the cribs required is 4.096 m. The design is illustrated assuming that ISA 75 × 75 × 8 is available. The calculation of load on the trestle is not shown here as there is no difference in the load computation method and readers can follow the procedure similar to the one explained in Chapter 7 for slab and beam case.

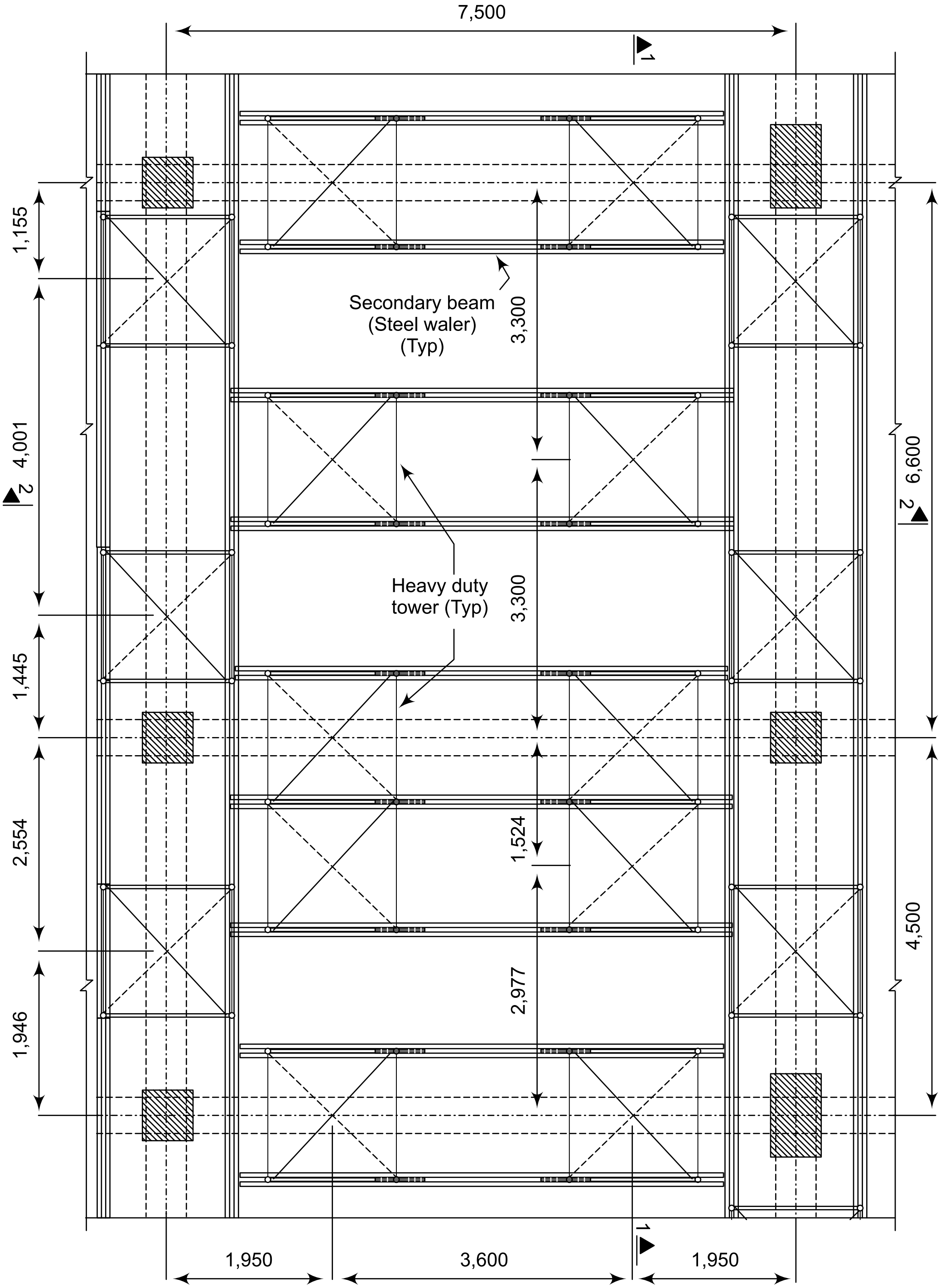


Figure 12.19 Arrangement of Shoring Towers in Plan for Slab and Beam Formwork.

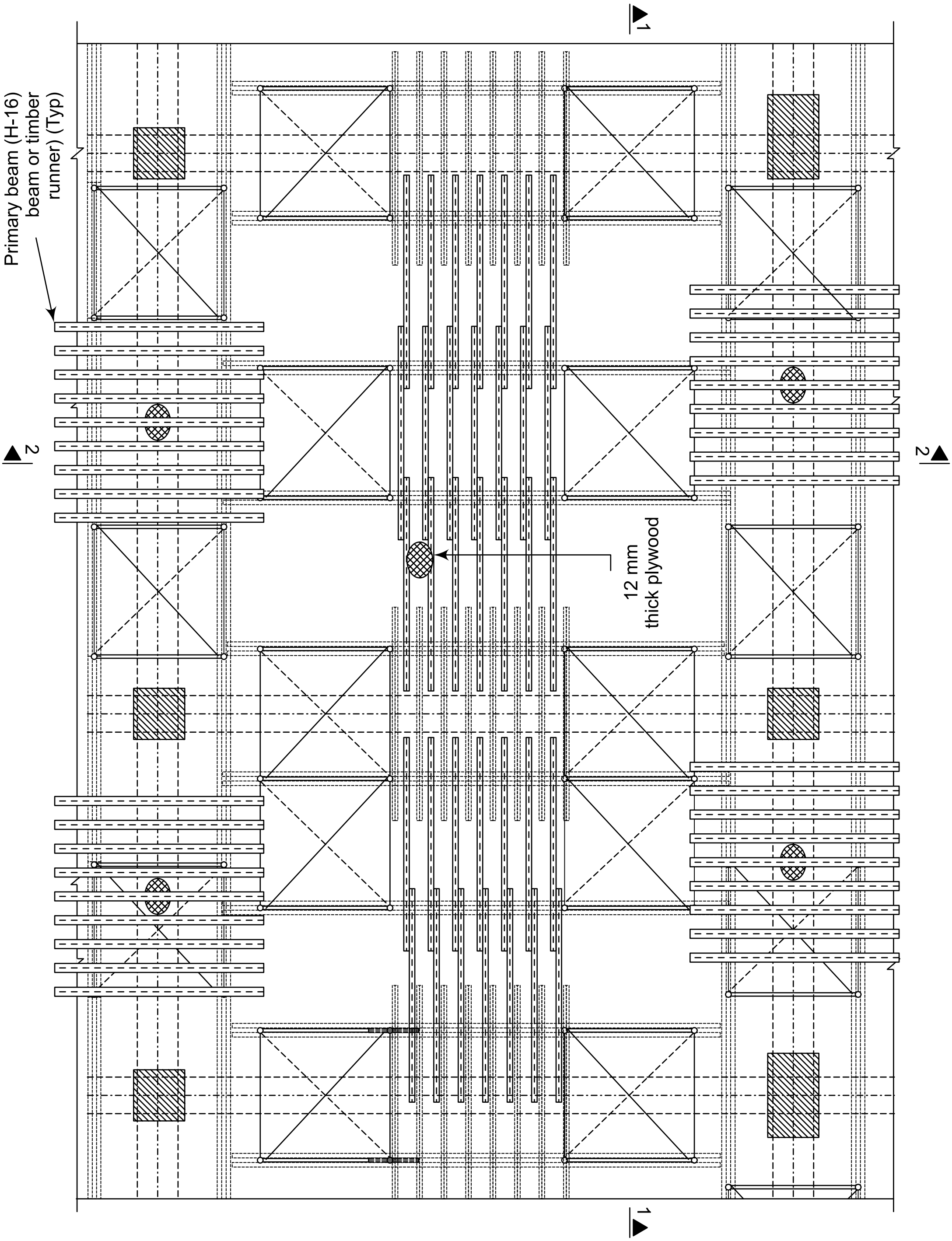


Figure 12.20 Plan of Formwork Arrangement Using Shoring Towers.

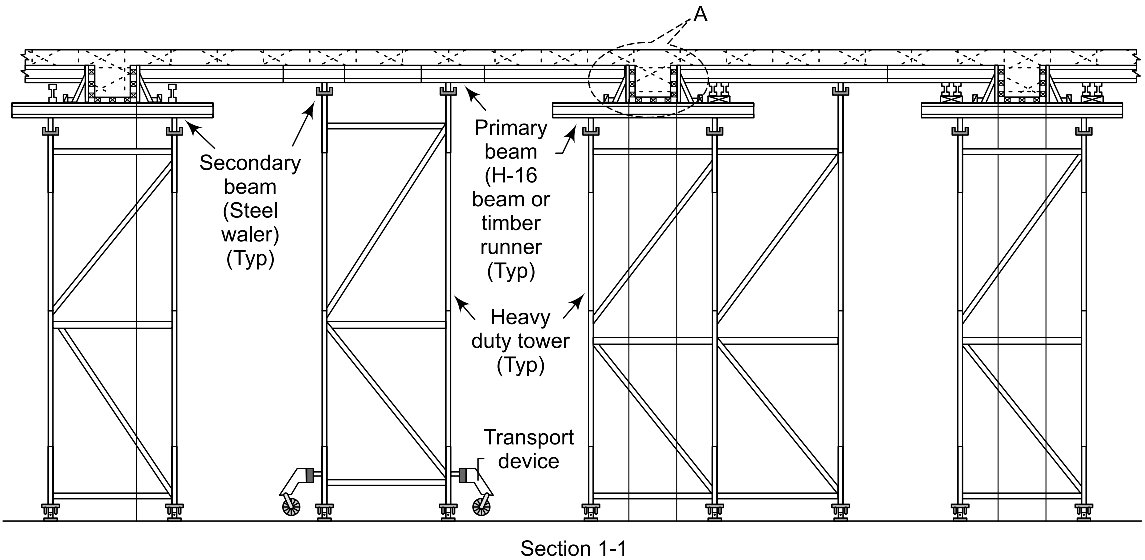


Figure 12.21 Sectional View of Shoring Towers for Slab and Beam Formwork.

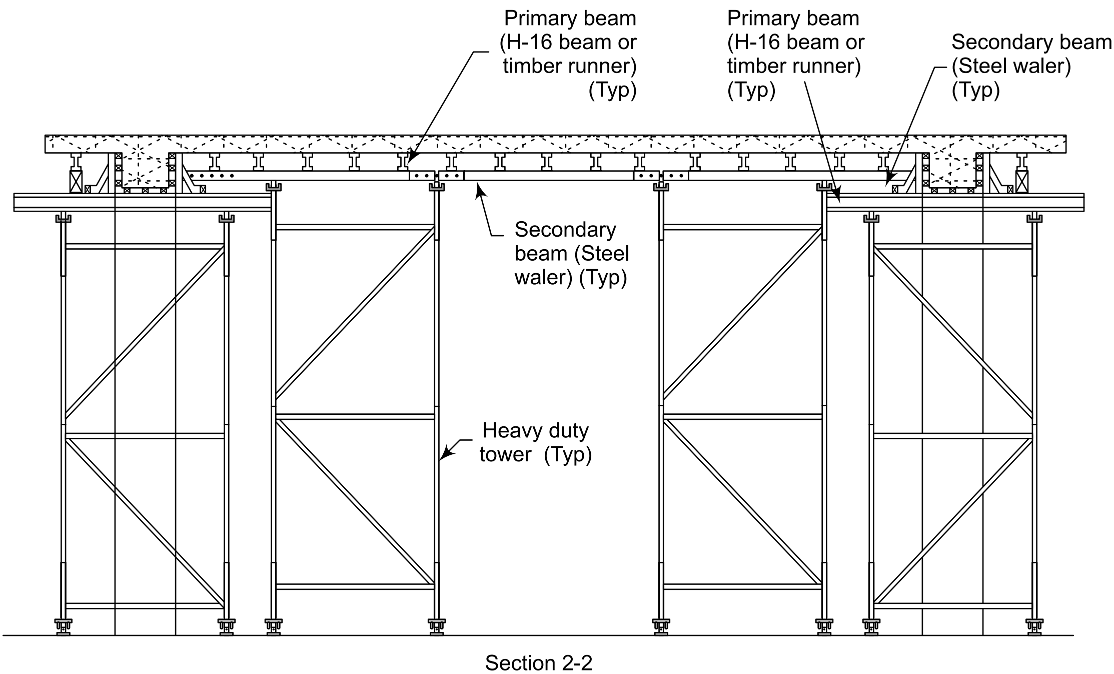


Figure 12.22 Sectional View of Formwork Arrangement Using Shoring Towers.

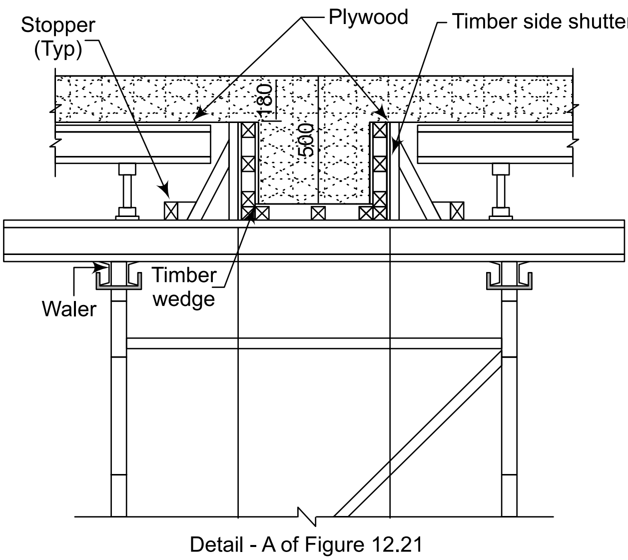


Figure 12.23 Details of Beam Formwork Supported on Shoring Tower.

Let us assume that the total load on one trestle including self weight of the trestle = 450 kN. The total moment in normal condition has been calculated to be 56 kNm and under seismic condition, is 95 kNm.

Maximum height of cribs = 4.096 m = 4,096 mm

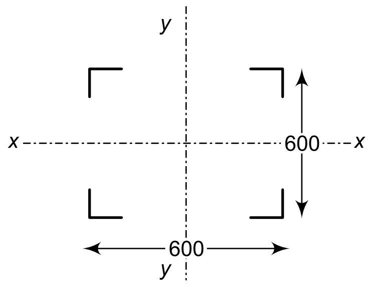


Figure 12.24 Trestle (Crib) in Plan.

Try, 4 ISA 75 × 75 × 8 (see Fig. 12.24)

For, ISA 75 × 75 × 8

$$I = 5{,}90{,}000 \text{ mm}^4$$
$$A = 1{,}138 \text{ mm}^2$$
$$C_x = C_y = 21.4 \text{ mm}$$

Area of whole assembly = 1,138 × 4 = 4,552 mm^2

Moment of inertia of Crib,

$$I_{xx} = 4 \times 5{,}90{,}000 + 4 \times 1{,}138 \times 278.6 \times 278.6$$
$$= 35{,}56{,}76{,}953.90 \text{ mm}^4$$
$$I_{yy} = 4 \times 5{,}90{,}000 + 4 \times 1{,}138 \times 278.6 \times 278.6$$
$$= 35{,}56{,}76{,}953.90 \text{ mm}^4$$

so,
$$r_{\min} = \sqrt{\frac{I_{yy}}{A}} = \sqrt{\frac{35{,}56{,}76{,}953.90}{4 \times 1{,}138}} = 279.5 \text{ mm}$$

$$l_{eff} = 4.10 \times 1.5 = 6.14 \text{ m} = 6{,}140 \text{ mm}$$

$$\frac{l_{eff}}{r_{\min}} = \frac{6{,}140}{279.5} = 21.97$$

$$Z_x = Z_y = \frac{35{,}56{,}76{,}953{,}90}{300} = 11{,}85{,}589.83 \text{ mm}^3$$

$$\sigma_{ac} = 147.41 \text{ N/mm}^2 \text{ (from Table 11.1 of IRC : 24 – 2001)}$$

Axial load capacity of crib $= \dfrac{4 \times 1{,}138 \times 147.41}{1{,}000}$

$= 671$ kN

Load carrying capacity = 671 kN > 450 kN (Safe)

Calculated axial compressive stress

$$\sigma_{ac,\text{ cal}} = \frac{450 \times 10^3}{4{,}552} = 98.86 \text{ N/mm}^2$$

Calculated bending compressive stress

$$\sigma_{bcx,\text{ cal}} = \frac{56 \times 10^6}{11{,}85{,}589.83} = 47.23 \text{ N/mm}^2 \text{ (Normal condition)}$$

$$\sigma_{bcx,\text{ cal}} = \frac{95 \times 10^6}{11{,}85{,}589.83} = 80.13 \text{ N/mm}^2 \text{ (Seismic condition)}$$

To make the structure safe

$$\frac{\sigma_{ac,\text{ cal}}}{\sigma_{ac}} + \frac{C_{mx} \cdot \sigma_{bcx,\text{ cal}}}{(1 - (\sigma_{ac,\text{ cal}}/0.60 f_{ocx})) \times \sigma_{bcx}} < 1.00$$

$$\sigma_{ac,\text{ cal}} = 98.86 \text{ N/mm}^2$$

$$\sigma_{ac} = 147.41 \text{ N/mm}^2$$

$$C_{mx} = 1.00$$

$$f_{ocx} = \frac{\pi^2 E}{\lambda_x^2} = \frac{\pi^2 \times 2.11 \times 10^5}{21.970^2} = 4314.42 \text{ N/mm}^2$$

$$\sigma_{bcx} = 0.62 f_y = 0.62 \times 250 = 155.0 \text{ N/mm}^2$$

Thus,

$$\frac{\sigma_{ac,\text{ cal}}}{\sigma_{ac}} + \frac{C_{mx} \cdot \sigma_{bcx,\text{ cal}}}{(1 - (\sigma_{ac,\text{ cal}}/0.60 f_{ocx}) \times \sigma_{bcx}}$$

$$= \frac{98.86}{147.41} + \frac{1.00 \times 47.23}{\left(1 - \dfrac{98.86}{0.60 \times 4{,}314.42}\right) \times 155} = 0.671 + 0.317 = 0.988 < 1.00 \text{ (Safe)}$$

In seismic condition, permissible stress shall be increased by 40%, hence above check in seismic condition has not been shown.

Design of Lacing:

Spacing of Lacing is governed by $\lambda_e < 50$ or

$$\lambda_e < 0.7 \times \lambda$$

which ever is less

$$\lambda_e = 0.7 \times 21.97$$
$$= 15.4$$

Hence, $\lambda_e = 15.4$

For, Angle $75 \times 75 \times 8$

$$r_{\min} = 22.8 \text{ mm}$$

So, $S \leq 15.4 \times 22.8 = 351.12$ mm

Adopt spacing 550 mm

$$b = 600 - 50 = 550 \text{ mm (25 mm end offset on both sides for lacing bar)}$$

So, $S = 2b \cot \varphi$

$$= 2 \times 550 \cot \varphi = 550$$

$$\Phi = 63.43° \; [40° < 63.43° < 70°] \; \textbf{Safe}$$

Transverse shear $= 0.025$ P

$= 0.025 \times 450 = 11.25$ kN

No. of lacing planes $= 2$ (single lacing)

Force on lacing bar $= \dfrac{11.25 \times \text{cosec } \varphi}{2} = 6.29$ kN

Length of lacing bar $= \sqrt{275^2 + 550^2} = 614.92$ mm

Using 25 mm φ plain bar

$$I = 19{,}174.80 \text{ mm}^4$$
$$r = 6.3 \text{ mm } (\varphi/4)$$

Effective length of welded bar $= 0.7 \times 614.92$ mm

$= 430.44$ mm

So, $\lambda = 430.44/6.3 = 68.32 < 140$ **Safe**

For, $\lambda = 68.32$

$$\sigma_{ac} = 113.10 \text{ N/mm}^2 \text{(from Table 11.1 of IRC : 24 – 2001)}$$

$$\text{Actual stress} = \frac{6.29 \times 1{,}000}{490.625} = 12.82 \text{ N/mm}^2 < 113.10 \text{ N/mm}^2 \; \textbf{Safe}$$

Use 25 mm plain bar as lacing @ 550 mm spacing

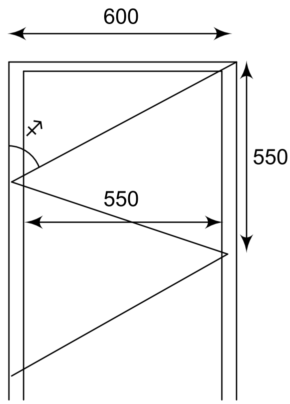


Figure 12.25 Lacing Details.

12.5 WORK INPUT FOR SHORING TOWERS

The towers account for a remarkably high percentage of the cost of the constructed concrete element, because the cost of such towers takes the overall forming cost higher than the common—quite high in themselves — estimates of 40-60% of the cost of the reinforced concrete. It is vital for the constructors to possess information primarily on the work-input data, as well as other related information on erection methods, crew size, and the like.

Shapira (1995) studied the work input for erecting and dismantling the shoring towers using direct/ stopwatch time study. Towers were assembled and dismantled using two-worker teams. Each tier in the square and rectangular towers had four frames and the triangular towers had three frames in each tier. The activities of erection and dismantling in the computation of time are shown in Table 12.2.

Table 12.2 Activities Considered in Erection and Dismantling for Work Input Analysis

Activities of erection	Activities of dismantling
• Carrying tower parts from the staging area to the assembly location up to 30 m away. • Oiling tower screws • Making tower locations on floor • Preparing wooden footings • Erecting and leveling tower base • Assembling tower maintaining plumb • Adjusting tower height • Installing temporary work platform on towers	• Oiling tower screws • Lowering the tower by closing adjustment screws • Dismantling the tower • Separating tower base plate from the wooden footings • Carrying tower parts back to the staging area

The time obtained by Shapira (1995) is shown in Table 12.3. The study was conducted on 5 and 8 m tower heights in fair weather conditions using experienced work crews. The towers studied were of two types: (1) Four-leg towers of either 1.40 m × 1.40 m square, or 2.00 m × 1.20 m rectangles, and (2) Three-leg towers using 1.40 m × 1.40 m triangles.

It can also be observed from Table 12.3 that the input times recorded for erection and dismantling are higher than those claimed by the manufacturers of shoring towers and hence they should be used with proper care.

Table 12.3 Work Inputs in Erecting and Dismantling Shoring Towers

Work Inputs (Worker-Hours)				
Activity	Four-leg Tower (Square/Rectangle)		Three-leg Tower (Triangle)	
	H = 5 m	H = 8 m	H = 5 m	H = 8 m
Erecting	2.29	3.20	1.83	2.56
Dismantling	0.40	0.64	0.32	0.51
Total	2.69	3.84	2.15	3.07

Note *H* = shoring tower height

Shapira and Goldfinger (2000) developed an empirical relationship for work input prediction for the assembly and dismantling of shoring towers. One typical relationship for the ALUMA towers for tower height up to 30 m is given below:

For assembly:

$$Y_1 = 0.0035 \times x^3 + 0.024 \times x^2 + 0.36 \times x + 0.30 \tag{12.1}$$

For dismantling:

$$Y_2 = 0.0028 \times x^3 + 0.016 \times x^2 + 0.15 \times x + 0.01 \tag{12.2}$$

where: x is number of tiers, Y_1 is the assembly work inputs in worker-hours, and Y_2 is the dismantling work input in worker-hours.

The above equations could also be useful for the multitier shoring towers beyond 30 m height and the actual input is expected to be within a reasonable error of about 10-20% from that of the predicted work inputs. It is also interesting to note that the dismantling work input is almost the same as that of the assembly work input for high towers. In other words, the dismantling work input is about 50% of the total time (assembly and dismantling). In case of small tower heights up to 20 m, the dismantling work input is between 20-40% of the total time for assembly and dismantling.

12.6 SHORING TOWERS REUSE AND ERECTION SEQUENCE

Although theoretically, the possible number of reuses of shoring towers can vary between 200-300, in practice it was found by Shapira (1995) that on an average, the shoring towers of height 5-10 m are reused every 2 months; thus about 6 uses per year. Considering a life of 15 years for the shoring materials, the number of reuses can practically be assumed to be about 90.

A shoring tower is not a single formwork element, but almost always part of an array composed of several identical/similar towers erected next to each other. The erection sequence of the entire array may affect the work inputs. The particular erection sequence for a given job would normally be the choice of the erection crew, and would be determined by factors such as tower configuration, erection method— for example with/without crane assistance, and workers' habits. In principle, there are three different possible sequences by which to erect a tower array: (1) vertical: each tower is assembled to its full height before work progresses to the next tower; (2) horizontal: the entire array is erected tier after tier; and (3) a combination of the above two methods and is the most common and favored method.

12.7 RECOMMENDATIONS

Some of the recommendations on the design and construction of shoring towers are given below. These are based on the study conducted by Shapira (2004) on extremely high towers: 17, 26, and 60 m high. The study was based on the application of ALUMA frames, KABIR frames, and PERI frames.

1. Given a choice of shoring towers, the system offering larger frame height and a variety of tiers should be selected. This is explained with the help of schematic diagrams in Fig. 12.26. System A offers frames in 1,200 mm, and 1,800 mm heights while System B offers frames in 1,500 mm, 2,500 mm, and 3,000 mm heights. The choice would be System B.
2. From among the various frame heights offered by a given system for a shoring tower, the largest frames to minimize the number of tiers for a given height should be selected. This would obviously result in reduced work input.

   Thus, Fig. 12.27 (b) would be the appropriate choice.
3. The larger (heavier) frames should be used at the bottom of the tower and smaller frames should be used at the top of the tower. This is shown schematically in Fig.12.28. The tower configuration shown in Fig. 12.28 (b) is the appropriate method and should therefore be adopted.

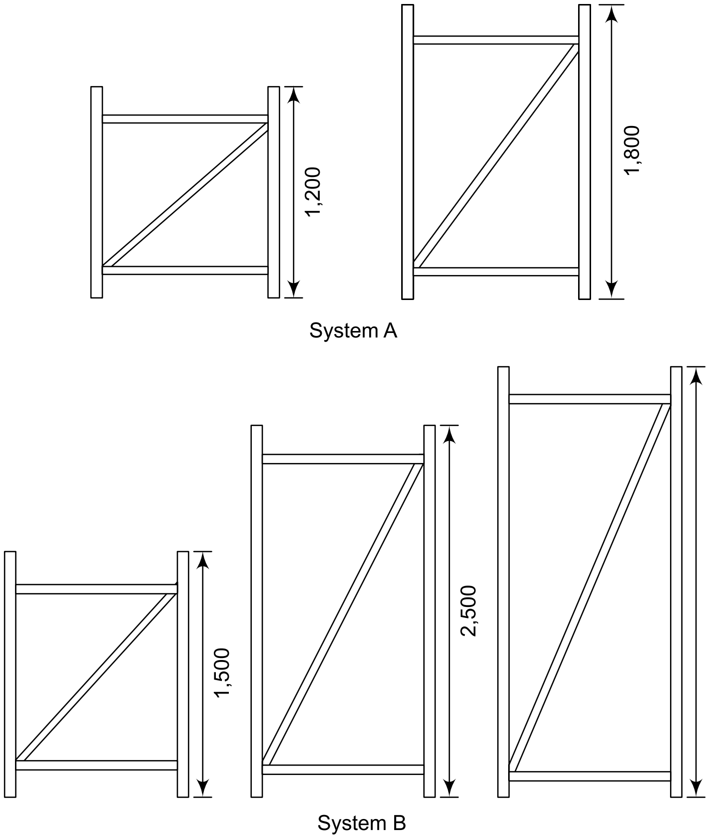


Figure 12.26 Two Systems –One with Smaller Frame Height and Less Variety and other with Larger Frame Height and More Varieties of Frames.

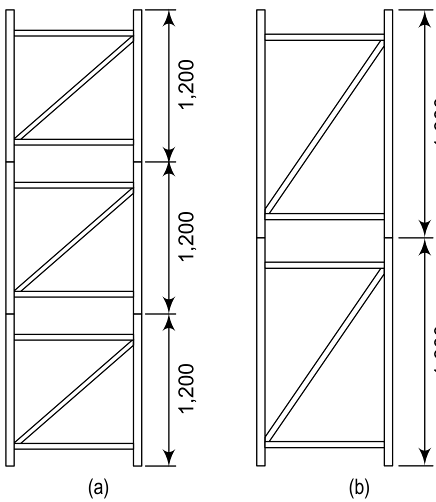


Figure 12.27 Two Arrangements of a Tower of Same Height.

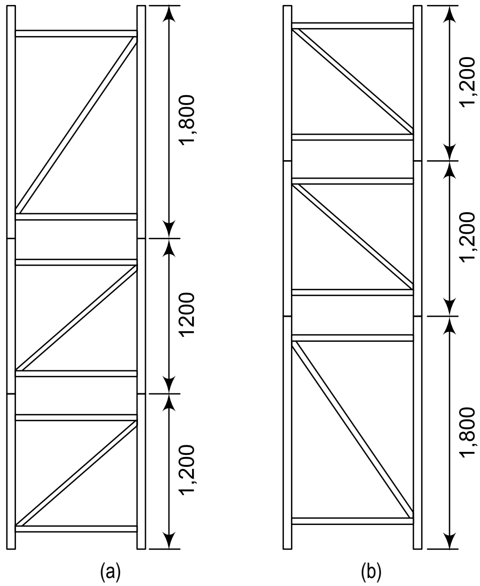


Figure 12.28 Two Arrangements of a Tower with Different Frame Arrangements.

4. The tower spacing should be maximized by choosing appropriate stringers and joists (thicker and heavier sections capable of spanning larger distances). By doing so, the load carrying capacities of the towers are not compromised excessively. The lighter sections of stringers and joists would necessitate large number of towers without offering any additional advantages. In fact it would make the shoring system costlier. The top figure in Fig. 12.29 shows closer tower spacing due to the limited load carrying capacity of the stringer while the bottom figure shows increased tower spacing on account of stronger stringer. Thus, preferably the bottom arrangement should be preferred.

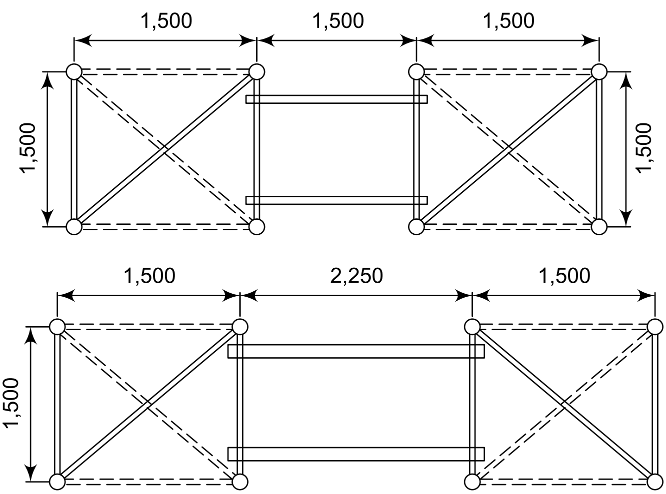


Figure 12.29 Two Arrangements of Towers in Plan.

5. It is desirable to use the shoring system with which the work crew is familiar. The number of workers in a crew should also be carefully selected. Shapira (2004) suggests two workers per crew for towers up to three tiers, three workers per crew for towers up to six tiers and four workers per crew for towers higher than six tiers.
6. For enhanced productivity, the shoring tower material should be stacked as near as possible to the tower assembly location. Depending on the system characteristics, a system of either horizontal pre-assembly on the ground or vertical *in-situ* assembly should be chosen. The space constraints in the pre-assembly and lifting area should also be considered in the comparison.
7. Bracing the towers should not be done based on intuition, rather it should be based on proper design. Failure to do so may either result in excessive number of bracings thereby increasing the cost, or it could result in the failure of the shoring system itself.
8. For shoring linear concrete elements such as beams, three-leg towers prove to be more efficient productivity-wise compared to four-leg towers. According to Selinger and Shapira (1985), the work inputs for three-leg towers are roughly 80% of that required for four-leg towers of the same type.

12.8 HORIZONTAL SUPPORTS

Similar to the vertical formwork supports or shores, there are many types of horizontal formwork supports also available. These are usually light weight latticed or boxed beams (Figs. 12.30 and 12.31) which also telescope one into the other (Fig. 12.32) and cater for a range of spans. These horizontal supports being capable of spanning across beams or wide spans, rest either on beam forms or other shores at the ends. They eliminate the need for intermediate supports thereby providing free access and working space during construction.

Figure 12.30 Horizontal Support with Latticed Structure.

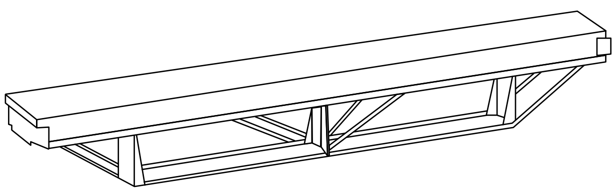


Figure 12.31 Another type of Horizontal Support.

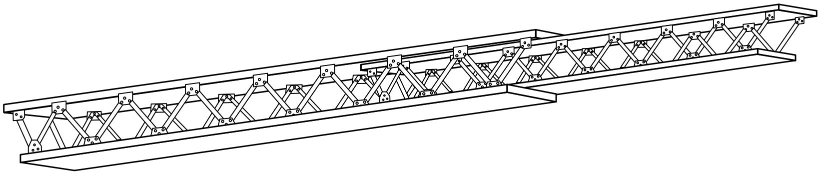


Figure 12.32 Horizontal Support with Telescopic Span.

There are many types of horizontal formwork supports available. These are given in Table 12.4.

Table 12.4 Details of Various Types of Formwork Supports

Name	Description	Length combination (mm)
SO/ESI	Short outer/Extr short inner	2,390 – 2,530
SO/SI	Short outer/short inner	2,470 – 4,160
SO/LI	Short outer/long inner	3,100 – 4,800

REVIEW QUESTIONS

Q1. True or False

(a) Formwork supports are of two types– horizontal and vertical.

(b) Vertical supports are of two types– single and multi-legged type.

(c) Single leg type is also known as shore or prop.

(d) Multi-leg type supports are frame or tripod or trestle.

(e) *l*/*d* ratio is an important design parameter for timber shores and thus while using timber shores, sufficient thickness of timber should be ensured to safeguard them against buckling.

(f) Drop heads are devices which are fitted on top of the props or supports to support the slab while the remaining form for decking could be struck for reuse.

(g) Multi-legged shoring tower—frame based system, tube and coupler system, and trestle system.

(h) Tower configuration, frame configuration, dimensional variation of frames, load carrying capacity, and assembly method are the major characteristics upon which the shoring tower design differs from each other.

(i) In the context of different possible sequences by which to erect a vertical tower array: each tower is assembled to its full height before work progresses to the next tower.

(j) In the context of different possible sequences by which to erect a horizontal tower array : entire array is erected tier after tier.

(k) Horizontal supports are capable of spanning across beams or wide spans rest either on beam forms or other shores at ends.

(l) Horizontal support eliminates the need for the intermediate supports thereby providing free access and working space during construction.

Q2. Match the following in the context of classification of the shoring towers

(i) Hurd (1963)	(a) Type A, B, C, D
(ii) Shapira and Raj (2005)	(b) Standard, heavy duty, and extra heavy duty

Q3. Prepare short notes on

(a) Type A, B, C, and D shoring towers. (b) Trestle (crib) shoring.

Q4. For the design of the vertical support for formwork, list out the various critical recommendations.

Q5. Perform a design analysis for proprietary shoring tower.

Q6. Prepare short notes on work input analysis for shoring tower.

Q7. List out the various activities involved while erecting and dismantling for work input analysis.

Q8. List out the recommendations on the design and construction of shoring towers in the context of the application of ALUMA frames, KABIR frames, and PERI frames.

Q9. List out the various types of horizontal supports and their unique characteristics.

Chapter

13 Scaffold

Contents: Introduction; Classification of Scaffolds; Timber Scaffolds; Metal Scaffolds; Types of Metal Scaffolds; Some Proprietary Scaffolds; Galvanized Scaffolds; Scaffold Boards; Scaffolds for High Clearance Structures; Design Issues; Possible Causes for Collapse of Scaffold Systems; Checklist

13.1 INTRODUCTION

Scaffolding is a temporary structure for gaining access to the higher levels of the permanent structure during construction. Scaffolds are often used because they are convenient, versatile, and economical. They are practically needed in all the stages of construction. Scaffolds are one of the essential formwork parts to provide temporary platforms at various levels for carrying out all those works which cannot be conveniently and easily carried out either from the ground level or any other floor of the building or by the use of a ladder. Besides providing access, the scaffoldings are also used for (i) centering for the formwork, and (ii) for supporting heavy loads at great heights.

13.2 CLASSIFICATION OF SCAFFOLDS

Scaffolds can be primarily divided into two broad heads, based on normal usage, as shown in Fig. 13.1. Discussion on these scaffolds is provided in the next section.

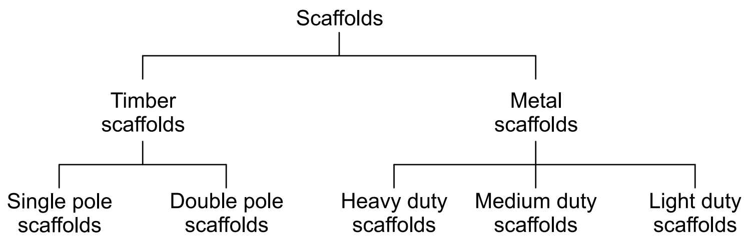


Figure 13.1 Classification of Scaffolds Based on Normal Usage.

Sometimes wooden shores are used in combination with modular steel scaffolds to provide temporary supports.

Depending on the type of construction, scaffolds can be primarily divided into the following broad heads (Fig. 13.2). These are discussed subsequently.

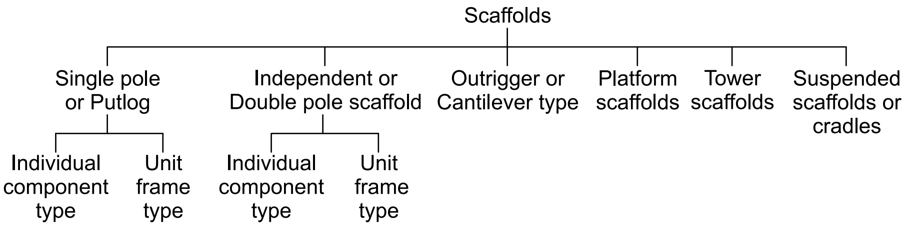


Figure 13.2 Classification of Scaffolds Based on Type of Construction.

13.3 TIMBER SCAFFOLDS

Timber has been used for building scaffolds from time immemorial and continues to be used even at present. The most common type of scaffold used in India even today is the *bally* or the bamboo scaffold. Not only are such scaffolds used for normal residential buildings, they are also used for tall multi-story building and for massive works like dams and barrages of considerable heights, where stronger and safer scaffoldings are called for. The bamboo or *bally* scaffoldings on some of the tall buildings present a dreadful sight because either they get bent or buckled in all directions or become crooked. Barring few cases where bamboo or *bally* scaffolds are neatly erected (Fig. 13.3), properly braced and well tied to the buildings, invariably, such scaffolds are in awkward and crooked shapes and present a dreadful sight particularly in tall buildings where they are definitely not safe and the workers on them are in a constant fear for safety. By and large, the bamboo or *bally* scaffolds are constructed by tying them with hemp rope. Due to long exposure to weather, not only the bamboo or *bally* gets distorted, but the hemp or coir ropes also get deteriorated, and the joints no longer remain firm. The scaffold goes out of plumb. Such scaffolds are built with scant regard to safety requirements, particularly relating to working platforms, guard rails, toe boards, etc. There is an urgent need to regulate the use of bamboo or *bally* scaffolds for tall buildings and ensure a proper code of practice in constructing them.

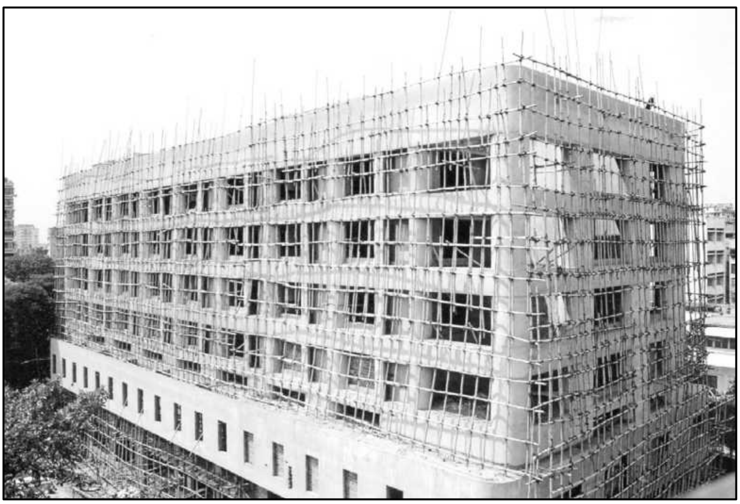


Figure 13.3 Bamboo Scaffold in Position.

13.3.1 Types of Timber Scaffolds

Timber or bamboo scaffolds are either of single or double pole type. In case of single pole type scaffolds, the single row of the upright poles is fixed close to the building or wall and connected horizontally by ledgers along with the length connected to the buildings or walls by means of cross timbers known as 'Putlogs'.

In case of double pole scaffolds which are also called 'Independent scaffolds', there are two rows of uprights kept about 1 to 1.5 m apart across the building and at suitable intervals along the length. The two rows of uprights are connected by cross timbers viz. putlogs or transoms and longitudinally by ledgers. Suitable diagonal bracings are also provided for the same. In case of bamboo or *bally* scaffolds built with sawn ropes or coir of hemp, and in case of scaffolds built with sawn timber sections, the joints are usually made with bolted/ nailed connections.

13.4 METAL SCAFFOLDS

The increase in building construction activity, particularly of tall structures, necessitates the use of safer and more dependable scaffolding. In order to cater to these needs, metal scaffolding of various types have been developed.

13.4.1 Classification Based on Usage

In general, metal scaffolding is classified into three types depending upon the usage. The three types are shown in Table 13.1. In the table, usage/application of each type is also mentioned. Heavy duty scaffolding is used for brick laying and concreting operations; medium duty scaffolding is used for plastering and electrical works, etc.; light duty scaffolding is used for lighter load applications such as painting, window cleaning and so on. The classification shown in Table 13.1 is only a general indication on the basis of normal usage.

Table 13.1 Metal Scaffold Classification Depending on the Usage

Sl. No.	Type of scaffolding	Usage/Application
1.	Heavy Duty Scaffolding	This is used by brick layers and masons and in concreting operations and for facade cladding work and demolition work.
2.	Medium Duty Scaffolding	This is used by carpenters, plasters, glaziers, sign board erectors, clearers, welders and electricians, etc.
3.	Light Duty Scaffolding	This is used by electricians, painters, window cleaners, etc.

13.4.2 Classification Based on Type of Construction

Generally, depending upon the type of construction, scaffolds are classified into the following types:

13.4.2.1 Putlog scaffolds or single pole scaffolds

Putlog scaffolds or Single pole scaffolds consist of single row of uprights connected together by ledgers. Putlogs are fixed to the ledgers and built into the wall of the building as the construction progresses. The scaffold system essentially consists of a base plate, ledger, uprights, double coupler for coupling the ledger to the uprights, putlog, putlog coupler, horizontal tie member, longitudinal diagonal brace, toe board, and guard rail. The positions of these components are shown schematically in Fig. 13.4.

Figure 13.5 illustrates the Putlog Scaffold (Unit Frame Type). The system consists of base plate, unit vertical, unit Putlog cross bar, unit type longitudinal diagonal brace, horizontal tie coupled with double couplers, scaffold boards, toe board, and guard rail.

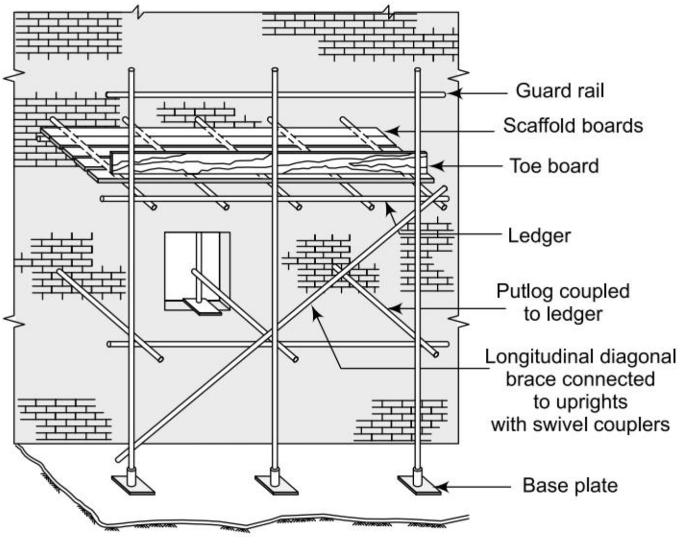


Figure 13.4 Putlog Scaffold (Individual Component Type).

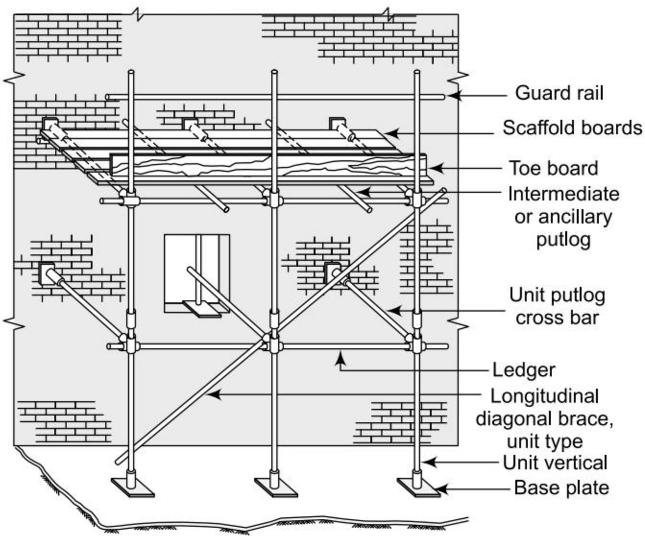


Figure 13.5 Putlog Scaffold (Unit Frame Type).

13.4.2.2 Independent scaffolds or double pole scaffolds

This consists of two rows of uprights connected together longitudinally by ledgers and transversely by putlogs or transoms. Figure 13.6 illustrates a typical independent scaffold (individual components type). The system consists of base plate, ledger, uprights, double coupler for coupling the ledger to the uprights, transom, putlog, putlog couplers to couple the transom to the putlog, longitudinal diagonal and cross braces, swivel coupler, toe board, and guard rail.

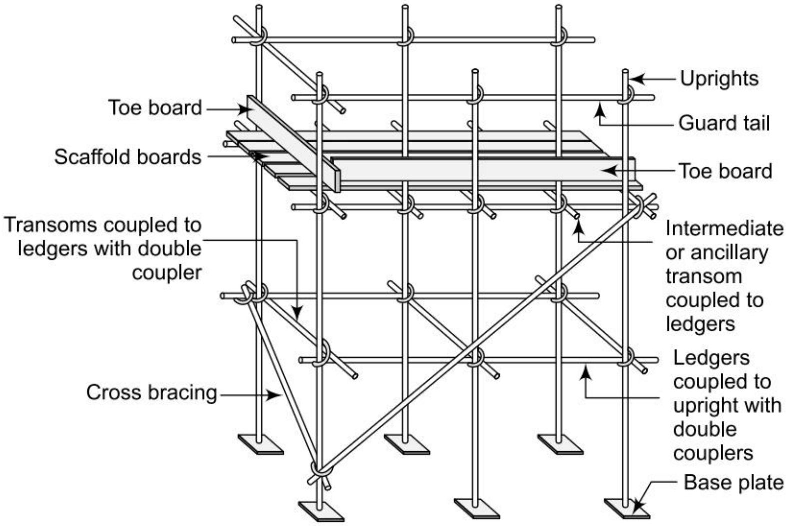


Figure 13.6 Independent Scaffold (Individual Component Type).

Figure 13.7 shows a typical Independent scaffold (Unit Frame Type) essentially consisting of the base plate, unit vertical, unit cross bar, ledgers secured to the unit cross bar, unit type longitudinal diagonal brace, scaffold boards, toe board, and guard rail.

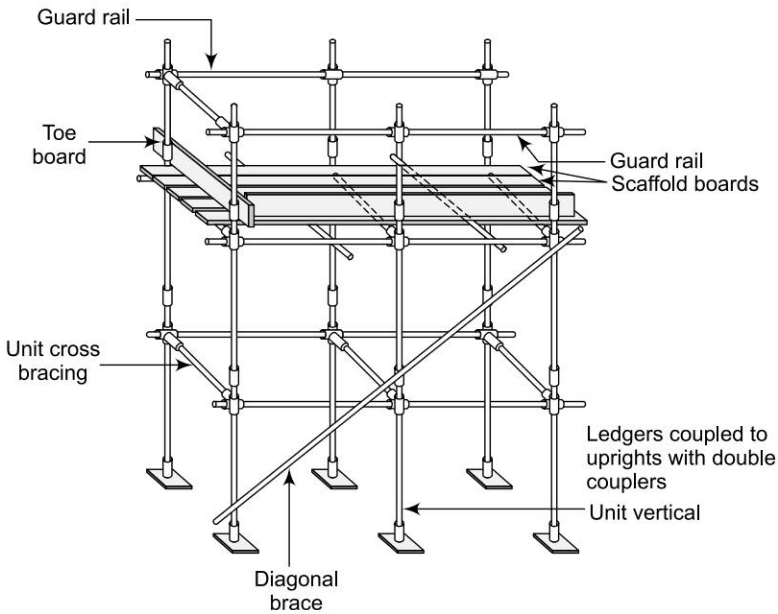


Figure 13.7 Independent Scaffold (Unit Frame Type).

13.4.2.3 Outrigger (Cantilever) scaffolding

This is usually an independent type of scaffolding which does not rest on the ground but is cantilevered from the face of the buildings or structures. Typical outrigger scaffolds are shown in Figs. 13.8 and 13.9.

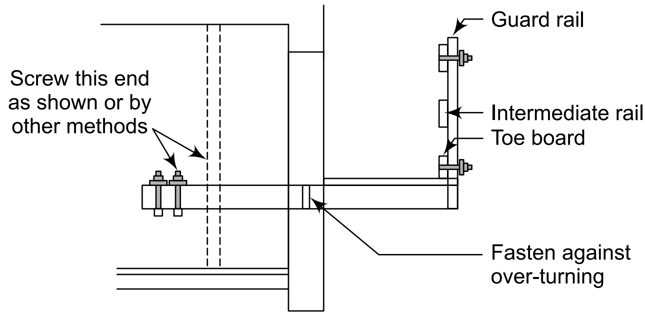


Figure 13.8 Outrigger (Cantilever) Scaffold.

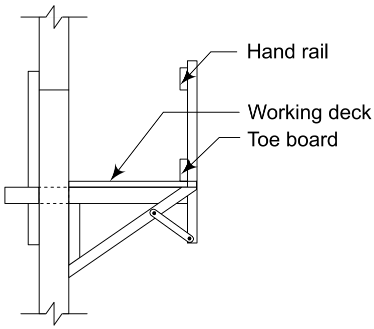


Figure 13.9 Outrigger (Cantilever) Scaffold.

13.4.2.4 Platform scaffolds

This consists of two or more rows of uprights connected together by ledgers and transoms and usually a working platform is placed on top of the scaffold. This type is used normally for supporting heavy loads at the top level and for providing an access platform at one level.

13.4.2.5 Tower scaffolding

This consists of uprights connected together by ledgers and transoms. This may be made mobile by mounting it on the castors.

13.4.2.6 Suspended scaffolds or cradles

This is an independent scaffold which is hung from a building or structure and not supported on the ground. Figure 13.10 illustrates the application of suspended scaffolds or cradles in a building construction project.

Each of the above type of scaffolds may be constructed for heavy, medium or light duty. By and large, metal scaffolds are made of steel tubes. Many countries have formulated standard specifications and codes of practice for the metal scaffolds. These are shown in Box 13.1.

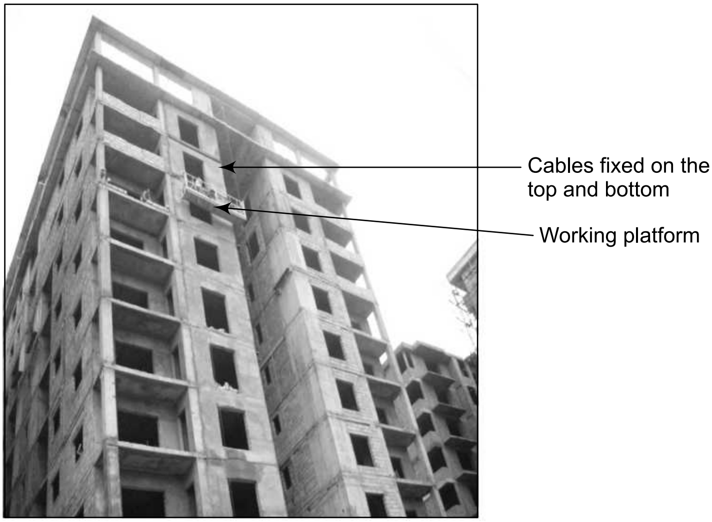


Figure 13.10 Suspended Scaffolds or Cradles.

BOX 13.1: Codes of practice for metal scaffolds

(i) Metal Scaffolding— British Standard 1139.
(ii) British Standard Code of Practice for Metal Scaffolding, Parts 1, 2 and 3.
(iii) Australian Standard 1576–1974. Metal Scaffolding Code.
(iv) Australian Standard 1575–1974. Tubes, Couplers and Accessories used in Metal Scaffolding.
(v) Indian Standard Specifications for Steel Scaffolding— IS: 2750–1964.
(vi) Indian Standard Code of Practice for Steel Tubular Scaffolding— IS: 4014 (Part 1)–1967 and IS: 4014 (Part 2)–1967.

13.5 TYPES OF METAL SCAFFOLDS

The designs of metal scaffolds that are currently in vogue are broadly of two types:

- Tube and fitting type, and
- Pre-fabricated unit- frame type.

13.5.1 Tube and Fitting Type

Most of the standard specifications have detailed the requirements of the tube and fitting type of scaffolds. The tube and fitting type generally consists of plain tubes which are used for making uprights, ledgers, transoms, and putlogs. Various types of clamps, viz. double or right angle coupler, swivel coupler, putlog coupler, joint pins, or end-to-end couplers are made of steel pressings or castings or forgings. In addition to the clamps, various accessories such as base plates/head plates— both fixed and adjustable type, platform brackets, modular cross bracings, are also available. Platform brackets are attached to the scaffold uprights to increase the width of the working platforms and also to give additional working lifts between the main platforms.

Many standard codes of practice specify the construction details of scaffolding with regard to the provision of working platforms, their minimum width, provision of guard rails, toe boards, etc. The codes of practice also give broad guidelines for bracing and tying of scaffolds in addition to following the safety provisions in building the scaffolds (Box 13.2).

BOX 13.2: Safety Provisions in Building the Scaffolds

(i) Every scaffold should be braced by means of longitudinal and transverse bracing systems so as to form a rigid and stable structure. Also every scaffold should be effectively tied to a building to prevent the movement of the scaffold either away or towards the building.
(ii) Where heavy winds or gale forces are expected, it is necessary to take special precautions and install additional ties to the scaffold to prevent overturning and collapse.
(iii) Guide rails and toe boards must be provided for all the working platforms to ensure the safety of the workmen.
(iv) All working platforms should be fully covered to prevent materials falling and causing injury to the workers or passersby.
(v) Safety nets or other screens should be provided to catch any falling materials.
(vi) The use of barrels, boxes, loose earth pads or other unsuitable objects as supports for uprights and working platform, should not be permitted.
(vii) Care should be taken to see that no uninsulated wire exists within 3 m of the working platforms, gang ways, runs, etc. of the scaffolds.
(viii) Scaffolds on thoroughfares should be provided with warning light, if general light is not sufficient, to make it clearly visible.
(ix) Men should not be allowed on scaffolds during storms or high winds.
(x) Grease, mud, paint, gravel or plaster or any such material should be removed from the scaffold platforms immediately.
(xi) Either sand or saw dust or any other suitable material should be spread on the platforms to prevent slipping.
(xii) All projecting nails from the platforms or other members should be removed.
(xiii) During dismantling of scaffolding, necessary precautions should be taken to prevent injury to the persons due to the falling of loose materials. The bracing and other members of the scaffolds should not be removed prematurely while dismantling the entire scaffold so as to avoid the danger of collapse.
(xiv) When scaffolds are to be used to a great extent and for a long period of time, they should be inspected from time to time to ensure their soundness.
(xv) Boards and planks used for platforms, gangways should be of sound quality and proper thickness, closely laid and securely fastened and placed.

13.5.2 Prefabricated Unit-Frame Type Scaffolds

Many designs of prefabricated unit-type scaffolds have been developed by proprietary concerns and are now being extensively used in most of the construction sites throughout the world for various scaffolding/centering/supporting needs. Such prefabricated units have been designed incorporating the following basic features:

- Prefabrication of adjustable components with few or no loose parts.
- Simple and fool-proof devices as far as practical within the system to prevent accidental displacement to ensure maximum safety.

- Speed and ease in erection at the site by unskilled labor.
- Known characteristics of each component enabling complete calculation of the loadings to ensure the use of minimum materials.
- High degree of versatility and durability enabling hundreds of uses for a wide range of application.

Some of the prefabricated types of scaffolding available are as follows:

13.5.2.1 Unit-frame or three piece frames

It consists of two vertical and one horizontal member with specially designed end fittings and when the three are assembled together, it forms an 'H' frame. The end fitting on the horizontal member also incorporates a fixing device for the longitudinal ledger. The unit frames can be erected one above the other and are spaced at suitable intervals depending upon the duty of the scaffolding.

These frames are generally made of tubes and are available in different categories, viz. light or heavy duty. The manufacturers provide complete data on the loading capacity, sequence of erection and safety precautions. They also supply other accessories, such as base plates, scaffold fittings and platform brackets with the system.

The advantage of this type of three piece frame is that the units can be spaced at any required interval and also the platforms can be had at any required levels and hence the scaffold of this type may be truly called as All Purpose type. As the three pieces of the system can be separated, transportation and storage cost are minimal.

13.5.2.2 Welded frame type

These scaffolds are made as welded units consisting of two uprights and one or more cross members to form a rectangular or 'H' frame. Such frames can be erected one over the other upto the required height. Length–wise such frames are connected either by scissors type cross bracers or ledgers.

In this system, the longitudinal spacing of the frame is decided by the length of the ledger or the cross bracings. Accessories such as base plates and scaffold fittings are also supplied to complete the system. The tying and bracing of such a system is usually done with tubes and couplers. The welded frame type scaffolds are also made of tubes in different grades, viz. light duty or heavy duty, as required.

13.5.2.3 Wedge lock type

The wedge lock type of the scaffold consists of the basic components, namely verticals, ledgers, transoms, and diagonals. The uprights have locking devices welded on them at regular intervals. The transoms, ledgers and diagonals have specially designed wedge lock assemblies fitted at the ends which engage in the locking devices on the uprights.

This type of scaffolding can be erected very fast and does not require any special tool except a small hammer to drive the wedges in. Along with the basic components, viz. uprights, ledgers, transoms and diagonals, other accessories such as the base plates, walk way brackets, scaffolds boards, are also supplied by the manufacturer.

The wedge lock type of scaffolding is available in standard and heavy duty range. The standard range is extensively used for formwork support incorporating the drop-head, and also as access scaffolds in ship building and maintenance yards. The heavy duty range is extensively used for building scaffold towers inside the chimneys, silos, etc. for use with suspended formwork.

All the above types of scaffolds apart from being used as access scaffolds, are also used for the supporting formwork or other heavy loads at great heights where normal propping is not feasible. These scaffolds are also used for building guyed aerial towers for telecommunications. Recent developments in steel scaffold equipment are the use of high tensile tubes to make them light and stronger.

13.5.3 Door Type Tubular Steel Scaffolds

The most commonly used metal scaffold is the door type tubular steel scaffold (Fig. 13.11). These scaffolds can be broadly divided into two types: (1) finishing scaffolds and (2) support or shoring scaffolds. The finishing scaffolds, as the name suggests, are used for executing finishing items while the support or shoring scaffolds are used for providing supports to the formwork. Door type tubular steel scaffold units come in standard shapes. Some of them are shown in Fig. 13.12.

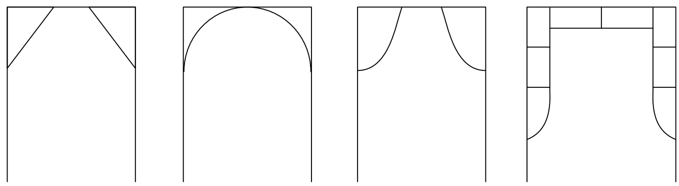


Figure 13.11 Different Configurations of Door-type Tubular Scaffolds.

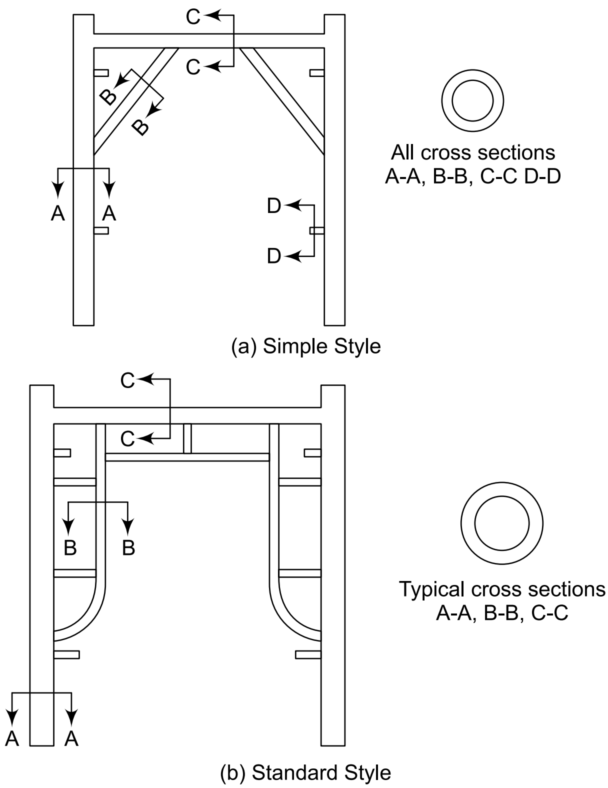


Figure 13.12 Door-type Steel Scaffold: (a) Simple Style; (b) Standard Style.

13.6 SOME PROPRIETARY SCAFFOLDS

13.6.1 L&T Access Scaffold System

L&T Access Scaffold System, also referred to as L&T Light Duty Scaffold, consists of 1.8 m high welded frames braced with horizontal bracings. The bracings also serve as hand rails. The scaffold can be safely erected up to a height of 40 m. Couplers and pins are used to connect two frames which are kept one over the other to attain the desired height. Minor adjustments in heights can also be achieved by tower spindles which are provided in the bottommost frames. The arrangement of the L&T Access Scaffold System is illustrated in Fig. 13.13.

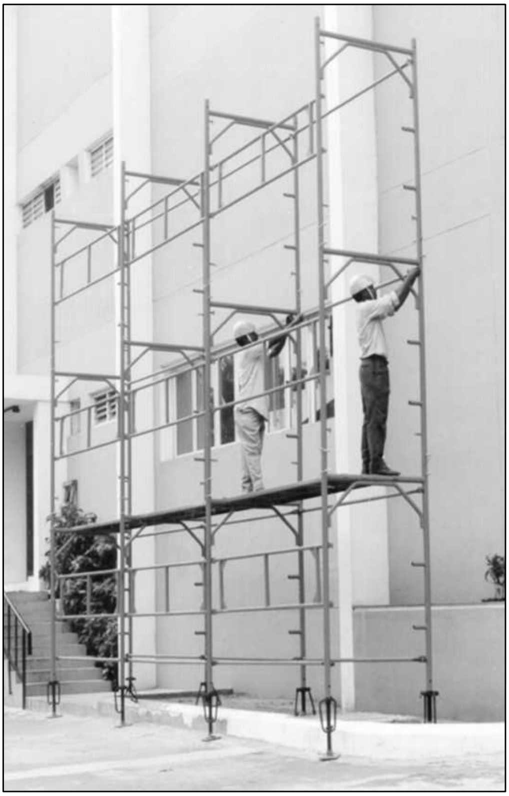


Figure 13.13 Scaffolds Primarily for Access (Courtesy L&T Formwork).

The towers are provided with tower spindles at the bottom for fine adjustments in height and leveling. The spindles are also used to adjust the levels of the scaffolds. The manufacturer recommends the bracing of the scaffold with the permanent structure every 4 m in height. The scaffold has provisions of fixing the working platform (scaffold board) at various levels depending on the requirement. The system being light weight, high labor productivity of the order of 0.15 manhours per m^3 is possible. Figure 13.14 shows the L&T Access Scaffold in use for an under construction building.

Figure 13.14 L&T Access Scaffold in Use for an Under Construction Building.

13.6.2 PERI Scaffold

PERI offers a number of solutions for scaffolding and working platforms to suit various applications. PERI UP scaffolds (Fig. 13.15) can be used for reinforcement work, closing the formwork, setting the form ties, concreting works, etc. The scaffold units are suitable for all job site applications. The completed scaffold units can be shifted from one location to another with the help of a crane (Fig. 13.16). The completed scaffold units can also be transported on a castor. The working platforms can be installed at different levels depending on the requirement.

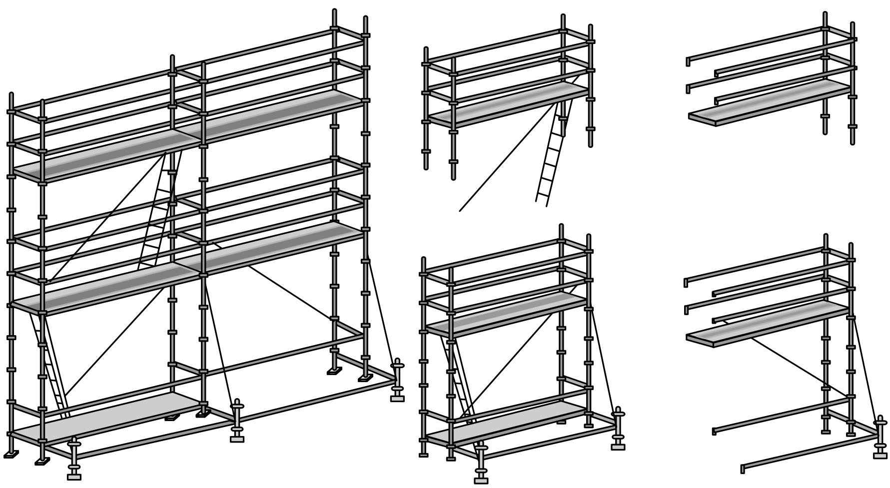


Figure 13.15 PERI UP Scaffold (Courtesy PERI).

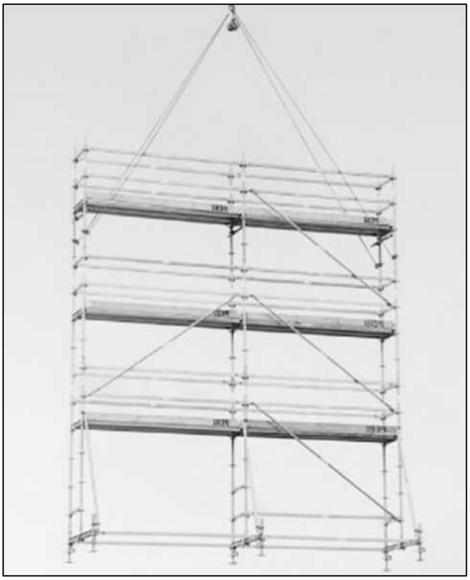


Figure 13.16 PERI UP Scaffold Being Shifted Using Crane (Courtesy PERI).

13.7 GALVANIZED SCAFFOLDS

For extensive use in coastal areas or in highly corrosive locations, galvanized steel scaffolding is preferred to the ordinary painted steel scaffolds. Although galvanized scaffolding is expensive initially, substantial savings are made in maintenance costs in the long run due to its longer life on account of corrosion resistance.

13.8 SCAFFOLD BOARDS

Scaffold boards for platforms are generally in timber, particularly in pine wood because of its light weight and strength. The thickness of the scaffold boards is generally specified in the standard specifications/ codes for different clear spans. The width of the board is generally 230 mm. Apart from timber boards, steel planks are also available. These are generally made of thin M.S. sheets with presses of cold formed flanges and provided with anti-skid surface treatment. The steel planks are also available in galvanized condition for use in the corrosive weather. It should have been noted that steel planks would not be suitable for platforms in extreme tropical climate and also where oil/ grease or other such slippery materials is likely to fall on the platforms.

13.9 SCAFFOLDS FOR HIGH CLEARANCE STRUCTURES

The high-clearance structures such as museums, gymnasiums, auditoriums, warehouses, exhibition halls, etc., utilize the door-type tubular steel scaffolds during construction, mainly for economic reasons (Peng *et al*. 1996).

A scaffold support system, often a combination of the modular steel scaffolds and wooden shores, is typically used for temporary support during the construction of high-clearance concrete buildings in Asia. Such temporary support systems are often used because they are convenient, versatile and relatively inexpensive.

Door type tubular steel scaffold units come in standard shapes. It is natural that they do not completely fill the full inner clearance of the structures, in spite of the frames being stacked one above the other. Under such circumstances, wooden shores are sometimes used to fill the balance clearance of the structures. This is shown in a schematic sketch in Fig. 13.17.

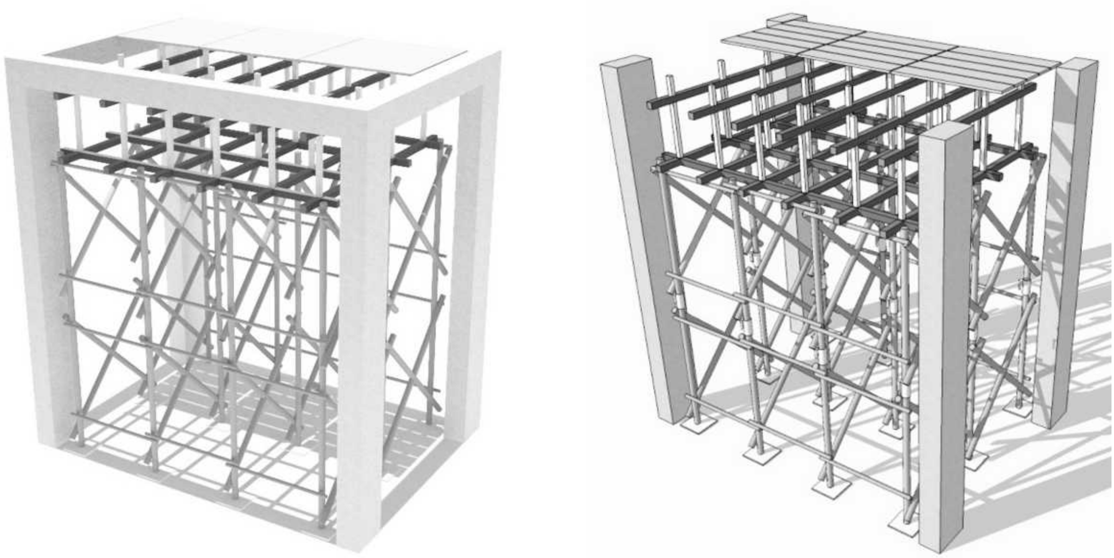


Figure 13.17 Typical High Clearance Scaffold System Arrangement.

13.10 DESIGN ISSUES

A scaffold support system is a temporary structural system used to support the construction loads such as fresh concrete, workers, formwork, steel, etc. After the construction is complete, the scaffold support system is generally removed. As a temporary structural system, its importance is often overlooked by the design engineers.

There is limited availability of guidelines for the design and use of scaffolding during construction. Designers also have little information describing the actual cases of scaffold support system collapse. Often, the construction engineers rely on their own experience in view of the lack of guidelines. Formwork crews also install the formwork and wooden shores based on their own experience, and in general, are unable to match the quality of the work done by the scaffold crew. Since quality gaps exists between the scaffold crew and formwork crew, it is difficult to control the overall quality of the entire temporary support system. This has led to the collapse of the scaffold support system during the construction of high-clearance structures.

Peng *et al*. (1996) developed a computer model based on actual construction practices to provide a description of the likely causes of scaffold support system collapse during construction. The analytical results of this research combined with the experience of the scaffold crew, Peng *et al*. (1996) came up with some recommendations for scaffold use by the design and construction industries. These are given below:

1. Scaffold support systems should use jack bases, even on a concrete foundation. This will allow for: first, an increase in system stiffness at the base; secondly, adjustments to different heights off the ground (such as when stairs are being built); and finally, adjustment of height due to improper or uneven installation of steel scaffolds. If a system of steel scaffold is twisted because of improper installation, some of the scaffolds may not be in contact with the ground, which may lead to instability problems.
2. The wooden planks beneath the wooden shores should be fastened directly to the dried reinforced concrete columns and walls.
3. Nails should be used at both the top and the bottom of the wooden shores in the scaffold support system.
4. In addition to using bamboo cross-braces, horizontal bamboo bracing should be installed at the top of the first story of the scaffolding (on the in-plane surface of the steel scaffold). The bamboo and the steel scaffolds should be fastened together with wires in order to prevent buckling of the bamboo. In addition, it is recommended that the bamboo braces on the out-of-plane surface of the steel scaffolds be replaced with steel braces since bamboo's capacity to resist the bending moment may be inadequate for this application. Above the first story, this type of horizontal bamboo and steel brace should also be used after every two or three stories in the case of multi-story steel scaffolds. In principle, these horizontal braces should also be fastened to the reinforced concrete columns and walls.
5. It is recommended that the designers specify the locations on the reinforced concrete walls and columns for wall connections for attaching the horizontal bamboo and steel braces and the wooden planks.
6. If possible, the wooden shores should be replaced with tubular steel adjustable shores. The end of these tubular steel adjustable shores can be connected the same way as the joints between steel scaffolds (i.e., in place of nail joints). This can greatly increase the critical load of the scaffold support system. If this is not possible, tubular steel adjustable shores should at least be used temporarily in the interior of the scaffold support system where the largest force is exerted.
7. Whenever possible, simple style steel scaffolds with one joint should be used. If possible, the manufacturers should enforce strict quality requirements for 'joint welding' and 'joint size' to control the stiffness and initial eccentricity at the joint of the steel scaffolds. In addition, the connection pins at the joints should be lengthened in order to increase the stiffness of the joints.

13.11 POSSIBLE CAUSES FOR COLLAPSE OF SCAFFOLD SYSTEMS

The collapse of the scaffolds not only leads to work delays and property loss, but it has also been responsible for numerous worker injuries and deaths. The probable causes are based on surveys of construction practice, existing construction accident records, and expert opinion.

Construction loads may exceed the critical load of the scaffold support system

According to Peng *et al*. (1996), the failure of a scaffold support system in high-clearance structures is usually a problem of structural stability. The collapse of the scaffold support system is often the result of the actual construction load exceeding the critical load of the system. At present, there are no guidelines which can be used to predict the critical load (i.e., the buckling load) of the entire scaffold support system.

Horizontal instability of the wooden shores

As shown in Fig. 13.17, it is difficult to accommodate the inner clearance of high-clearance structures by using a stacked arrangement of steel scaffolds alone and thus there is a need to use wooden shores at the top in combination with steel scaffolds. The wooden shores are used primarily for filling the gap between the formwork and steel scaffolds. Peng *et al*. (1996) reported that it is almost impossible for wooden shores to buckle under general construction loads. However, the connections at their ends are unable to carry moment and the bottom of the wooden shores may move horizontally together (diaphragm action) with the wooden planks on the top of the steel scaffolds after construction loads are applied on the formwork. When the angle at which a wooden shore is installed exceeds a certain value, the horizontal component of the resulting axial force exceeds the available frictional force at this contact point. Thus, these angled wooden shores may fall out of position, causing the formwork on the top of the wooden shores to collapse, along with the freshly placed concrete. This, in turn, may lead to a sudden collapse of the entire scaffold support system.

Partial loading of the fresh concrete may reduce the critical load for the scaffold support system

The concrete pour duration for high-clearance structures is usually one day except in the case of a very large slab pouring area. Peng *et al*. (1996) opine that, the concrete load over the complete slab area may be considered to be a sequence of different partial load cases. After the critical loads of each partial load case are compared, the minimum critical load may be assumed to be the critical load of the overall scaffold support system.

A specific (possibly asymmetric) placement pattern of fresh concrete may decrease the critical load of the system

The fresh concrete is usually placed according to some specific pattern. However, a uniform load is usually assumed in structural analysis. According to Peng *et al*. (1996), the placement pattern may cause the critical load of the temporary support system to be smaller than that under the uniform load assumed in the design. Thus, the design strength of the temporary support may be inadequate for the actual external load.

Zimmer and Bell (2006) report on the centre scaffold collapse of a 100-story building named John Hancock center located in Chicago. The scaffold wreckage killed 3 motorists and severely injured several other passers by. Zimmer and Bell (2006) investigated the technical and procedural causes of this failure and found that under the platform's self weight and the down draft wind loads acting upon it, the uplift rollers and wire ropes lashing, holding the north outrigger to the building's roof track failed. The scaffolds were erected for the purpose of façade restoration.

Peng *et al*. (1996) presented a simplified procedure for estimating the critical loads for the scaffold system. They also investigated the effect of bamboo cross braces on the scaffold system. Bamboo cross braces are often used to provide increased stiffness of the entire scaffold system. The axial stiffness of bamboo is very large compared to the bending stiffness when the bamboo is attached to the scaffold system. However, bamboo's ability to resist is poor and it fails easily in flexure.

13.12 CHECK LIST

The check list (adapted from Peng *et al*. (1996)) for the installation of simple-style scaffold support system is provided in Table 13.2. The examiner needs to put yes or no in the appropriate column depending on the actual condition.

Table 13.2 Checkl ist for Installation of Simple-style Scaffold Support System

Sl. No.	Checklist item description	Compliance in terms of yes or no		Remarks
		Yes	No	
1.	Are jack bases installed at the bottom of all steel scaffolds?			
2.	Are adjustable shoring heads installed at the top of all steel scaffolds?			
3.	Are the exterior bamboo cross-braces tied with wires at both the ends?			
4.	Are the exterior bamboo cross-braces tied with wires in the middle every 2-3 meters?			
5.	Is the interior horizontal bamboo (on the in-plane surface of the steel scaffold) installed at the top of the first story of steel scaffolds?			
6.	Is the interior horizontal bamboo (on the in-plane surface of the steel scaffold) attached to the reinforced concrete columns and walls?			
7.	Is the interior horizontal bamboo (on the in-plane surface of the steel scaffold) tied with wires on at least one joint for every steel scaffold?			
8.	Are all wooden planks at the bottom of the wooden shores placed on adjustable shoring heads?			
9.	Are wooden planks at the bottom of the wooden shores fastened to the reinforced concrete columns and walls?			
10.	Are nails used at the top of all wooden shores?			
11.	Are nails used at the bottom of all wooden shores?			

REVIEW QUESTIONS

Q1. True or False

(a) Scaffolding is a permanent structure for gaining access to higher levels of permanent structure during construction.

(b) The three major uses of scaffoldings are—providing access, centering for the formwork, and supporting heavy loads at great heights.

(c) Scaffolds are classified according to the normal usage and on the basis of the type of construction.

(d) Safety is compromised in using Bamboo or *Bally* scaffolds for tall buildings.

(e) The two types of metal scaffolds in vogue are – Tube and fitting type and pre-fabricated unit – frame type.

(f) The two types of door type tubular steel scaffolds are— (i) finishing scaffolds and (ii) support or shoring scaffolds.

(g) Possible causes for the collapse of scaffold systems are –

  (i) Construction load > critical load

  (ii) Horizontal instability of wooden shores

  (iii) Partial load of fresh concrete may reduce the critical load for the scaffold system

  (iv) Asymmetric pattern of concrete placement

Q2. Match the following:

(i) Single pole scaffold	(a) Independent scaffold
(ii) Double pole scaffold	(b) Putlogs

Q3. Match the followings:

(i) Heavy duty scaffolding	(a) used by electricians, painters
(ii) Medium duty scaffolding	(b) used by bricklayers and masons and in concreting operations and for façade cladding work and demolition work
(iii) Light duty scaffolding	(c) used by carpenters, plasterers, glaziers, cleaners, welders.

Q4. List out the basic features of the pre-fabricated unit-frame type scaffold and discuss its various types.

Q5. List out and discuss the characteristics of various proprietary scaffolds like –L&T access scaffold system, PERI scaffold, galvanized scaffold, scaffold board.

Q6. Enumerate the various design issues related to scaffolding.

Q7. Do the critical analysis of the checklist provided in the text for safety is case of scaffolding.

Chapter

14 Formwork for Precast Concrete

Contents: Introduction; Advantages, Limitations and Reasons for Less Share of Precasting; Moulds for Precast Concrete; Precasting Process; Methods of Crew Organization in Precast Construction; Case Studies

14.1 INTRODUCTION

In this chapter, the formwork for precast construction is discussed. Precast construction is adopted when a large number of similar elements are to be produced in concrete in a number of civil and infrastructure works. Precast elements refer to the concrete building and structural elements that are made using moulds at a centralized facility and then transported to the building site to be assembled in the facility being constructed (Chan and Hu, 2002). Depending upon the handling facilities available, the size and shape of the precast elements are decided.

Precasting is ideally suited for mass scale production of concrete elements such as: floor elements, columns, lintels, door and window frames, railway sleepers, hollow blocks, wall panels, etc. Countries with a large public housing program have increasingly turned to the use of precast building/structural elements and site automation to increase site productivity (Chan and Hu, 2002). In case of multispan bridge construction, particularly where site conditions makes the provision of falsework difficult or expensive, precasting of bridge is usually resorted to in a casting yard close by, and then the elements are hoisted or launched into position by mechanical means. This method, apart from speeding up the work, ensures good quality of concrete in strength and in finish.

Precast construction offers a number of advantages. The precast concrete is the ideal material for health, safety and protection of the environment. Studies have shown that precast concrete products can provide a service life in excess of 100 years.

The precast construction process in general is also explained in the chapter. The decision to choose precast construction over *in-situ* construction is dependent on a number of factors such as: the production facility, the cost of transportation, the number of precast elements to be produced, the size and shape of precast members to be produced, and so on.

Precast concrete elements can be produced either in dedicated precast concrete factories or in a precast yard specifically developed for catering to the needs of a project either on the site itself or at a distant location depending on the availability of land and other facilities. In India, there are very few companies which produce precast concrete elements depending on the client's requirement in a factory set-up. Hindustan Prefab Limited is such a company which produces a variety of precast concrete products useful in different applications. The company produces precast pre-stressed bridge girders and slabs, railway sleepers, portal frames and sections, pre-stressed concrete electric poles, fencing posts, precast columns and many other products to suit the client's requirements. The products of this company have been used in a number of reputed government and private projects. The details of the company can be found at http://www. hindprefab.com. Hindustan Prefab Limited is a government owned company established soon after independence and has a long history. In India, however, there are not many private companies which produce precast concrete elements. Most of the times, the precast concrete elements are produced by the contractors in the precast yard, at or near the work site, to produce the project specific demands. After the needs are satisfied, the precast yard is dismantled. Such precast concrete production offers limited opportunity to use the principles of industrialization as opposed to the factory based precast concrete production. Also, limited opportunity of standardization and automation exists in such a method of precast concrete production. The discussion in this chapter is limited to precast concrete production by the contractors in the precast yard and not in the dedicated factories.

Different types of forms or moulds are discussed. Some cases in precast construction with emphasis on the type of forms adopted are also provided.

14.2 ADVANTAGES, LIMITATIONS AND REASONS FOR LESS SHARE OF PRECASTING

14.2.1 Advantages

Precast construction, as opposed to the *cast-in-situ* construction of structural and architectural concrete elements, offers a number of advantages to different project participants. Some of them are given below:

- Precasting ensures accuracy, uniformity, and good quality concrete under well-controlled conditions. This is possible because compared to the *cast-in-situ* construction, precast uses comparatively high-end technology and advanced equipments.
- The cost of the formwork is generally less in precast construction. This is because less scaffolding and less number of forms are needed. The requirement of scaffolding is less because the precast operations are performed mostly at the ground level. Lesser number of forms are needed because it is possible to plan for more reuses in precast construction. Another reason for the decrease in cost is the decrease in overall project duration. The precast construction offers saving in time as it can be taken up concurrently along with the *cast-in-situ* works, such as the foundation and basement works. The production and erection processes do not get affected by the vagaries of weather and the great fluctuations in the laborers' productivity which further results in saving in the project execution time.
- Further, there is less requirement of the on-site labor and there is less wastage of materials in production and erection.

- Concrete placement is easier and better controlled. The finishing operations are simplified because workers have easy access to the forms, and improved control of the finish quality is possible. It is possible to achieve better architectural appearance for the precast concrete elements.
- Precast construction offers more productivity of crew members as they work at the ground level. The reinforcing steel can be placed accurately and its position maintained.
- Vertical transportation is simpler; only the completed piece and minimum of scaffolding needs be lifted.
- The precast concrete increases efficiency because the cold and hot weather, and the rains do not interfere with the production schedule and do not delay production. In addition, weather conditions at the jobsite do not significantly affect the schedule. This is because it requires less time to install the precast as compared to the other construction methods, such as cast-in-place concrete.
- Because the precast concrete products are typically produced in a controlled environment, they exhibit high quality and uniformity. The variables affecting the quality typically found on a jobsite— temperature, humidity, material quality, craftsmanship— are nearly eliminated in a plant environment.
- The precast concrete products produced in a quality-controlled environment and used with high-quality sealants, offer a superior solution to the water-tightness requirements. Standard watertight sealants are specially formulated to adhere to the precast concrete, making watertight multiple-seam precast concrete structures possible.
- Because of the standard and modular nature of many precast concrete products, structures or systems of nearly any size can be accommodated.
- The precast does not require the use of specialized heavy cranes. The precast concrete elements can be installed in a short time, as these elements are designed and manufactured for relatively simpler connections.
- Precast concrete elements can be shaped and moulded into different sizes and configurations to serve the functional and aesthetic requirements. The elements can be produced in virtually any color and a wide variety of finishes, to achieve the desired appearance.
- Precast concrete requires little or no maintenance, which makes it an ideal choice for nearly any design solution.
- Precast concrete products arrive at the jobsite ready to be installed. There is no need to order raw materials such as reinforcing steel and concrete, and there is no need to expend time setting up the forms, placing concrete, or waiting for the concrete to cure.

14.2.2 Limitations

Unlike the *in-situ* concrete, one does not have to compromise on the dimensional accuracy in precast concrete construction. The specification on deviation from the planned dimension of the precast concrete elements is relatively stricter. In addition, proper upkeep of the formwork to guard against dimensional changes is a must. Also, it requires more attention in stacking and transporting, and we have to take care while erecting the elements in position. For erection works, the site must have some lifting device. Precast concrete construction involves repetitive work, thereby bringing monotony in the design and hampering creativity and innovation.

14.2.3 Reasons for Low Share of Precast Construction

It can be easily observed that precast concrete construction offers numerous advantages and few limitations. This is truer in a factory like set up. In spite of the advantages the method offers, we find that its use is relatively low both in the developed as well as the developing countries. According to the study conducted by Polat (2010), the average share of precast concrete systems in the construction industry across the European Union is 20–25%, and it goes up to 40–50% in the northern European countries. The share of reinforced concrete construction supplied by precast producers is 6% in the U.S. (Sacks *et al*. 2004), and it is only 2% in Turkey (YEMAR Report 2006). The share of precast concrete systems of the overall building construction market in the U.S. is approximately 1.2% (Eastman *et al*. 2003; Sacks *et al*. 2004). According to European statistics (Sacks *et al*. 2004), precast structures account for 56% of the building structures in Finland, 28% in Germany, 26% in Britain, and nearly 20% in Spain. The reasons why the precast concrete systems are not extensively used in the US and Turkey were also identified by Polat (2010), and Arditi *et al*. (2000), and they are given below:

- Lack of qualified workforce specialized in precast concrete structures;
- Limited variety of precast concrete components;
- Inferior performance of precast concrete systems in the recent earthquakes occurred in Turkey;
- Public policy and political concerns;
- Lack of expertise in precast concrete design; contractors' unawareness of significant cost savings; and major shortage of expert personnel that can design and manage precast concrete building systems;
- Size/weight restrictions on truck loads have a significant impact on the design of precast concrete systems, which in turn hinder the extensive utilization of these systems.

A research needs to be carried out to find out the share of precast concrete construction in the Indian Construction Industry. As it is, it appears that the percentage share of precast construction in Indian scenarios is quite low. The reasons for the low share should also be researched. Most of the reasons cited above seem to be applicable to the Indian Construction Industries as well.

14.3 MOULDS FOR PRECAST CONCRETE

As the moulds are subject to a large number of repetitions, they have to be sturdy enough to maintain their shape and dimensions. They are also subject to vibrations if the concrete is vibrated with external mould vibrators. The surface of the moulds should be sufficiently hard so that they do not get damaged during repeated use. The dimensional accuracy in the mould is very important so that when the precast members are erected in position and assembled to obtain the finally required layout and dimensions, they all fit into each other within the tolerances.

The mould bottoms are usually stationary and the sides are removable. The stationary part of the mould is usually made in concrete and the removable part in structural steel.

The mould should have sufficient chamfers and side batters for smooth demoulding. Excessive adhesion of the concrete to the mould can damage both the unit and the mould and can result in poor finish.

To avoid this danger, form release agents can be applied to the mould surface before concreting.

The moulds should be mounted on the rubber pads to enable external vibration with the mould vibrators. The moulds should be supported on firm foundations so that there are no support settlements due to the vibration and the moist soil ground.

The moulds used for precasting differ from those used for *in-situ* construction in many respects, such as:

- Robust construction to withstand large repetitive uses;
- Simple assembly and stripping devices to ensure high turnover of moulds and reduced handling costs;
- Absence of elaborate falsework;
- Incorporation of special arrangements within the moulds for accelerated curing, such as steam jackets where required and vibrators for compaction.

The choice of the mould type and shape depends upon the following:

- The total number of elements to be produced;
- The desired rate of output;
- The shape and construction feature of the element;
- The facilities available at the production yard for casting, curing, storing and handling; and
- Economic aspects.

14.3.1 Horizontal and Vertical Castings

Precasting is done either by horizontal moulding or by vertical moulding. The choice between the two methods again depends upon the nature of the element and space available for storage. The horizontal casting method is favored for ribbed or curved elements, multilayered elements, which require a particular surface finish. Vertical castings are favored for single layer slab panels, which require no special finish on their surface, such as in the case of internal wall panels, which also have no protruding reinforcement from their edges. Mechanical handling facility will result in quick turnover of moulds.

The horizontal moulds may be fixed or tilted type. In fixed moulds, moulding and demoulding is done in the horizontal position whereas in tilting moulds, moulding is done in the horizontal position and demoulding is carried out when the moulds are tilted to almost a vertical position.

Shutter vibrators are often mounted on the underside of the mould for horizontal casting and on the sides for vertical casting. Individual mould or a battery of moulds may be used in vertical casting. Internal walls or floor panels are often cast vertically using battery moulds. Vertical moulding is the best for the panels, which require smooth surface on both sides.

14.3.2 Stationary and Mobile System

The methods of precasting with reference to the position of the mould can be broadly classified as the stationary system and the mobile (traveling) system. In the stationary system, all the production activities are performed in one place. In other words, the precast element hardens and is ready for demoulding at the point where it is moulded. In the stationary system, the horizontal or battery moulds are usually used. In the mobile system, moulds are moved on a rolling line or a conveyor from one workstation to another till the elements are hardened, with different activities being performed at each workstation. In most of the cases, the movable mould line is connected to a curing chamber. Horizontal moulding in the stationary system is the most universal method. In its simplest form, this mould comprises of only edges or side form plates placed on hardened type. In contrast to vertical moulding, this method demands a large production area.

14.3.3 Brick/Masonry Moulds

Brick/Masonry moulds can be used advantageously for the construction of some precast elements such as channel units, angles, purlins, hyperboloid shell units, etc. They can also be used to construct various precast elements used in the buildings. These are commonly used in small-scale production, particularly in producing patterned finish on the façade panels of buildings.

In this mould, bricks/stones are used to construct the surface in an approximate shape. The surface is then plastered. Alternatively, a layer of RCC is also sometimes laid on top of the brick/stone surface. The side faces are tapered so that it is easy to demould the cast concrete elements (Fig. 14.1).

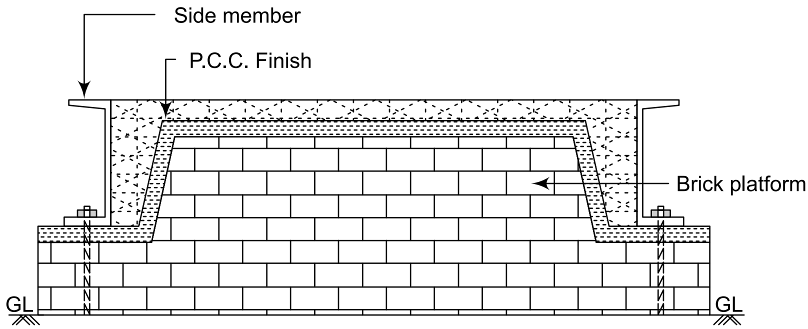


Figure 14.1 Brick/Masonry Mould in Position.

A release agent is also applied to prevent concrete sticking. A coating of paraffin wax is also sometimes applied to the moulds. The wax helps in blocking all the micro pores.

Some of the advantages offered by brick/masonry moulds are given below:

- accurate dimensions can be maintained constantly;
- more reuses are possible as compared to timber moulds;
- it requires minimum maintenance.

14.3.4 Wooden Moulds

Wooden moulds are suitable where only a few units are to be produced. These moulds are mostly adopted for small production programmes. Intricate detailing is possible in the case of wooden moulds and thus the wooden moulds can be used for providing architectural features also. Release agents are applied to the mould to prevent the adhesion of the concrete. For providing a smooth finish to the concrete surface, wooden moulds are coated with smooth linings, such as plastic sheets or thin sheets. Thin sheet metals are also sometimes used in the wooden moulds to give a smooth finish. The wooden moulds deteriorate rapidly on account of wear and tear, and thus they need repair and maintenance regularly to produce good quality concrete elements consistently. A typical wooden mould for the construction of precast RC beams is shown in Fig. 14.2. The dowel at the end is also shown in figure.

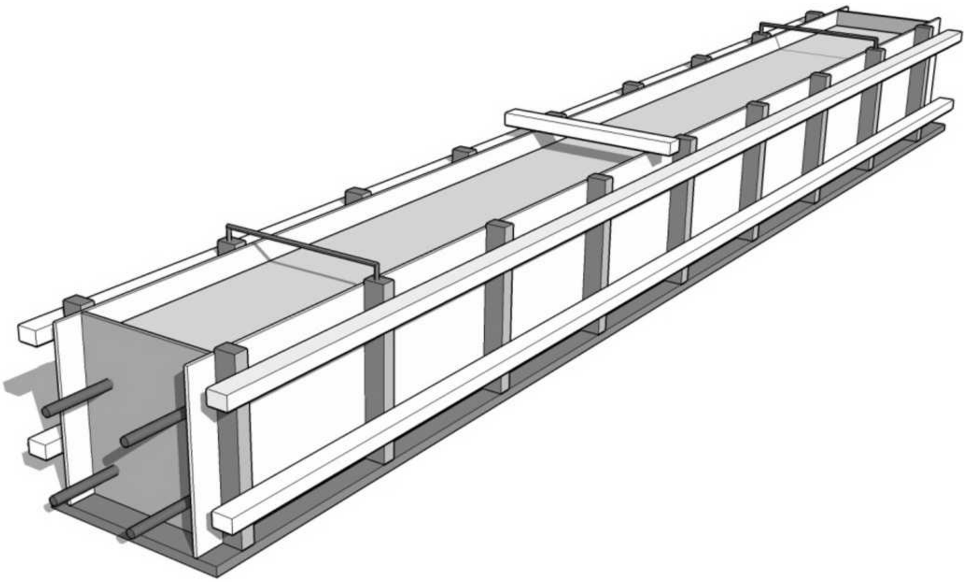


Figure 14.2 Wooden Mould for Casting RC Beams.

14.3.5 Steel Moulds

Steel moulds completely satisfy the requirements of volumetric stability, large number of repeat uses, less adhesive to concrete; but they are relatively more expensive compared to wood. But when large number of elements are to be moulded, steel moulds are by far the most economical. Steel moulds are sturdy and hence vibrators can be easily mounted on them. It is important to take care while designing the moulds that only fewer parts are to be dismantled for demoulding. This will ensure dimensional control and stable moulding. Release agents can be applied to prevent adhesion to the concrete. Figures 14.3 and 14.4 show two precast yards in which steel moulds have been used. In Fig. 14.3, the steel moulds have been used for casting trough slab while in Fig. 14.4 the steel moulds are used in casting the wall panels of an underpass.

Figure 14.3 Steel Moulds for Precast Trough Slabs.

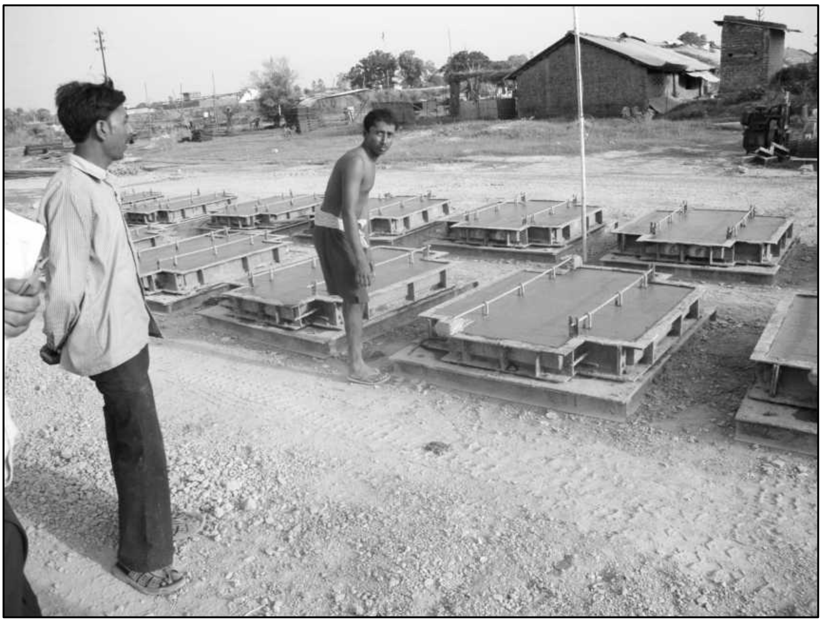


Figure 14.4 Steel Moulds in Use for Casting Wall Panels for an Underpass.

14.3.6 Plastic Moulds

In the industrially advanced countries, moulds made of glass fiber reinforced plastic are very common. Plastic moulds being light can be easily transported. They have better resistance, they do not deform, they are rust proof, adhesion to the concrete is less and hence dismantling is easy. In combination with wood and steel, plastic can be used for producing wall panels and also for producing patterned or architectural finishes. Resin coated and plastic lined surfaces are most suitable for producing smooth concrete finishes. This type of mould is also becoming common in the country especially for casting waffle slabs.

14.3.7 Vibratory Moulds

Small components like lintels, fencing panels, grills, kerbstones, are generally cast on vibratory tables. These vibratory tables are usually adopted in the assembly line production where concrete is poured in the moulds kept on the vibratory table. The vibratory moulds ensure proper compactness of the concrete.

14.3.8 Stressing Moulds

The development of the assembly line production methods and the use of pre-tensioned production in building industries necessitated the design and use of self-stressing moulds. Self-stressing moulds are usually made of steel and two ends are designed for the end thrust. Various pre-tensioned products like floor unit, panels, railway sleepers, are cast in these moulds.

14.4 PRECASTING PROCESS

The various operations involved in precast construction for a precast yard based production are described in detail in the following sections.

14.4.1 Study of Drawings

From the available drawings, an element-wise detailed schedule is prepared. The details should cover the dimension related information, as well as the reinforcement details, the grade of concrete, and the lifting arrangements. The time available to produce individual elements should be arrived from the time schedule of the entire project. Sufficient leeway should be given for curing and transportation from the production location to the erection location.

14.4.2 Precast Yard Development

Once the decision in favor of precasting has been arrived at, sufficient land for the precasting operation is to be developed. The land, depending on the availability, can be at the project location itself or it may be developed at some distant location. An orderly layout of the precasting yard saves time and labor. There must be sufficient space to arrange the casting bed. Space should also be there to tie the reinforcement and to allow for the movement of transit mixers and transportation vehicles. The selection of the area for locating precast construction is dependent on a number of factors. For example, the casting yard should not be located in a low lying area. There should be proper drainage facility at the entire casting yard. The casting area should be leveled and it should have easy access to the erection areas. There must be sufficient space available for stocking the precast elements for the designated number of days. A typical layout for a precast yard is shown in

Figs. 14.5-14.11 schematically while the actual layout adopted by one of the contractors engaged in Delhi Metro construction is shown in Fig. 14.12. The places for performing different activities are clearly marked in Fig. 14.5. Depending on site constraints, the actual layout may vary. Figure 14.6 shows the details of the arrangement adopted for the space for the mould shown in Fig. 14.5. Further blow up of Fig. 14.6 is shown in Fig. 14.7. Figure 14.8 shows the plan of Mould P-2, while Figs. 14.9, 14.10, and 14.11 show the sections 1-1, 2-2, and 3-3 taken from Fig. 14.6 and Fig. 14.8, respectively.

Figure 14.12 Centralized Casting Yard.

The application of the gantry crane is a required feature in most of the precast facilities. The gantry cranes move on rail tracks. Thus, sufficient lengths of tracks are laid out in the precast yard to allow the movement of the gantry crane. After the rails are laid, the gantry crane is installed. The gantry cranes are helpful in lifting the reinforcement cage, forms for the precast concrete elements, and for shifting the elements to the stacking yard. The details of a gantry crane are provided in Figs. 14.13-14.15. Figure 14.13 shows the plan of the gantry crane while Fig. 14.14 shows the sectional view. Figure 14.15 shows the details of rails fixing. The gantry cranes travel on rails and thus can cover a large area of the precast yard.

14.4.3 Casting Bed Preparation

The type of casting bed used will depend on the items to be cast and the type of forms to be used. For shapes to be cast using the bed as a bottom form, the surface should be level, smooth, hard, and free from cracks and holes. The bed must be watertight and rigid so that the weight of the concrete does not deform it and the passing equipment does not cause movement of the forms. The foundation for casting beds, which carry heavy loads, should be designed to prevent differential settlement.

14.4.4 Formwork/Mould Preparation

The requirements of formwork for precast construction are also similar to what has been discussed in Chapter 1. The form builder is concerned not only with making forms of the right size,

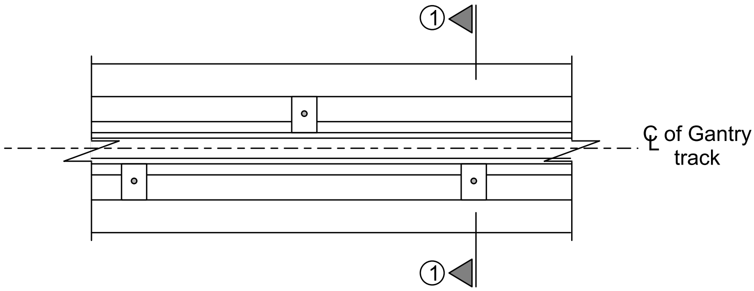


Figure 14.13 Plan of Gantry Rail.

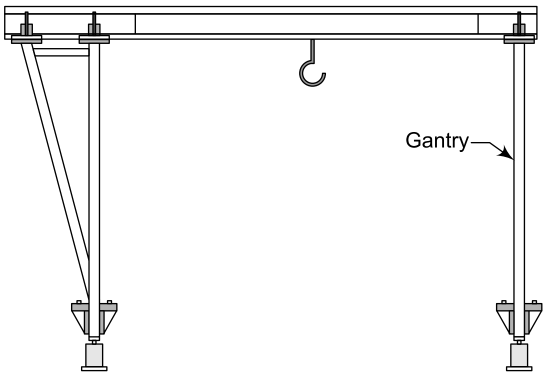


Figure 14.14 Sectional View of a Typical Gantry Crane.

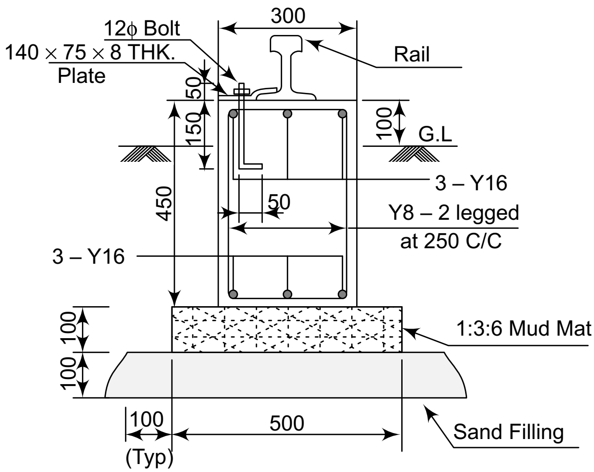


Figure 14.15 Section 1-1 of Figure 14.13.

his objectives are to see that the formwork is erected and de-shuttered fast and in a safe manner, and good concrete quality and surface finish is achieved. In order to achieve these objectives, it is essential that the right formwork system with the right detailing is adopted.

The materials chosen for making the form or the mould is governed by factors such as the shape of the element to be cast, the complications involved, the accuracy desired, the type of finish that is desired, and the number of reuses. The duration of precasting and the cycle time of precast operations govern the reuse of forms. Since an economical precasting operation frequently depends on early stripping of the forms for reuse, provision for adequate curing without the forms should be made. The commonly used form materials are timber, steel, aluminum, and glass fiber-reinforced plastic.

For limited reuse, untreated wood/timber is a better option. For large production, they are difficult to strip and costly to keep in repair. Metal moulds are suited for large production. They have provision for adjustment to varying sizes. Plastic and glass-reinforced plastic are also suited for large production as they give prolonged service with proper care.

Proper care should be taken to ensure the line and level of the precast element. Also, beading/chamfer wherever provided should be properly executed.

14.4.5 Reinforcement Tying/Strand Laying

Reinforcement can be tied either at the form location or can be transported to the formwork from some other place. Care should be taken for detailing at the bracket and pocket locations. Proper arrangement for cover blocks should also be kept. It should be ensured that the reinforcing steel used for precast construction is from an approved source. The tests provided in the Indian Standard (IS: 1786-1985) for checking the suitability of reinforcement should be performed at specified intervals. The reinforcement tying should be as per the approved drawing and using approved binding wire according to IS: 2502–1963.

14.4.6 Materials for Concreting

The materials required for the concreting operation should be procured well in advance and relevant tests for its suitability should be conducted at regular intervals. The chemical composition and physical tests on cement such as fineness, consistency, initial and final setting time, and compressive strength at 3, 7 and 28 days should be done once per batch of the cement in accordance with the provisions of IS: 8112–1989 and IS: 12269–1987. The materials used for concreting should be stored properly to avoid deterioration due to moisture and contamination by various impurities. The tests on coarse aggregate and fine aggregates should also be conducted at the specified frequency based on the provisions of IS: 383–1970 and IS: 2386 (Part 1 to 8)–1963. The concrete of specified grade can be designed based on the available ingredients.

14.4.7 Concreting

Concrete can be produced in the precast yard area itself or can be procured from RMC plants. In the latter case, the grade of the concrete should be doubly ensured to avoid any mistake. The workability and compaction should be ensured especially near the bracket and pocket location. Before placing the concrete, cleaning of the mould should be ensured. The area to be concreted must be free from debris and any extraneous matter. During the concreting, samples must be collected for checking of compressive strength and workability as per the requirement.

14.4.8 Compaction

Compaction should be ensured while concreting. The vibration can be given in the form of immersion vibrators or through mould vibrators. It is always useful to vibrate the concrete optimally. It is always better to over-vibrate than to under-vibrate, though optimal vibration is always desirable. Special care should be given while designing the form to take care of vibration. For this, the mould should be sturdy and joints secured. The mould should not lose dimensional accuracy once vibrated.

14.4.9 Handling

For lifting the concrete unit from the mould, suitable equipments like gantry on rail or a crane of sufficient capacity can be used. Suitable provision for handling the precast concrete element must be made at the time of casting.

All precast units must be adequately shored and braced in position until final connections are made and cured if cast in place concrete forms part of the joint.

The precast units, usually, do not need any finishing. However, sometimes minor touching up may be required, and for this purpose, after lifting from the mould, they can be placed on raised pedestals to obtain sufficient headroom below the soffit of the units.

14.4.10 Curing

Curing by spraying water or flooding, and applying a curing compound on the surface of the units are some of the commonly adopted methods. In order to reduce the curing time, there are methods to accelerate the process of hydration and thus gaining strength by the concrete. Curing of precast elements can be done in any one or the combination of ordinary curing, steam curing, and chemical curing. In ordinary curing the concrete element is covered with wet jute bags. Water may be sprayed with curing pipes, or water sprinkler. Sometimes flooding with water is carried out in ordinary curing. In steam curing, either the mould is heated up or encasing of the concrete element is done and steam is provided. In chemical curing, chemical compounds are applied to the concrete element which serves the purpose of curing.

14.4.11 Stacking/Transporting Arrangement

Stacking should be done in a manner to avoid damage to a precast element. Preferably stacking should be done in a manner to avoid multiple handling. Stacking should be done as per the designer's instructions and wherever supports are needed, they should be provided.

The choice of transportation vehicle will depend on many factors. Here again, proper care should be taken to avoid any damages. Transportation of elements should be done in a sequential manner and it should be governed by the erection schedule. For this, identification marks on each element should be provided.

14.4.12 Erection and Grouting

Erection of precast elements can be done either manually or through machines. It can be either crane or some other type of hoists depending on the weight and shape of the precast element. All precast elements must be safely and adequately secured and braced in position immediately after erection. If the precast elements are only a week or 10 days old at the time of erection, which normally is the case, connection must be designed to accommodate shrinkage. Jointing of different precast elements can be done either by casting a portion of it *in-situ* or by grouting it, as the case may be. Grouting is done during morning / evening hours to avoide high ambient temperature and grout temperature.

14.5 METHODS OF CREW ORGANIZATION IN PRECAST CONSTRUCTION

There are different ways in which the work at the precast yard can be accomplished. For example, in one of the methods, separate crews are mobilized for performing different tasks. Carpentry crew is assigned for mould setting and demoulding, reinforcement crew is assigned for reinforcement cutting, bending, and tying. This crew may tie the reinforcement at the casting bed or it may bring the prefabricated reinforcement cages from the reinforcement yard to the casting bed. Concreting crew performs the concrete pouring, compacting, and finishing task. This crew is also responsible for minor touch up work that may be required to be done after demoulding. Rigger crew may be assigned for shifting the concrete elements from the casting bed to the curing location and subsequently to the stacking and finally to the erection location. This is very common in the Indian precast construction works.

The above system works well when the concrete elements to be produced are identical. There is not much idling of crew in this method as the crews can work at a balanced pace. The crew sizes are determined keeping in mind the pace required. This system works better if mobile moulds are used. In mobile moulds, the crews are stationed at their respective workstations and the mould reaches them. Thus crews don't have to move beyond their respective workstations. After one set of crew performs its assigned task, moulds are moved to the next workstation where another set of crew performs its designated tasks.

In case of stationery mould, the crews have to move to the location where moulds are placed. After one set of crew performs its designated tasks they, leave the mould location and the other set of crew enters to perform its assigned tasks.

14.6 CASE STUDIES

In this section, we discuss a few live cases wherein extensive precasting of different kinds of concrete elements was undertaken. The elements ranged from simple ones like precast beams, columns, spiral staircase, very thin pre-stressed beams, to complicated ones like waffle slabs, I-Girder, H-Girder, Shell elements, Tunnel elements, and Folded plate elements.

14.6.1 Precast Beams, Columns, Spiral Staircase, Precast Pre-stressed Beams for a Natural Draft Cooling Tower

A large number of concrete beams and columns were to be cast for natural draft cooling towers within a short span of time. These beams and columns formed part of the basin works in the cooling water. The basin consisted of RCC floors of about 100 m diameter. All along the perimeter, there was a reinforced cement concrete wall of about 2.5 m height. The basin supported a framed skeleton consisting of precast columns and beams. Columns and beams were of varying cross sections depending on the load carried out by them. Columns and beams were cast in a precast yard located about 50 m from the cooling tower location. Details of the casting arrangement for a typical precast beam are shown in Figures 14.16 to 14.24.

14.6.2 Precast Waffle Slab, and Bubble Dome for Parliament Library Building

A large number of bubble domes which are part of the larger spherical dome were to be constructed in a very short time for the construction of the parliament library building in New Delhi. Besides, a number of difficulties were anticipated if the bubble domes were to be *cast-in-situ*. All these factors were responsible for the contractor to select the precasting option. M-50 concrete grade (fiber reinforced) was used for the casting of the bubble domes. The bubbles were square, polygon and triangular shaped in the plan. The bottom surface curvature was in two directions and each of the bubble domes was weighing approximately 5 tonnes.

The bubble domes were cast at a platform at about 1.5 m height above the ground level. The entire formwork was supported on heavy duty tower.

Figure 14.25 Casting of Bubbles in Progress for the Parliament Library Project.

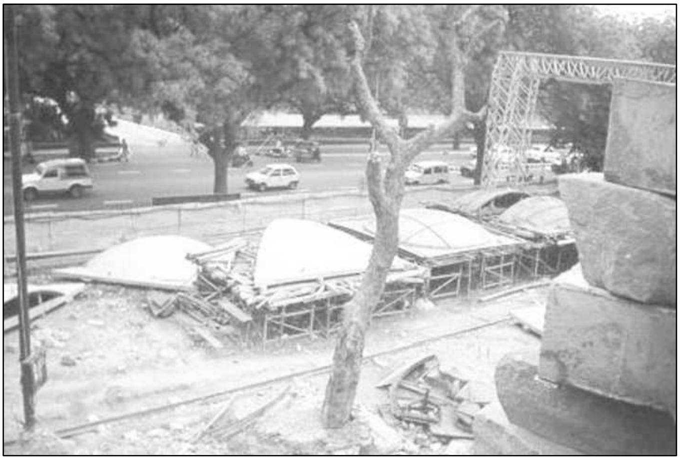


Figure 14.26 Another View of Casting of Bubbles for the Parliament Library Project.

14.6.3 Folded plates for ITC Saharanpur

Folded plates of 20.70 m length, 2.01 m flap width, and 1.90 m height were precast by Larsen and Toubro Limited, ECC - Construction Division for the construction of the factory building at ITC Saharanpur. The thickness of these folded plates varied between 40-50 mm. Concrete of M-40 grade was used for the construction and aggregates for 10 mm down were used for concrete production. The contractor used 2 moulds to meet the progress schedule. Each of these moulds was weighing approximately 25 tonnes.

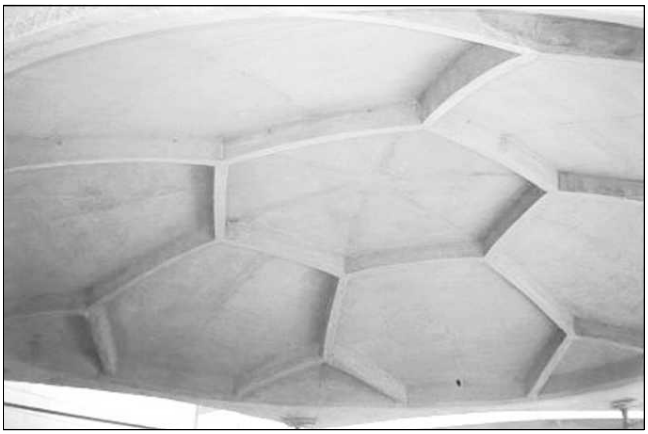


Figure 14.27 Rib Pattern on the Inner Surface of Bubbles for the Parliament Library Project.

The contractor had mobilized one senior engineer, one junior engineer, and one foreman for the precast work. A total of 90 workers of different specializations such as mould fabrication, mould setting, cutting, bending and tying of reinforcement, concreting, mould dismantling, rendering, steam curing, etc. were involved in the precast operation. The contractor mobilized two mixer machines, eight hydraulic jacks of 10 ton capacity, one hydraulic pump, two gantry cranes, three welding transformers, ten 10 ton capacity turn buckles, and many other small equipments for the casting work.

For the casting of folded plates, the following step by step process was used:

Step 1: Moulds were cleaned and release agents were applied on the inside mould surfaces.

Step 2: The reinforcement for flaps were tied, side shutters were fixed. The embedments were fixed according to the drawing.

Step 3: The flaps of folded plate were cast from the center to the ends.

Step 4: The flaps were lifted after about three hours and aligned along the centre line.

Step 5: The side shuttering of the diaphragm was fixed and reinforcement for the diaphragm and bottom slab was tied. The lifting hooks were fixed at appropriate locations as per the drawing.

Step 6: The outer shutter of the diaphragm was fixed keeping the pocket inside, in position and aligning before lifting the flap. Foams of 6 mm thickness were fixed at the junction of the bottom slab shutter and flap to avoid the leakage of slurry.

Step 7: The diaphragm and bottom slab were cast.

Step 8: Steam curing pipe line was fixed and hood was placed for covering and making a sealed container for steam curing.

14.6.4 I-Girder for Nizamuddin Bridge and Dwarka Flyover

Precast, pre-stressed girders are very common in the construction of bridges and flyovers. Two examples of the construction of precast, pre-stressed girders are presented. In the flyover construction at Dwarka, the arrangement of moulds for casting girders is shown in Fig. 14.28. The moulds used were fabricated of steel members. The bottom shutter was fixed to the firm bottom. Along the depth of the girder, tie rods at two locations were used. Alignment props were resting on the elevated concrete wall.

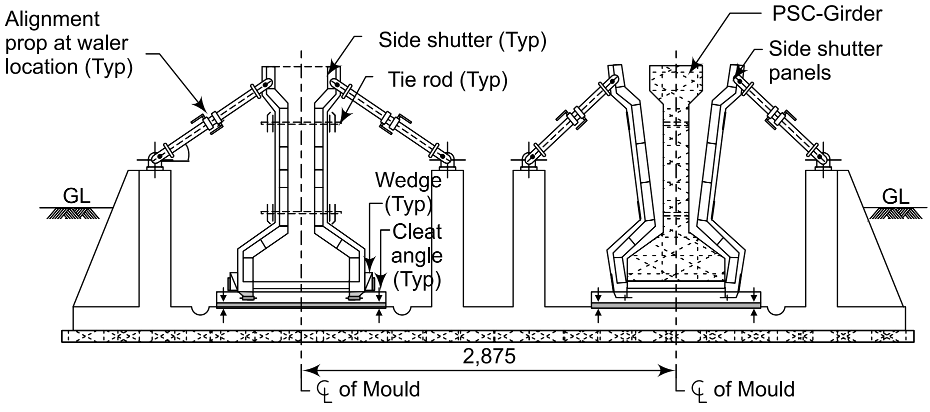


Figure 14.28 Mould Pit and Shuttering Arrangement for Precast I Girders.

A more or less similar arrangement was adopted for the casting of girders for the second Nizamuddin Bridge in New Delhi. The view of reinforcement and pre-stressed cables are shown in Fig. 14.29. The strongback for lifting the reinforcement cage and placing it on the casting bed is also seen in Fig. 14.29. Figure 14.30 shows the side shutters in position. The gantries used in the casting yard can also be seen.

14.6.5 Dome Elements for Kanteerva Stadium at Bangalore

Figure 14.31 shows the inside view of a sports stadium constructed in Bangalore. The dome segments were cast in the precast yard and erected. Figure 14.31 also shows the temporary support arrangement for dome segments at the centre of the dome. The view of the completed structure from outside is shown in Fig. 14.32. The structure won the ICI-MC Bauchemie Award for the 'Most Outstanding Concrete Structure' for the year 1995-96 from the Indian Concrete Institute.

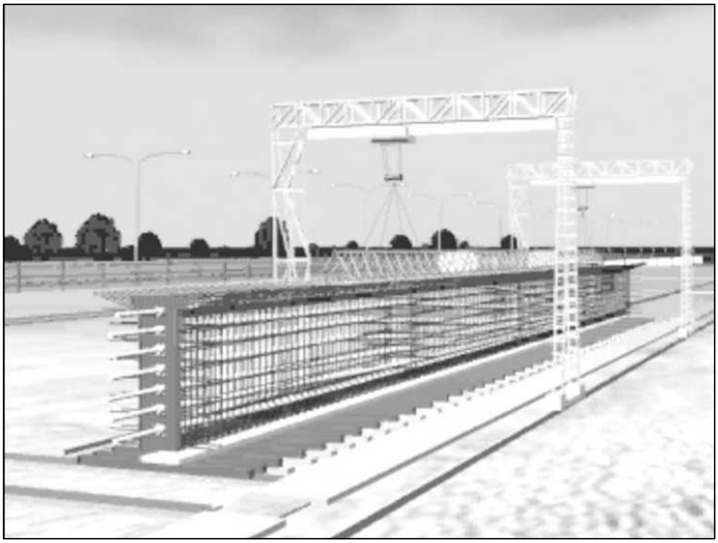


Figure 14.29 Casting Arrangement at the Precast Yard.

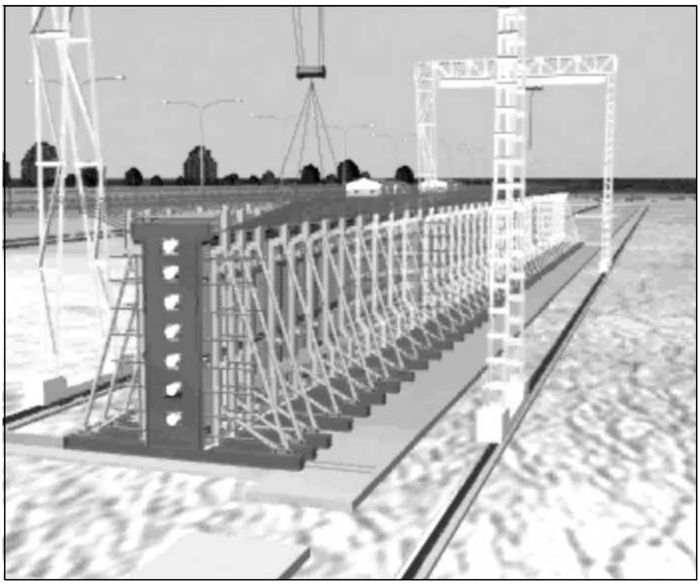


Figure 14.30 Another View of Girder Casting Arrangement at Precast Yard.

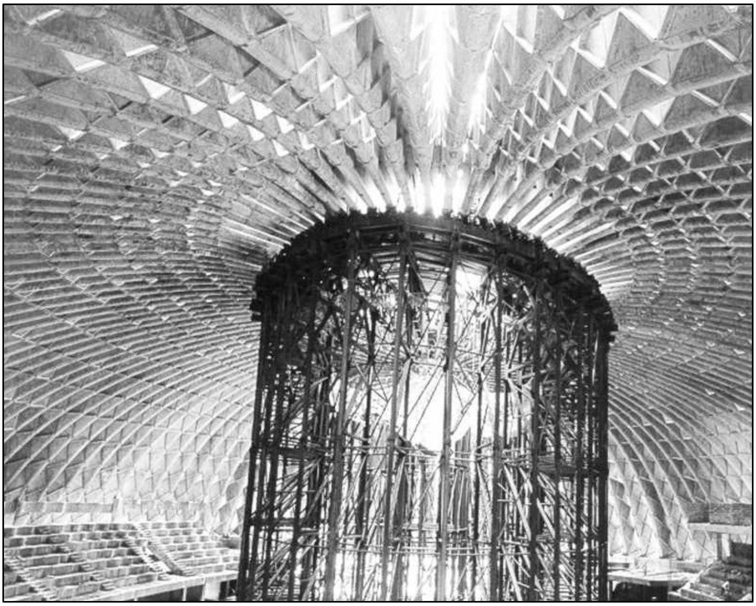


Figure 14.31 Inside View of Dome Segments Supported Temporarily at the Center.

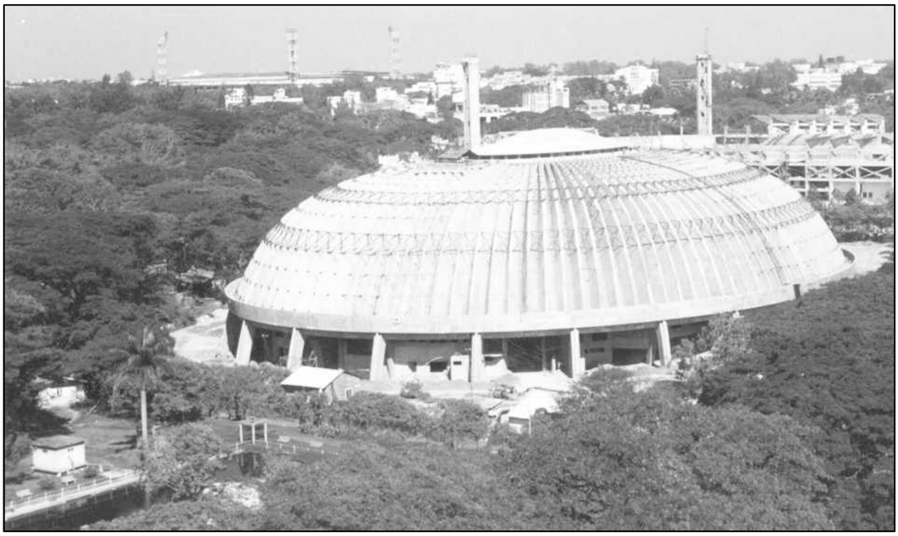


Figure 14.32 Completed Structure of Stadium.

14.6.6 Tunnel Elements for Delhi Metro Rail Project

Figures 14.33 and 14.34 show the two different views of the moulds used for casting the tunnel elements for the Delhi Metro Rail Corporation project in New Delhi. The moulds were made of special alloys capable of producing flawless elements again and again. The moulds were resting on an elevated platform.

Figure 14.33 Mould for Tunnel Element.

Figure 14.34 Mould for Tunnel Element in Position.

The mould has in-built provisions for grouting holes and other accessories (Fig. 14.34) to be used at the place of installation of the tunnel elements.

Prefabricated reinforcement cage (Fig. 14.35) is lowered into the tunnel mould.

Figure 14.35 Pre-fabricated Reinforcement Cage for the Tunnel Element Being Lifted.

14.6.7 Precasting of Crash Barrier

Figure 14.36 shows the casting arrangement for the crash barrier used for a bridge project. The side shutters are made up of steel plates and flats. The side shutters are aligned using props. The gantry is also seen in the figure (Figs. 14.36 and 14.37). The contractor used the hydra crane for shifting the crash barrier to the stacking location. Figure 14.38 shows the prefabricated reinforcement cages ready for being placed in the mould of the crash barrier. The cast crash barrier segments are shown in Fig. 14.39.

Figure 14.36 Casting Arrangement of Precast Crash Barrier.

14.6.8 Precasting in Delhi Metro Rail Construction

The contractors of the Delhi Metro project employed precast construction heavily. Some of the precast concrete elements used in the Delhi Metro project viaduct construction are shown in Fig. 14.40. Figure 14.41 shows the transportation arrangement for the slab segment of the bridge. The arrangement for low bed in the trailer can be noted. Figure 14.42 shows the stacking and curing arrangement in the storage area of the precast yard. The mould used for casting the slab segment and the prefabricated reinforcement cage can be noted in Fig. 14.43.

Figure 14.37 Handling Arrangement for Precast Crash Barrier.

Figure 14.38 Prefabricated Reinforcement Cages Ready for Placement in the Crash Barrier Mould.

Figure 14.39 Demoulding in Progress for Crash Barrier.

Figure 14.40 Pier Segment Being Raised (Note Lifting Tackle, Weight 75 ton).

Figure 14.41 Low Bedded Trailer— Transportation of Segment to Site.

Figure 14.42 Curing of Segments Using Sprinkler System.

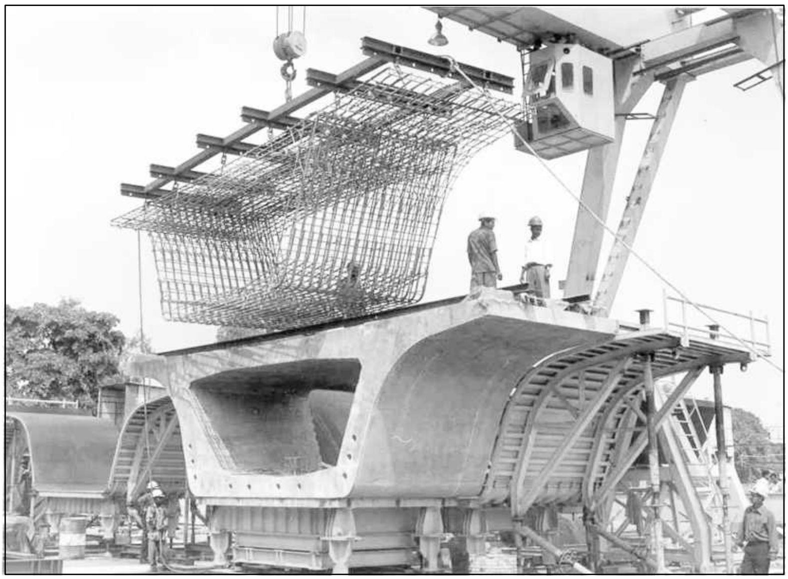


Figure 14.43 View Showing Placement of Reinforcement Cage in External Mould.

REVIEW QUESTIONS

Q1. True or False

(a) Precast element refers to the concrete building and the structural elements that are needed using moulds at a centralized facility and then transported to the building site to be assembled in the facility being constructed.

(b) The choice of the mould type and shape in precast construction depends on
   (i) Total number of elements to be produced.
   (ii) Desired rate of output.
   (iii) Shape and construction feature of the element.
   (iv) Facilities available at the production yard for casting, curing, storing, and handling.
   (v) Economic aspects.

Q2. Discuss the characteristics of the following types of moulds:
   (a) Horizontal and vertical,
   (b) Stationary and mobile system,
   (c) Brick and masonry moulds,
   (d) Wooden and steel moulds.

Q3. Discuss in detail the various precasting processes.

Q4. List out the advantages, limitations, and reasons for low share of precast concrete**.**

Q5. List out the differences in moulds used for pre casting and *in-situ* construction.

Q6. Write short notes on precasting processes associated with the following projects:
   (a) Parliament library building precast waffle slab and bubble dome
   (b) Folded plates for ITC Saharanpur
   (c) I-Girder for Nizamuddin bridge and Dwarka flyover
   (d) Dome elements for the stadium project at Bangalore
   (e) DMRC tunnel elements

Chapter

15

Pre-Award Formwork Management Issues

Contents: Introduction; Pre-award Formwork Management; Customer/client Requirement; Study of Drawings, Layouts of the Structure, and Specifications; Estimate of Cycle Time of Formwork Activities; Selection of Formwork System; Formwork Economy Considerations in Planning and Design Stage of a Project; Factors to be Considered in Planning for Form Reuse in Building Construction; Computations of Formwork Material Requirement; Cost Estimation of Formwork; Illustration of Formwork Cost Planning; Estimate of Unit Rates for Formwork Items

15.1 INTRODUCTION

The formwork management issues have been covered in two broad stages: pre-award and post-award stages in a project. In the pre-award stage, the emphasis is on formwork initial planning, value engineering, and costing. In the post-award stage, the emphasis is on detailed formwork planning, mobilization and demobilization of the material, and executing the formwork activities within the stipulated schedule, cost, and quality parameters. Besides, one has to take care of the upkeep/ maintenance, the accountability of the materials, and the training of the crew and the supervisor also. The pre-award management issues are discussed in this chapter, while the post-award management issues are discussed in Chapter 16.

15.2 PRE-AWARD FORMWORK MANAGEMENT

Similar to the other activities involved in a project, a contractor has to plan for the formwork activities and the sequence in which they are to be performed at the time of the preparation of a project bid. Failure to have a proper formwork plan can harm the contractor in both the possible ways. For example, it may result in a situation where the contractor has lost the bid while in the other situation, the contractor may be awarded the job and he finds that during implementation, he is in big trouble. The contractor may even lose money on such projects.

In some of the cases, the information needed for planning may not be available at this stage. In such cases, efforts must be made to discuss with the consultants/ designers/architects to understand physically the structures and use this approximate information for planning purposes. Non availability of information should not be an excuse for non-planning. Sometimes, the information gathered unofficially can also help in the initial planning exercise.

A schematic sketch showing the various tasks involved with the pre contract award formwork management is shown in Fig. 15.1. A brief discussion on these tasks follows in the subsequent sections.

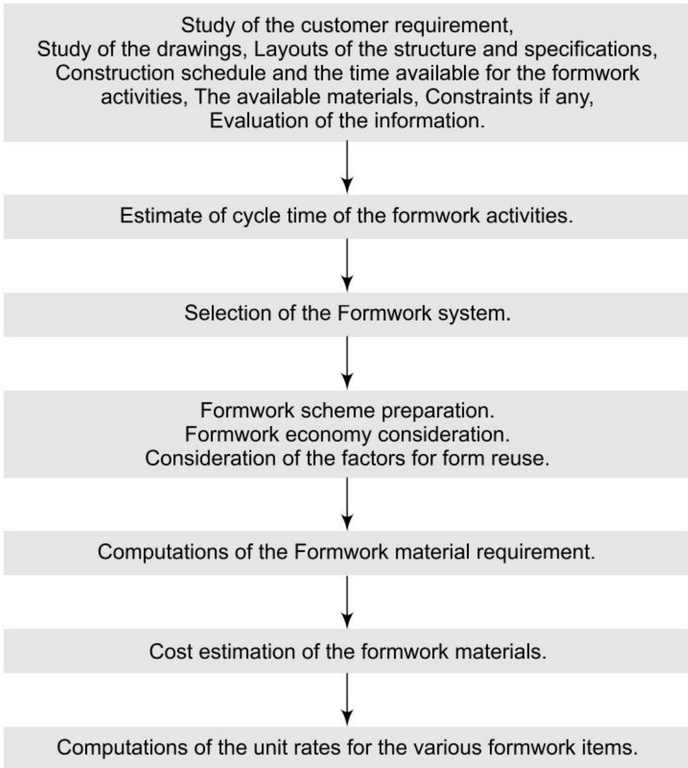


Figure 15.1 Schematic Sketch Showing the Various Tasks Involved with the Pre-contract Award Formwork Management.

15.3 CUSTOMER/CLIENT REQUIREMENT

From the available tender documents, the key requirements of the client/customer are noted down. Some typical details from a specification on formwork for a project are shown in Box 15.1.

Box 15.1: Contents of Specifications

- Reference to the code(s) for design and construction of formwork.
- Details of the submittals to be provided by the contractor, for example: working drawings, shop drawings, material specifications, and manufacturer's specifications, installation instructions for each type of formwork, accessory and plan of reshoring.

- Formwork design related information such as: the loads and the lateral pressure, the maximum permitted deflection.
- Instructions on formwork detailing and fabrication, and guidelines on the required dimensions, the plumb, and the joints.
- Instructions on supervision of the formwork activities.
- Instruction on delivery, storage, protection, and handling, of the formwork materials. Acceptance criteria of the formwork materials.
- Prior to commencement of the work, existing dimensions, elevations, locations and conditions applicable to the work shall be field verified.
- Instruction on the time allowed between erection of the forms and placing of the concrete for the various trades to properly install their work.
- Instruction on application of load on recently cast concrete.
- Specifications for lumber, sheathing material, and shoring. Instructions on fabrication of form panels in conformity with the approved submittals.
- Specifications for formwork accessories, such as form ties, hangers, clamps, lugs, cones, washers, form liners or other devices.
- Specifications for triangular fillets if applicable.
- Instructions for cleaning and maintaining the formwork materials.
- Instruction on inspections for cleanliness and accuracy of the alignment prior to the placement of the concrete.
- Instruction on location, snugness and tightness of joints.
- Instruction on level of horizontal joints and plumb of vertical joints.
- Instruction on location, size, and construction of openings.
- Specification on expansion joint material, water stops, and other embedded items to be built into the forms, and the instruction on their placement and support against displacement.
- Instruction on filling of voids in sleeves, inserts and anchor slots.
- Instruction on setting up of edge forms or bulkheads and intermediate screed strips for the slabs to obtain required elevations and contours in the finished slab surface.
- Instruction on supervision for checking deviations from desired elevation, alignment, plumbness and camber at the time of concreting.
- Guidelines for taking remedial measures in case at the time of casting, weakness develops and the formwork shows settlement, deflection or distortion.
- Specification on form coating material and instructions for their application.
- Instructions on reshoring, permissible loading on reshores, timing of placement of reshores, instruction on tightening of reshores.
- Instructions and guidelines for removal of forms, form ties, and bracings to insure complete safety to the structure.

- Instructions on removal of top forms on sloping surfaces of concrete so as to not allow the concrete to sag. Instructions on notification to be given to the engineer in cases where newly stripped surfaces require patching.
- Instruction on requirement of forms for re use. Instructions on cleaning and re-oiling of forms prior to re-use.

15.4 STUDY OF DRAWINGS, LAYOUTS OF THE STRUCTURE, AND SPECIFICATIONS

The study of the available drawings and layouts of the structure would enable the planner to explore the various formwork system options available for the proposed project. The specifications pertaining to the formwork materials and formwork system are noted. For example, in some specifications, it is prohibited to use plywood and timber as formwork materials. Some of the specifications these days categorically specify the use of steel/aluminum formwork. Similarly, in some of the specifications these days, the type of formwork system such as Doka, Mivan, etc. are categorically specified. In some of the specifications, the number of reuses that are permitted for plywood and timber are also specified. The provisions regarding the stripping time for the various formwork elements are marked. Some of the specifications reproduce the stripping time specified in IS: 456–2000 for stripping of the formwork while in some specifications, the stripping time values specified may be stringent than the IS: 456–2000 codal provisions.

The constraints of the project in terms of accessibility of the site, and capacity of the equipment to handle the materials are also studied. In order to generate a number of formwork system alternatives, the information on the following factors is also collected:

- The capacity of the concrete mixing and placing equipment.
- The specification on the construction joints (if specified). This may limit the size of the concrete lifts or the placement units.
- The details of the form construction and the facilities for fabricating or making the formwork.
- The possible reuse of the forms which is dependent on the stripping time and other requirements. The reshoring requirements given in the specification if the project consists of multi-story construction. The weather condition at the job site should also be looked into. This may have influence on the protection requirements and the stripping time, which will in turn affect the reuse and the reshoring practices.
- The type of surface on which the formwork is supported (concrete, sand, clay, wet, frozen, etc.).
- The relative advantages of job-built, shop-built and readymade forms.

Suppose at this stage, X_1, X_2, X_3, and X_4 systems of formwork have been identified, which can meet the objectives of the formwork items specified in the tender documents. The contractor would be interested in choosing the best alternative out of the four alternatives.

15.5 ESTIMATE OF CYCLE TIME OF FORMWORK ACTIVITIES

Depending on the nature of work proposed for the project (for which bid is being prepared), an estimate of the cycle time for the various formwork activities is to be made. From the construction schedule prepared for the proposed project, the time allotted for the various RCC activities is identified and a detailed breakup of the various formwork activities for the different structures of the project is prepared. This aspect has been discussed in detail in the illustrative case study later in the chapter.

15.6 SELECTION OF FORMWORK SYSTEM

The selection of the best alternative out of a set of selected alternatives is based on the cost of fabricating the formwork, the possible reuse, and the cost of fixing and removing of the formwork. The best alternative is chosen primarily on the lowest cost criterion. The estimated number and types of cranes and other equipment required, and the efficiency with which the concreting crews, reinforcing crews, and other crews will fit into the construction schedule should also be given due weight when deciding the formwork system. The comparison of the alternatives should also take into account the mobilization time of the formwork system, the preparation time, and the familiarity of the system by the contractor's crew.

The final selected system should satisfy the client's requirement i.e., the provisions laid down in the specification should be met. Besides that, the system selected should not interfere with the other trades of the project and thereby upsetting the entire project schedule.

15.6.1 Guidelines for Selection of Formwork System

For pre-award planning purposes, Table 15.1 can be referred to. For example if the project mostly consists of multi-story buildings, then a system with steel plates or plywood supported on Collapsible Tube Props can be utilized. Further, the distinction between a steel, aluminum, and plywood sheathing is again governed by the number of repetitions envisaged for the project. If the repetition is of the order of 100-200 it is recommended to use either steel or aluminum. However, when the repetition is of the order of 10-12, plywood/timber should be preferred. The construction schedule to quite an extent, decides the number of reuses possible within the project. Some of the materials can be used in a number of projects and in such cases, the market potential and the versatility of the formwork system are to be taken into account.

Table 15.1 Recommended Systems to be Followed for Different Structures in a Project

Sl No.	Structure involved in the project	Recommended systems
1.	Multi-story building with story height less than about 4 m	Prop system with steel plates or plywood, depending on repetition.
2.	Factory, Shed with floor height more than 4 m	Tower system of staging with steel plates or ply, depending on repetition.
3.	Tall structures like Silo, Chimney	Climbing formwork or slipforming.
4.	Shell roof	Tower with ply/timber.
5.	Bridge or Dam	Study different possibilities and choose.

15.7 FORMWORK ECONOMY CONSIDERATIONS IN PLANNING AND DESIGN STAGE OF A PROJECT

Formwork is a costly item and it has significant influence on the final bid price that the contractor submits for a bid. Also formwork is one of those activities which offers the maximum opportunity of applying engineering and managerial skills and thereby arriving at a competitive cost. It is also known that the planning and tendering stage offers the maximum opportunity of saving in cost especially in the formwork items. A contractor can bring down his cost by adopting and implementing many practical and achievable things. The reduction in cost is more in case when the contractor is entrusted with both the design and construction responsibility. In the following sections, we discuss some of the ways in which the cost can be brought down in this stage.

15.7.1 Simple Layout, Uniform Bay Sizes

It is recommended that the layout of the building is planned in such a way that it looks organized dimension wise. It should reflect simplicity and modular coordination of standardized elements, including duplication of parts and methods of linkage. Some of the ways in which this can be achieved are having provisions of uniform bay sizes, symmetry, and alignment of columns. All these ensure a means of control of the accuracy, tolerances and fit. In cases where a regular shape is not possible the concept of simple unit addition should be relied on to develop complicated configurations of the building.

The suggestions given above would ensure the possibility of a large number of repetition of formworks which would allow less assembly (since jointing can be avoided), less waste and less execution time which would ultimately result into a smooth flow of construction and cost saving.

Naturally, the designer must not allow this repetition of elements and processes to take over and result in visual monotony and boredom. The components must be combined in such a way as to yield an optimum number of formal variations.

15.7.2 Choosing Suitable Materials

Materials for making forms are chosen in such a way as to provide for the economy of the formwork. Some of the criteria for choosing the right materials are:

- Materials should be locally available.
- It should be a low-cost material not only in terms of the investment cost but in terms of the recurring cost like maintenance and repair cost also.
- The selected materials should have a long service life.

15.7.3 Choosing Proper Sizes of Members

If member sizes are chosen at the design stage itself according to the available formwork materials, it results into a considerable economy in formwork. This aspect is very important because, if the designer provides member sizes that demand very expensive or infeasible types of formwork, it would unnecessarily be increasing the formwork cost.

15.7.4 Enhancing the Number of Repetitions of Members

If the number of repetitions of the members is large, then the formwork can be used repetitively, to achieve considerable economy. The following points may serve as guidelines:

- Avoid unnecessary introduction of many sizes of members.
- Avoid unnecessary reduction in sizes of members as we go on to the top floors of the buildings.
- Use modular types of forms.

15.7.5 Reducing the Size of Members

By reducing the sizes of the members to be cast, we can reduce the cost of the formwork because of a reduction in the effective surface area of the form. So reduction in the sizes of the members can help in achieving formwork economy.

This can be done by the following ways:

- Pre-stressing;
- Increasing the grade of concrete being poured;
- Increasing the reinforcement;
- Improving the construction practices on-site.

15.7.6 Matching the Edges of Beams and Columns

The edges of the beams and columns in any beam-column junction, should preferably match for economic provision of the formwork at that particular point. Otherwise, if the size of the beam exceeds that of the column or vice versa, it would pose additional cost of formwork at that junction.

15.7.7 Planning for Maximum Reuse

One of the important ways to achieve economy in formwork is to plan for maximum reuse. The provisions of stripping and reshoring play an important role in planning for maximum reuse. For example, it would be economical to build fewer vertical forms of beam since the stripping time for vertical formwork of beams (16-24 hours as per IS: 456–2000) are much shorter than the soffit formwork of beams (7 days as per IS: 456–2000). This would help attain a large usage of the vertical formwork.

Similarly, in case of column formwork, a large number of reuses is possible if the building design has an identical column cross section or has limited dimensional changes. A large number of reuse is possible if a slipform is chosen for structures such as chimneys, silos, towers, and lift shafts. The use of modular formwork can also yield large form reuses.

The cost of formwork very much depends on the quantum of mobilization initially done. Hence, it is very essential that it is estimated correctly with the cost projections. The requirement of materials should be indicated along with a schedule. In addition to planning for the material requirement, the contractor should also plan the labor requirement, category wise, correctly.

15.8 FACTORS TO BE CONSIDERED IN PLANNING FOR FORM REUSE IN BUILDING CONSTRUCTION

The importance of reuse of the forms has been emphasized at several places in this text. In order to be economical, it is essential to reuse the forms as many times as possible without compromising on the overall project schedule.

Formwork is a costly resource and the initial investment required to procure the formwork is substantial. Thus, reuse becomes all the more important. When procuring the formwork, the reuse could be planned either for a single project or in several projects. More reuses are possible if similar building form units can be repeated with the same set of formwork.

Some of the factors to be considered essential in planning for form reuse in building construction are: division of regions, allocation of tower cranes, number of form sets prepared, number of labor crews, etc. These are described briefly in the following sections:

15.8.1 Division of Regions for Form Reuse

In formwork literature (see Huang *et al*. 2004), the total area which is covered by full sets of modular formwork is defined as a region. Each region is assumed to have the same number of building units such as walls, columns, beams, and slabs.

The concept of a region and building units is explained with the help of a diagram in the context of a multi-story building. In a multi-story building, there could be a single or several regions in a building floor. As defined earlier, each of the regions are symmetrical in terms of the number and sizes of the walls, columns, beams, and slabs. The reuse of form materials is dependent on the number of regions in a floor, the number of floors, and the number of such buildings in a project and several other projects.

For example, if we see Fig. 15.2, there is only one building in the project and there is only one region in each floor. There are n floors in the building. Thus, reuse of the formwork materials can be simply expressed as given below:

$$\textit{Number of reuses in the project} = \textit{number of floors} \times \textit{number of regions} = n \times 1 \tag{15.1}$$

If the cycle time to complete the formwork operations (right from fixing the formwork to dismantling and making it ready for next use) is d days per region, the total duration for completing the formwork operation for the building is given by the following expression.

$$\begin{aligned} \textit{Total duration} &= \textit{cycle time for one region} \times \textit{number of regions in a floor} \times \textit{number of floors} \\ &= d \times 1 \times n \end{aligned} \tag{15.2}$$

In case of Fig. 15.3, there is one building of n floors with two regions in each floor. Thus,

$$\textit{Number of reuses in the project} = \textit{number of floors} \times \textit{number of regions} = n \times 2 \tag{15.3}$$

and

$$\begin{aligned} \textit{Total duration} &= \textit{cycle time for one region} \times \textit{number of regions in a floor} \times \textit{number of floors} \\ &= d \times 2 \times n \end{aligned} \tag{15.4}$$

It is clear from the above discussion and the expressions of number of reuses and duration for the various cases, that more regions in a building floor would result in more reuses, but it would also mean more interfaces between the regions and thus increase in durations for completing the formwork operations.

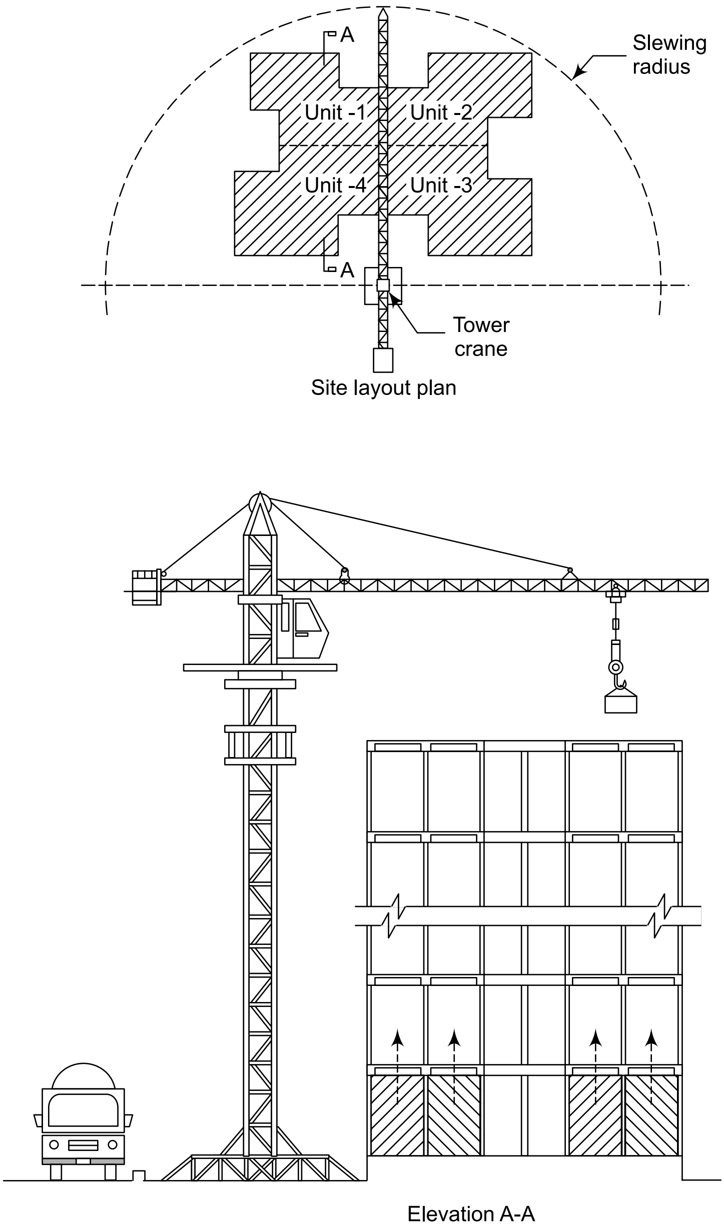


Figure 15.2 Single Building of *n* Floors and One Region.

15.8.2 Allocation of Tower Cranes

Cranes are proving to be very important resources in modern construction. They are used for a variety of functions. In the context of concrete construction, they are used for moving the steel, moving the formwork from one location to the other, for installing and removing the formwork, and for pouring of concrete. Depending on the nature of the work, both tower cranes as well as mobile cranes can be employed.

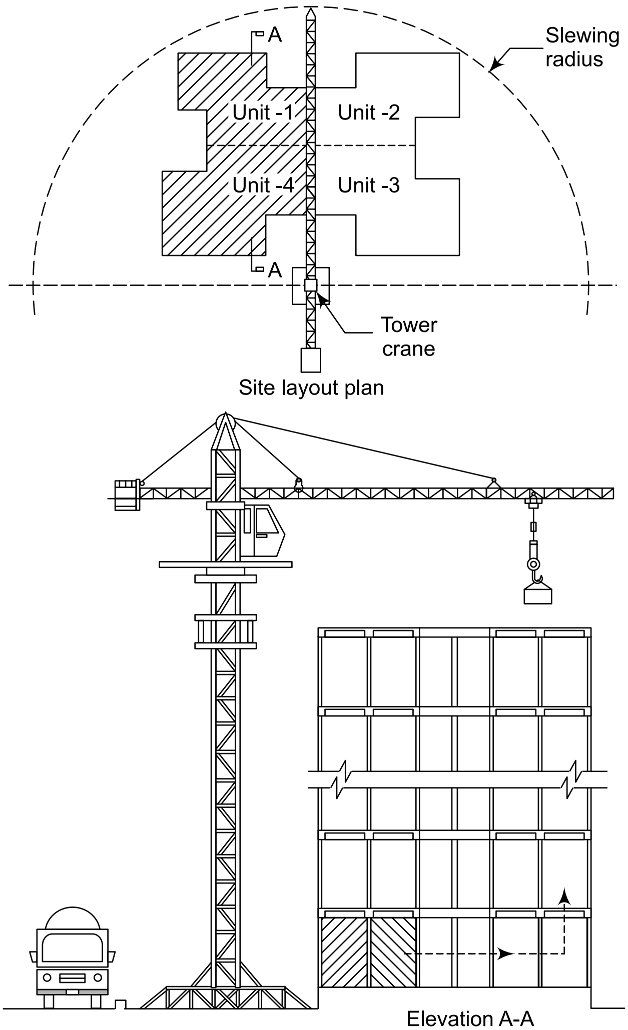


Figure 15.3 Single Building of *n* Floors and Two Regions.

The use of a tower crane has become synonymous with medium- to high-rise construction, be it for buildings or for bridges, dams, cooling towers, etc. Tower cranes can be fixed or can be mounted on rails. In the latter case, it is referred to as a travelling tower crane. Sites with multiple tower cranes in operation are very common these days, especially for fast-track construction projects involving flying forms and gang forms.

Depending on the nature of the work, the following assignment of tower cranes could be adopted at a given site:

(a) A single tower crane being assigned to a single region as shown in Fig. 15.2.
(b) A single tower crane being assigned to multiple regions as shown in Fig. 15.3.
(c) Multiple tower cranes being assigned to a single region as shown in Fig. 15.4.
(d) Multiple tower cranes being assigned to multiple regions as shown in Fig. 15.5.

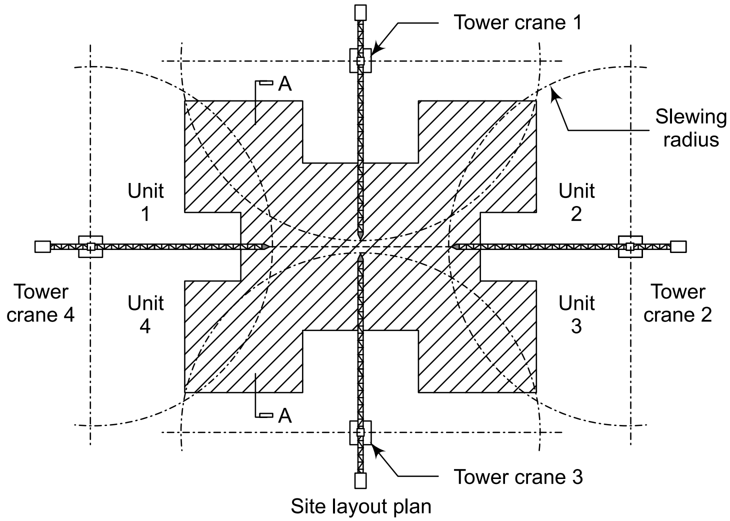


Figure 15.4 Multiple Tower Cranes Being Assigned to Single Region.

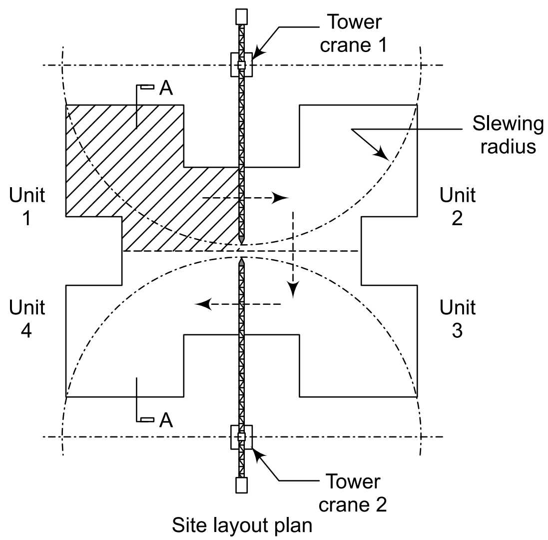


Figure 15.5 Multiple Tower Cranes Being Assigned to Multiple Regions.

15.8.3 Number of Form Sets Prepared

Number of form sets required to be mobilized for a particular building element is dependent on a number of factors such as the area to be covered within the available time, cycle time for the formwork operations, and the number of working days in a week. In a conventional formwork system, the number of sets is defined as the total number of forms required to cover one floor of a building. In the case of modular formwork, though there is no universally accepted definition, a reasonable definition could be achieved by replacing the floor with a region. Thus, the number of sets here is the total number of form sets mobilized to cover one region. Now, depending on the movement of the form, the number of sets can be decided. For example, the number of sets required would be less if the forms are to be moved to different regions on the same floor whereas the number of sets required would be more if the forms are not moved within the region in a given floor, i.e. only vertical movement of form sets from the lower floors to the upper floors. If there are dedicated sets for a number of regions in a given floor, work in number of regions can be executed concurrently provided there are sufficient crews and cranes.

15.8.4 Number of Labor Crews

The labor crews are required for such tasks as form and steel erection, the concrete pouring, etc. The number of labor crews available definitely influences the selection of the form reuse scheme. Increasing the number of crews can improve the productivity of the operation, but only if the building floors are appropriately divided and adequate numbers of cranes and form sets are properly allocated.

15.9 COMPUTATIONS OF FORMWORK MATERIAL REQUIREMENT

The formwork material planning is a systematic process and is carried out in the following steps:

Step 1

From the construction schedule prepared at the time of tendering, the duration of activities involving RCC is found out. Normally, the duration of formwork activities is taken as the duration for RCC activities plus the mobilization and demobilization periods. The mobilization and demobilization periods can be assumed as one month each for a typical project. Large projects may have slightly bigger mobilization and demobilization periods, and may require to be mobilized and demobilized in a phased manner.

Step 2

The distribution of different formwork items can be obtained from the bill of quantities if it is available. In case the bill of quantities is not available, the estimate of the different formwork quantities may be based on some assumptions. A typical breakup which may be assumed for a typical building project is given in Table 15.2.

Table 15.2 Breakup of Different Formwork Items for a Typical Building Project (for Planning Purpose)

Sl. No.	Type of formwork required	The percentage of total formwork area
1.	Foundation	5%
2.	Column	10%
3.	Wall	15%
4.	Slab and beam	65%
5.	Domes/ Precast element if any	5%

In case the total formwork area is also not known, the same can be estimated based on the estimates of the concrete quantity. For example, if the concrete wall quantity is known, the formwork area can be assumed based on a typical wall dimension in a building project. Similarly the column formwork area can be estimated from the total column concrete quantity by assuming a typical column dimension for a building project.

Step 3

The month-wise formwork area to be executed is estimated by assuming either an uniform distribution of formwork quantities of different types or by assuming a particular distribution as felt appropriate. The template for estimating the formwork area on a monthly basis for the different formwork items is shown in Table 15.3. It may be noted that the classification of the formwork items is broadly based on the formwork items specified in Delhi Schedule of Rates. Some of the items suggested in Delhi Schedule of Rates have been left out as they constitute a small percentage of the overall formwork area and it may not be advantageous to go into that much detail at the time of tendering.

Table 15.3 The Month-wise Break up of Formwork Items for the Project

Sl. No.	Type of elements	Total quantity (m^2)	Month-wise split up of quantity of formwork planned to be executed (m^2)				
			1	2	3	4	5
1.	Footings/Raft/Pile caps	4,796	4,796				
2.	Walls	44,667	8,933.4	8,933.4	8,933.4	8,933.4	8,933.4
3.	Columns	11,352	2,270.4	2,270.4	2,270.4	2,270.4	2,270.4
4.	Beams and Slabs with floor height up to 4.5 m	41,479	8,295.8	8,295.8	8,295.8	8,295.8	8,295.8
5.	Beams and Slabs with floor height more than 4.5 m	16,328	3,265.6	3,265.6	3,265.6	3,265.6	3,265.6
6.	Staircase	2,706	541.2	541.2	541.2	541.2	541.2

Step 4

After the estimate of the formwork area to be executed on a monthly basis is prepared, an estimate of the cycle time for the different formwork activities is made. For this, the data from past projects can be utilized after suitable adjustment depending on the nature of the project for which the bid is being prepared.

A typical estimate of the possible usage of the formwork materials for different formwork activities and the corresponding cycle time for the formwork is given in Table 15.4 for planning purpose.

Table 15.4 Typical Possible Usage and Cycle Time for Various Formwork Activities

Type of formwork elements	Possible usage	Cycle time (days)
Footings/Raft	8 uses / month	3
Walls	5 uses/ month	5
Columns	8 uses / month	3
Beams and Slabs	2 uses/month	15

Step 5

The formwork area to be catered or mobilized for various formwork items can be estimated based on the total area to be executed in a month, cycle time, and the number of working days in a month. For example, let's compute the area of the wall shutter that is to be mobilized if it is desired to execute 1000 m^2 in a month. Assume the cycle time for wall formwork to be 5 days and the number of working days in a month to be 25 (assuming five Sundays).

$$\textit{The wall formwork area to be mobilized} = \frac{\textit{Total formwork area planned for the month} \times \textit{Cycle time}}{\textit{Number of working days in a month}} \tag{15.5}$$

Hence,

$$\text{The wall formwork area to be mobilized } = \frac{1{,}000 \times 5}{25} = 200 \text{ m}^2$$

The area to be mobilized for different formwork items can be computed in the above manner on a monthly basis. It can be observed that the reduction in cycle time can have a great effect on the requirement of the formwork materials. For example, if the wall formwork cycle time is reduced from 5 to 4 days and 3 days, the material requirement would come down from 200 m^2 to 160 m^2 and 120 m^2, respectively, which is a saving of 20% and 40% in the materials requirement respectively.

Step 6

The above step has yielded the overall area to be mobilized for a given formwork type on a monthly basis. This in itself is not sufficient. One would also require the breakup of the major material types for accurate costing. For example, how much is the requirement of plywood, timber, and steel for the total project? Similarly, what is the requirement of scaffolding material and so on?

It may be emphasized here that during the tendering process, the time available for performing detailed planning and computations is not sufficient. Thus, for estimating the cost, most of the contractors rely on estimating the materials requirement (from step 5) on certain thumb rules developed from past projects. These thumb rules are based on certain assumed formwork schemes.

The summary of some such thumb rules for some typical formwork schemes using L&T formwork system are given in Table 15.5. Similar tables can be prepared using other formwork systems also.

Table 15.5 Some Thumb Rules Useful for Computing Formwork Materials Requirement

Item Description	Steel kg/m^2	Timber m^3/m^2	H-16 Rm/m^2
Footings/Raft/Pile caps	20	0.085	N.A.*
Walls	60	0.006	3.5
Columns	100	0.006	7.0
Beams and Slabs with floor height up to 4.5 m	25	0.028	4.0
Beams and Slabs with floor height 6 m	75	0.028	3.5
Access scaffolding (Area in elevation) Assembly of Heavy Duty Tower	16		

* Normally use of H-16 beam in foundation work is avoided.

The use of Table 15.5 has been illustrated for the wall formwork example explained previously. It may be recalled that the wall formwork area to be mobilized was 200 m^2 for a cycle time of 5 days and 25 working days in a month. The requirement of steel, timber, and H-16 beam (for L&T wall formwork system) from Table 15.5 is 60 kg/m^2, 0.006 m^3/m^2 and 3.5 m/m^2, respectively.

Thus the requirement of steel for 200 m^2 would be $60 \times 200 = 12{,}000$ kg

The requirement of timber for 200 m^2 would be $0.006 \times 200 = 1.2\ m^3$

The requirement of H-16 for 200 m^2 would be $3.5 \times 200 = 700$ m

In the above manner, for the different types of formwork items, the material requirement can be estimated and the requirement of similar items can be combined.

Case 1 Planning Exercise

The pre-award formwork planning exercise has been illustrated with respect to a real project data. The project is planned in two phases: 1A and 1B. During the planning exercise, it is imperative to note down the major milestones to be achieved in each of the phases. The overall construction schedule for the project is referred to. This is shown in Figure 15.6. The milestones and their estimated schedule based on the overall construction schedules in phase 1A and 1B are shown in Tables 15.6 and 15.7, respectively.

Table 15.6 Milestones in Phase 1A

Milestones Description	Period (Days) from date of LOA
For Buildings MLCP, SDB 1, CANTEEN	
Starting of MLCP Structure	92 Days
Completion of MLCP Structure	316 Days
Starting of SDB1 Structure	92 Days
Completion of SDB1 Structure	303 Days
Starting of CANTEEN Structure	92 Days
Completion of CANTEEN Structure	273 Days

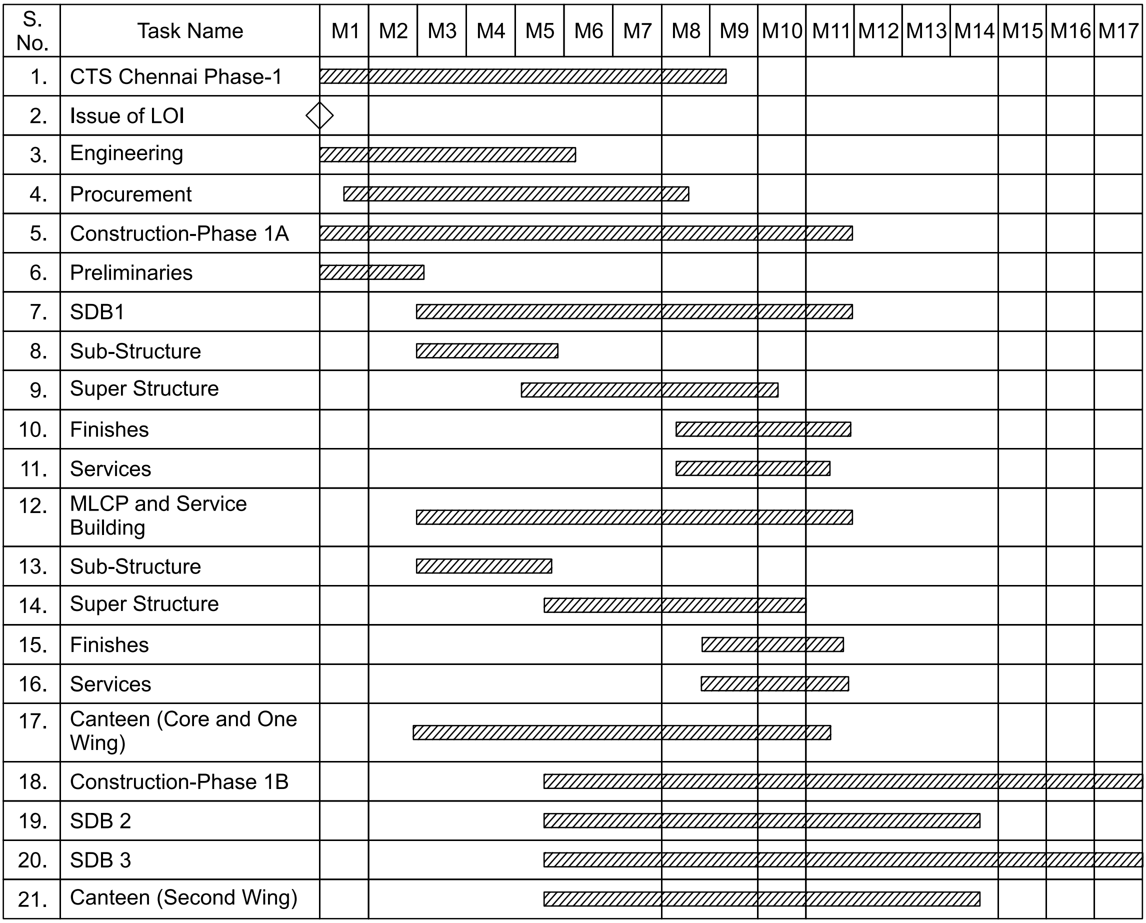


Figure 15.6 Construction Schedule for the Project.

Table 15.7 Milestones in Phase 1B

Milestones Description	Period (Days) from date of LOA
For Buildings SDB2 and SDB3	
Starting of SDB 2 Structure	162 Days
Completion of SDB 2 Structure	377 Days
Starting of SDB 3 Structure	162 Days
Completion of SDB3 Structure	469 Days

The relevant data for preparing the formwork plan is culled out from the specification, and the available drawings for the project. The data so obtained are tabulated in Table 15.8.

Table 15.8 Project Data

Sl. No.	Description	Remarks
1.	Plan size of the building	95.3 m × 35.3 m
2.	Overall scope of work	3,86,345 m^2
3.	Duration of the project	518 days
4.	Types of columns	Rectangular/square/plus-shaped
5.	Types of slabs	RCC beam and slab
6.	Catering area	18,100 m^2

Area to be catered (CA) in m^2 is calculated for every element like columns, walls, slabs and beams, etc. separately from Eq. (15.6).

$$CA = \frac{SA \times C_1}{R} \tag{15.6}$$

where,

SA = Shuttering area to be executed in a month

$= \dfrac{\text{Total shuttering area } (TSA)}{\text{Duration in months } (M)}$

C_1 = 1.10 normally 10% factor is applied for spare resource to be mobilized to meet any additional requirements or replace damaged parts of formwork elements during the work progress.

R = Number of repetitions in a month = working days in a month/cycle time (CT)

$= \dfrac{26}{CT}$

where, CT = Cycle time in days

$$\text{Thus } CA \text{ for foundation} = \frac{4{,}796}{1} \times 1.1 = 609 \text{ m}^2 \text{ (see Table 15.9)}$$

$$CA \text{ for columns} = \frac{(11{,}352/5) \times 1.1}{26/3} = 288 \text{ m}^2 \text{ (see Table 15.9)}$$

The workings for both the phases are shown in Table 15.9 and Table 15.10, respectively. It may be noticed that the formwork material is dependent on the area to be catered in each category (Foundation, Columns, RC walls, Flat slab, Beams and slab, Staircase) which in turn are dependent on the formwork system proposed for the work, the duration in which the work is to be completed, and the cycle time. Needless to say, all these will have a bearing on the costing issues of the formwork.

Table 15.9 Formwork Workings –CTS –SDB1 and Canteen-Phase 1A

Sl.No	Description	As Per BOQ	Formwork System	Unit	Duration in Months	Cycle Time in days	Area to be Catered
1	2	3	4	5	6	7	8
1.	Foundation	4,796	**L&T-Floor forms**	m^2	1	3	609
2.	Columns	11,352	**L&T-Column FW**	m^2	5	3	288
3.	RC walls	44,667	**L&T-Wall FW**	m^2	5	7	2,646
4.	Flat slab	41,479	**L&T-Flex Table**	m^2	5	10	3,510

Sl.No	Description	As Per BOQ	Formwork System	Unit	Duration in Months	Cycle Time in days	Area to be Catered
1	2	3	4	5	6	7	8
5.	Conventional beams and slab	16,328	**L&T-BFS,HDT**	m^2	5	14	1,934
6.	Staircase	2,706	**L&T Flex**	m^2	5	14	321
	Summary	**1,21,328**		m^2			**9,308**

Table 15.10 Formwork Workings –CTS –SDB 2 & SDB 3-Phase 1B

Sl.No	Description	AS Per BOQ	Formwork System	Unit	Duration in Months	Cycle Time in days	Area to be Catered
1	2	3	4	5	6	7	8
1	Foundation	4,164	**L&T-Floor forms**	m^2	1	3	529
2	Columns	15,031	**L&T-Column FW**	m^2	6	3	318
3	RC walls	33,564	**L&T-Wall FW**	m^2	6	7	1,657
4	Flat slab	68,545	**L&T-Flex Table**	m^2	6	10	4,833
5	Conventional beams and slab	32,049	**L&T-BFS,HDT**	m^2	6	14	3,164
6	Staircase	5,099	**L&T Flex**	m^2	6	14	503
	Summary	**1,58,452**		m^2			**11,004**

15.10 COST ESTIMATION OF FORMWORK

The cost estimation of the formwork like any other estimation, involves estimating the cost of labor, material, plant and equipment, and the associated overhead. However, the estimation of the materials cost is little different in case of the formwork as we shall see subsequently. The overall cost of the formwork can be written as the sum of the direct and indirect cost of the formwork. The breakup of the direct cost for the formwork can be visualized as shown in Fig. 15.7.

15.10.1 Material Cost

The materials for the formwork items include the cost of the direct materials (such as plywood and steel) as well as the cost of the consumables such as nails, shuttering oil or release agents, binding wire, etc. The allocation of the cost of direct materials to a particular item or project depends on the life, scrap value of the material at the end of the project, and can be calculated in mainly two ways. In the first case,

$$\textit{Cost of materials per use} = \frac{\textit{Cost of materials} - \textit{Scrap value}}{\textit{Number of repetitions}} \qquad (15.7)$$

In the second case, in order to calculate the cost of materials for the item/project, we calculate it as:

$$\textit{Cost of materials per use} = \frac{(\textit{Cost of material} - \textit{Scrap value}) \times \textit{Period of use for item per project}}{\textit{Life of materials}} \qquad (15.8)$$

While the first case is used for calculating the costs of items like timber and plywood, the second case is used for items like steel, metal components, aluminum, etc.

It is to be recalled here that the requirement of formwork materials depends on the total area of shuttering; the total value of staging for the entire project; the period of completion and the construction program; the striking time for the various parts of the structures; the strength requirements of the permanent structures; and the type of the structure.

There is a cost involved towards storing, watch and ward, of the formwork materials. However these are considered collectively in the overall project indirect cost and a percentage is added in the formwork items based on pro rata basis.

Consumables

Some of the consumables used in the formwork activities are form coating agents, nails, binding wires, inserts, lost ties, and hardware, etc. While the cost of form coatings, nails, and binding wire may not be appreciable in comparison to the material and labor cost, the cost of consumable such as insert, lost ties, and other lost hardware could be substantial. Some inserts and hardware are lost after each use. Thus, if the formwork schemes anticipate application of inserts, lost ties, and other hardware in the previously placed concrete to anchor the formwork, for the next lift, the cost of these consumables should be properly accounted for.

Illustration of material cost

1. For timber
2. For steel
3. For plywood costing, we consider the following three cases:

Case 1

- Area to be catered 1,000 m^2.
- Area mobilized for 40 m^2 based on cycle time and total duration available.
- Total repetition possible = $\frac{1{,}000}{40} = 25$.
- Since plywood cannot last for this many repetitions, take cost of plywood per m^2.

  $= \frac{1.05 \times \text{plywood cost}}{12}$, where 5% is the wastage and 12 is the repetition.

Case 2

- Area to be catered 1,000 m^2.
- Area mobilized for 200 m^2 based on cycle time and total duration available.
- Total repetition possible = $\frac{1{,}000}{200} = 5$.
- In case we are not able to make use of this plywood at the project site, take cost of plywood per $m^2 = \frac{1.05 \times \text{plywood cost}}{5}$, where 5% is the wastage and 5 is the repetition.

Case 3

- Area to be catered 1,000 m^2.
- Area mobilized for 200 m^2 based on cycle time and total duration available.
- Total repetition possible = $\dfrac{1{,}000}{200} = 5$.
- In case this plywood is further used for some other application at the project, take cost of plywood per m^2 as = $\dfrac{1.05 \times \text{plywood cost}}{12}$, where 5% is the wastage and 12 is the total repetition.

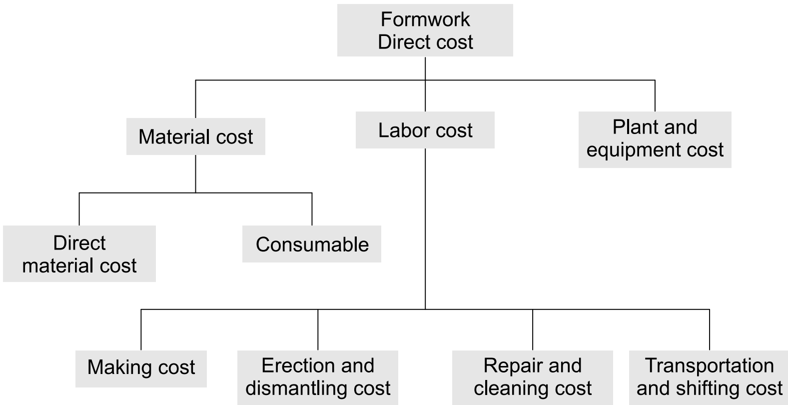


Figure 15.7 Details of Formwork Direct Cost.

15.10.2 Labor Cost

The labor cost is the sum of the labor component of the transporting cost of the formwork materials from the stores to the fabrication yard, the cost for making the formwork, the shifting of the shutters to the work site location, the cost of erection and dismantling the formwork. Apart from this, labor costs are incurred in cleaning and minor repairing of the formwork after each use of the shutters besides major repair after few uses and the engagement of carpentry labor during concreting operation.

15.10.2.1 Labor for making formwork

This is a cost incurred while making the formwork components at the site. The timber components are usually made at the site. The labor for cutting, sizing, and making of the panel and other components are included under this item. The allocation of this cost per use/item/project can be done in the same manner as it is adopted for direct materials. In the same way, the labor spent on the fabrication of steel items can also be allocated.

15.10.2.2 Labor for erection and dismantling (fixing and removing)

This includes the cost of skilled and unskilled labor used for the erection and assembly of various components of the formwork in their place of line, level and alignment. The cost of labor force to be maintained during the concreting for minor adjustments of formwork and contingency will also have to be considered under this head. Also, the labor employed for dismantling of the components is to be included.

The estimate of labor cost for erecting and dismantling of the various formwork schemes may be based on man-hours per m^2 based on the past experience with similar type of schemes. For a detailed analysis of labor cost for erection and dismantling, the crew size for the different operations involved with erection and dismantling activities and the time consumed need to be considered. The detailed analysis would indeed be giving the accurate estimate.

15.10.2.3 Cost of repair of formwork

After a few repetitions, the formwork may need some minor repairs. The total cost of repairs at the various stages of form use during a project can be distributed evenly over the entire shuttering area in the project. This will include material, labor and plant used for the purpose of repair.

15.10.2.4 Transporting and shifting

In large project sites, the formwork materials may have to be transported or shifted from one place to another. Both the cost of labor and equipments utilized for this purpose can be distributed evenly over the total shuttering area.

15.10.2.5 Cleaning and storage

The formwork materials, after their use at a particular location, have to be cleaned and stored before their use in another location within the same project. Final cleaning and storage may also be necessary at the end before transportation to another site and this cost should also be considered and evenly distributed over the entire shuttering area in the project.

Similar to the material estimate, the contractors rely on estimating the labor requirement on certain thumb rules developed from past projects. These thumb rules are based on observations of the time spent on the different formwork operations associated with a formwork activity. The summary of some such thumb rules for some typical formwork activities are given in Table 15.11.

Table 15.11 Some Thumb Rules Useful for Computing Formwork Labor Requirement

Item Description	Man-hour/m^2 (for labor)
Footings/Raft/Pile caps	0.9
Walls	1.7
Columns	1.8
Beams and Slab with floor height up to 4.5 m	1.5 (Slab and Beam)
	0.8 (Flat slab)
Beams and Slab with floor height 6 m	2 (Slab and Beam)
Access scaffolding (Area in elevation) Assembly of HDT	0.2

The use of Table 15.11 has been illustrated for the wall formwork example explained previously. It may be recalled that the wall formwork area to be executed for a typical month was 1,000 m^2. The requirement of labor would be $1{,}000 \times 1.7 = 1{,}700$ man-hours = 170 man-days assuming 10 working hours per day. Thus, the wall formwork alone would require 170/25 = 6.8, say, 7 crew members. Similarly, the labor requirement for the different formwork activities can be computed on a monthly basis. The detailed breakup of labor into carpenter and helper may not be required at the time of tendering.

15.10.3 Plant and Equipment Cost

Plant and equipment are needed for making, erecting, dismantling, repair, and transport of the formwork. The plant and equipment could be either major plants or small tools and tackles. One typical example for major equipment is the tower crane which may be required for shifting and erecting the formwork. In the case of major plants, hire charges or depreciation along with operating costs are included as plant costs. For small tools and tackles, the total cost is usually absorbed in the cost of the project. The plant cost incurred for making is accounted similar to that of labor used for making of the formwork. The cost of equipments like tower cranes is computed proportional to the time of utilization of these equipments for formwork.

15.11 ILLUSTRATION OF FORMWORK COST PLANNING

Table 15.12 shows the summary of results obtained as a result of the formwork cost planning exercise for the case study. For different types of elements, the total formwork area is shown in column 2 of this table. The area to be catered for each of these formwork elements is shown in column 3. The method to compute the area to be catered has already been explained earlier. The number of possible reuses of the form has been assumed and is shown in column 4. The requirement of steel components (in kg), the purchase requirement of timber (in m^3), the requirement for plywood (in m^2), and the labor (work hours) are shown in columns 5, 6, 7, and 8, respectively, of Table 15.12. The computation process for entries in columns 5, 6, 7, and 8 has already been explained earlier.

The data shown in Table 15.12 is helpful for understanding the kind of investment the contractor has to make in formwork related activities. For example, the contractor now understands that he/she has to mobilise 4,40,565 kg (about 441 t) of steel components of formwork, purchase 241.19 m^3 of timber, and 12,146 m^2 of plywood, besides engaging 2,02,323.4 work hours (20,232.34 man-days with 10 hours considered as working time every day) of formwork crew. From this information, the major cost involved in formwork activities can be estimated.

15.12 ESTIMATE OF UNIT RATES FOR FORMWORK ITEMS

In the following section, the estimates of some common formwork items are given. It can be noted that the estimate is dependent on the system of formwork adopted for a particular application. The general steps in preparing the estimate of unit rates for formwork items are explained below. A generic template for estimation is also suggested.

The unit rate is arrived at by adding the material cost, labor cost, plant and equipment cost, overhead, and profit margin. These are explained below:

15.12.1 Material Cost

The materials in formwork consist of plywood, timber, H-16 beam, steel, and consumables. The cost of usage of each of these materials is explained below:

Table 15.12 Formwork Cost Planning

Type of elements	Total area (m^2)	Area to be catered (m^2)	No. of possible reuse of form sets	Requirement of steel components (kg)	Timber purchase (m^3)	Plywood purchase (m^2)	Labor (work hours)
1	2	3	4	5	6	7	8
Footings/ Raft/Pile caps	4,796	609	8	609 × 20 = 12,180	609 × 0.085 = 51.77	609 × 1.1 = 669.9	4,796 × 0.9 = 4,316.4
Walls	44,667	2,646	17	2,646 × 60 = 1,58,760	44,667 × 0.006/12 = 22.33	44,667 × 1.1/12 = 4,094	44,667 × 1.7 = 75,933.9
Columns	11,352	288	39	288 × 100 = 28,800	11,352 × 0.006/12 = 5.67	11,352 × 1.1/12 = 1,040.6	11,352 × 1.8 = 20,433.6
Beams and Slab with floor height up to 4.5 m	41,479	3,510	12	3,510 × 25 = 87,750	3,510 × 0.028 = 98.28	3,510 × 1.1 = 3,861	41,479 × 1.5 = 62,218.5
Beams and Slab with floor height of 6 m	16,328	1,934	8	1,934 × 75 = 1,45,050	1,934 × 0.028 = 54.15	1,934 × 1.1 = 2,127.4	16,328 × 2 = 32,656
Staircase	2,706	321	8	321 × 25 = 8,025	321 × 0.028 = 8.99	321 × 1.1 = 353.1	2,706 × 2.5 = 6,765
Total				4,40,565	241.19	12,146	2,02,323.4

A. Cost of usage of Plywood

The expression for computing the cost of usage of plywood is given below:

$$\textit{Cost of using plywood} = \frac{\begin{array}{c}\textit{Consumption of plywood in square meter for } 1\,m^2 \\ \times \textit{ Purchase cost of plywood for } 1\,m^2\end{array}}{\textit{Maximum number of uses envisaged}} \tag{15.9}$$

In case of wall formwork, let's assume that 1.05 m^2 of plywood is required including 5% wastage and the plywood is purchased at a rate of Rs. 500 per m^2. If the plywood is expected to last for 12 uses, then the cost of usage of the plywood is computed as below:

$$\text{Cost of using plywood} = \frac{1.05 \times 500}{12} = \text{Rs.}\frac{43.75}{\text{m}^2}$$

B. Cost of usage of Timber

The expression for computing the cost of usage of timber is given below:

$$\textit{Cost of using timber} = \frac{\begin{array}{c}\textit{Consumption of timber in cubic meter for } 1\,m^2 \\ \times \textit{ Purchase cost of timber for 1 cubic feet}\end{array}}{\textit{Maximum number of uses envisaged}} \tag{15.10}$$

For example, if the timber is purchased at a rate of Rs. 7,000 per cubic meter and the consumption of the timber in 1 m^2 of wall formwork is 0.006 cubic meter and it is expected that the timber would last for 12 uses, then the cost of usage of the timber is computed as below:

$$\text{Cost of using timber} = \frac{0.006 \times 7{,}000}{12} = \text{Rs.}\,\frac{3.50}{\text{m}^2}$$

C. Cost for usage of H-16 beams

$$\textit{Cost of using H-16 beam} = \frac{\begin{array}{c}\textit{Consumption of H-16 beams in running meters for} \\ 1\,m^2 \times \textit{Investment cost of H-16 beams for 1 m} \\ \times \textit{ Rate of depreciation per month}\end{array}}{\textit{Number of uses in a month}} \tag{15.11}$$

Suppose for a wall formwork the requirement of H-16 beams is 4 m for 1 m^2 of wall formwork and the cost of the H-16 beams is Rs. 300 / m. If the rate of depreciation is 3% per month and the H-16 beam is expected to be reused 5 times in a month, then the cost for usage of the H-16 beam is given as:

$$\text{Cost of using H-16 beams} = \frac{4 \times 300 \times 3\%}{5} = \text{Rs.}\frac{7.2}{\text{m}^2}$$

D. Cost for usage of steel items

$$\textit{Cost of steel components} = \frac{\begin{array}{c}\textit{Consumption of steel in kg for } 1\,m^2 \times \textit{Investment cost} \\ \textit{of steel for 1 kg} \times \textit{Rate of depreciation per month}\end{array}}{\textit{Number of uses in a month}} \tag{15.12}$$

For example if the requirement of steel components for 1 m^2 of wall formwork is 50 kg, the investment cost of steel is Rs. 45/kg, and the rate of depreciation is 3% per month, and it is expected that the steel components will be used 5 times in a month, then from the application of the above formula,

$$\text{The cost of steel components} = \frac{50 \times 45 \times 3\%}{5} = \text{Rs.}\ \frac{13.5}{\text{m}^2}$$

Alternatively, instead of depreciation, sometimes, the lease rental or hire charges for steel components on per kg basis may be considered. The resulting expression is as given below:

$$Cost\ of\ steel\ components = \frac{Consumption\ of\ steel\ in\ kg\ for\ 1\ m^2 \times Leased\ rental\ or\ hire\ charges\ of\ steel\ for\ 1\ kg}{Number\ of\ uses\ in\ a\ month} \tag{15.13}$$

Depending on the formwork type and the concrete element such as wall, column, slab, and beam, the requirement of steel would vary. For example, if L&T system is used for wall, column, and slab (flex system), the requirement of steel components is 50 kg, 98 kg, and 35 kg, respectively. The expression for the calculation of the cost remains same however.

E. Cost for usage of consumables

Consumables include form coating agents such as diesel and grease, or some proprietary form coating agents, nails, and binding wires. These are normally considered on a lump sum basis.

In some cases, it is possible to calculate the spread or coverage of form coating agent. In such cases, the cost of form coating agent can be estimated from the following expression:

$$Cost\ of\ form\ coating\ agent = \frac{Cost\ of\ form\ coating\ agent\ per\ liter}{Coverage\ or\ spread\ in\ m^2\ of\ 1\ liter\ form\ coating\ agent} \tag{15.14}$$

Suppose the cost of 1 liter of form coating agent is Rs. 20 and it has a coverage of 10 m^2, then the cost of form coating agent is Rs. 2/m^2.

The consumption of nails can be calculated by the following expression:

$$Cost\ of\ nails = \frac{Weight\ of\ nails\ in\ kg\ for\ 1\ m^2 \times Cost\ of\ nails\ per\ kg}{Number\ of\ reuse\ envisaged\ for\ the\ form} \tag{15.15}$$

Suppose 0.20 kg of nails is needed to make 1 m^2 of form, cost of nails is Rs. 40 per kg, and the form is expected to last for 10 uses then the cost of nails would be computed as below:

$$\text{Cost of nails} = \frac{0.20 \times 40}{10} = \text{Rs.}\ \frac{0.8}{\text{m}^2}$$

15.12.2 Labor Cost

A. Labor cost for making of formwork

The labor cost for making of formwork is obtained from the following expression:

$$Labor\ cost\ for\ making\ of\ formwork = \frac{Cost\ of\ making\ 1\ m^2\ of\ form}{Number\ of\ reuses\ envisaged\ for\ the\ form} \tag{15.16}$$

The cost of making the form is either taken from the productivity norms or based on the subcontractor quotation. If the cost of making 1 m^2 of form is Rs. 50 and the number of reuses envisaged for this form is 10, then the cost for making of formwork works out to be Rs. 5/m^2.

B. Labor cost for fixing and removing of formwork

The labor cost for fixing and removing of formwork can be estimated based on the prevailing rates for the similar item, or based on the productivity norms, or it could be based on the subcontractor's quotation if it is decided to be executed by a subcontracting agency.

15.12.3 Plant and Equipment Cost

Depending on the involvement of plant and equipment in the formwork activities, the cost can be estimated. For this, the hire charges of the respective plant and equipment and their expected time of use need to be considered. The total cost of plant and equipment is normally equally distributed on the total formwork area. For example, if the total charges towards plant and equipment hire charges is Rs. 10 lakhs and the total formwork area in the project is 1,00,000 m^2, then the cost of plant and equipment for 1 m^2 is Rs. 10.

15.12.4 Overhead and Profit

Depending on the contractor, the overhead cost and the profit margin vary. Normally, the contractor works out the total overhead cost and distributes it equally in prorated basis in all the items of the project. The profit margin is normally applied on a percent basis.

The unit rate is arrived at by adding the material cost, labor cost, plant and equipment cost, overhead, and profit margin as explained above.

A generic template which may prove to be useful for estimating the unit rates for the different formwork items is given in Table 15.13. The template can prove to be useful for different formwork systems, both proprietary as well as conventional. The template includes all possible cost heads. If some cost head is not applicable for a given system of formwork, the same can be excluded. For example, if H-16 beam is not to be used in a formwork system, the same may be excluded from the computation.

Table 15.13 Generic Template for Preparing Estimate of Unit Rate for 1 m^2 of Formwork

Sl. No.	Item Description	Amount (Rs.)
1.	Steel	$\text{Cost of steel components} = \dfrac{\text{Consumption of steel in kg for 1 m}^2 \times \text{Investment cost of steel for 1 kg @} \times \text{Rate of depreciation per month}}{\text{Number of uses in a month}}$ $\text{Cost of steel components} = \dfrac{\text{Consumption of steel in kg for 1 m}^2 \times \text{Leased rental or hire charge of steel for 1 kg}}{\text{Number of uses in a month}}$
2.	Timber	$\text{Cost of using timber} = \dfrac{\text{Consumption of timber in cubic meter for 1 m}^2 \times \text{Purchase cost of timber for 1 cubic meter}}{\text{Maximum number of uses envisaged}}$
3.	Plywood	$\text{Cost of using plywood} = \dfrac{\text{Consumption of plywood in square meter for 1 m}^2 \times \text{Purchase cost of plywood for 1 m}^2}{\text{Maximum number of uses envisaged}}$

Sl. No.	Item Description	Amount (Rs.)
4.	H-16 beam	Cost of using H-16 beam = $\dfrac{\text{Consumption of H-16 beams in running meters for 1 m}^2 \times \text{Investment cost of H-16 beam for 1 m} \times \text{Rate of depreciation per month}}{\text{Number of uses in a month}}$
5.	Consumable-Form coating agent	Cost of form coating agent = $\dfrac{\text{Cost of form coating agent per liter}}{\text{Coverage or spread in m}^2\text{of 1 liter form coating agent}}$
6.	Consumable-Nails	Cost of nails = $\dfrac{\text{Weight of nails in kg for 1 m}^2 \times \text{Cost of nails per kg}}{\text{Number of reuses envisaged for the form}}$
7.	Labor charges for making	Labor cost for making of formwork = $\dfrac{\text{Cost of making 1 m}^2\text{ of form}}{\text{Number of reuses envisaged for the form}}$
8.	Labor charges for fixing and removing 1 m^2	Based on productivity norms or on subcontractor's quotation
9.	Overhead and profit	Varies from contractor to contractor
10.	Unit rate/ m^2	Sum of the above

REVIEW QUESTIONS

Q1. True or False

(a) The two broad stages in formwork management are pre-award and post award.

(b) Wall formwork area to be mobilized.

$$= \frac{\text{Total formwork area planned for month} \times (\text{Cycle time})}{\text{Number of working days in a month}}.$$

(c) Cost of material per use = $\dfrac{\text{Cost of material-Scrap value}}{\text{Number of repetitions}}$.

(d) Cost of material per use

$$= \frac{(\text{Cost of material-Scrap value}) \times (\text{Period of use for item per project})}{\text{Life of materials}}.$$

(e) Form work direct cost includes - material cost, labor cost, and plant and equipment cost.

Q2. Match the following

(i) Pre-award stage	(a) Emphasis on detailed formwork planning, mobilization, and demobilization of material.
(ii) Post award stage	(b) Emphasis on formwork initial planning, value engineering, and costing.
(iii) Number of reuses in project	(c) Cycle time for one region × Number of regions in a floor × Number of floors.
(iv) Total duration	(d) Number of floors × Number of regions.

Q3. Match the following

(i) Material cost	(a) Making cost, erection and dismantling cost, repair and cleaning cost, transportation and shifting cost.
(ii) Labor cost	(b) Direct material cost and consumables.
(iii) Multi-story building	(c) Tower with plywood/timber.
(iv) Factory	(d) Study different possibilities and choose.
(v) Tall Structures like silo, chimney	(e) Prop system.
(vi) Shell roof	(f) Tower system.
(vii) Bridge or Dam	(g) Climbing formwork or slipforming.

Q4. Sequence the followings

(a) Selection of the formwork system.
(b) Estimate of the cycle time of the formwork activities.
(c) Cost estimation of the formwork materials.
(d) Computations of the formwork material requirements.
(e) Computations of the unit rates for the various formwork items.
(f) Formwork scheme preparation.
(g) Study of the customer requirement.

Q5. List out the steps in the computations of the formwork material requirement.

Q6. Discuss the pre-award formwork management.

Q7. Prepare a generic template for preparing the estimate of unit rate for 1 m^2 of formwork.

Chapter

16 Post-Award Formwork Management Issues

Contents: Introduction; Immediate Planning on Award of Contract; Post-award Formwork Management; Other Costs Affected by Formwork Plan; Striking Time; Formwork Economy During Construction Stage; Formwork Management— Key Positions and Their Responsibilities

16.1 INTRODUCTION

As discussed in the earlier chapters, it is evident that formwork is a vital material in any concrete construction. It was also brought to notice that any saving in the formwork can have a large implication in overall saving for the project. It is imperative therefore to have a scientific management of the formwork system in place in any construction organization that is involved with concrete construction. As far as a contractor is concerned, formwork management system is a must. The post-award formwork management should address the following major issues.

16.2 IMMEDIATE PLANNING ON AWARD OF CONTRACT

As soon as the contract is awarded, the plan of action for material mobilization must be in place. For this, the contractor can take input from the plans prepared at the time of tendering i.e., before the award of contract. The plan should consider any major changes observed in the pre-award specifications and after the award of contract specifications. These changes may have taken place during the negotiation process. The process of mobilization can even be started for long lead items based on the pre-award plan.

16.3 POST-AWARD FORMWORK MANAGEMENT

A schematic sketch showing the various tasks involved in the post-contract award formwork management is shown in Fig. 16.1. A brief discussion on these tasks follows in subsequent sections.

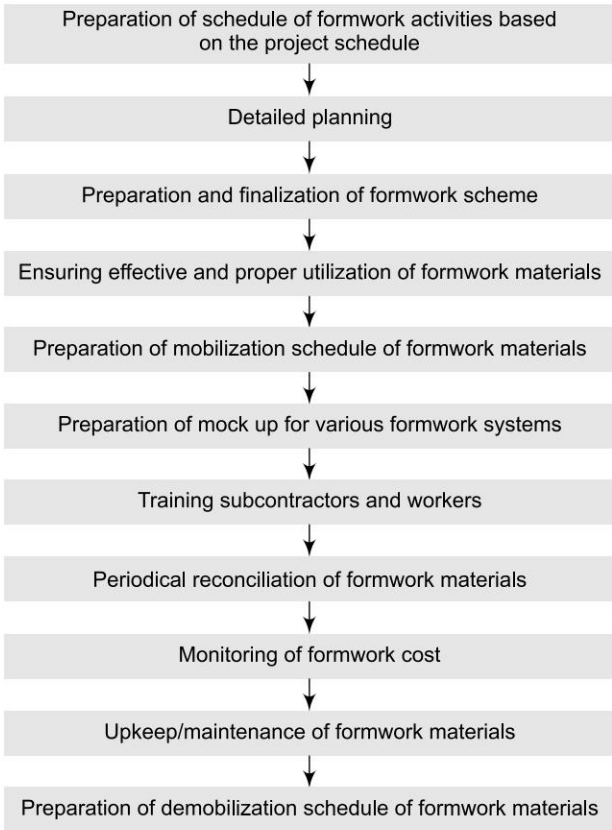


Figure 16.1 Schematic Sketch Showing Various Tasks Involved with the Post-contract Award Formwork Management.

16.3.1 Preparation of Schedule of Formwork Activities Based on the Project Schedule

On award of the contract, the contractor modifies the project schedule to take into consideration any changes that might have taken place between the schedule submitted at the time of tendering and the award of contract. From the project schedule, the formwork person culls out the schedule for the concrete construction works to make a detailed schedule of the formwork activities alone.

The contractor should estimate the quantities of various items of the formwork required for the entire project to meet the agreed construction schedule. This exercise should commence with the available information. In case the information available is not adequate, the consultants/designers/architects can be consulted to understand the nature of the structure proposed to be constructed. Sometimes even an informal meeting with the consultants/designers/architects can lead to information useful for planning. The model of the proposed constructed facility if available, can also be a source of information useful for planning purposes. Non-availability of information should not be regarded as a deterrent for the planning exercise, and efforts must be made to plan, irrespective of the likely errors that would creep in the plan due to the lack of details.

16.3.2 Detailed Planning

The amount of formwork material mobilized for a project has an important bearing on the cost of the formwork. Thus it is prudent on the part of the contractor to see that the requirement of the formwork material is correctly estimated and the materials are procured accordingly. It is advisable to prepare a schedule for the requirement of the formwork materials. Besides the correct estimate of the formwork materials, the labor requirement for the formwork activities should also be planned. The category of labor required, for example the number of carpenters, and helpers, etc. should also be spelled out correctly.

Illustration of detailed formwork planning exercise for a building project

Let's take a building project which has two towers (Fig. 16.2). Each tower has two basements and seven stories (Fig. 16.3). The lower basement is 72 m × 66 m while the upper basement is 54 m × 42 m. The seven stories are in uniform plan size of 48 m × 38 m. The concrete work (formwork + reinforcement + concreting) is scheduled to be completed in 10 months time.

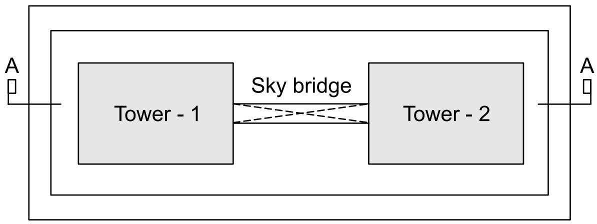


Figure 16.2 Plan of the Example Building.

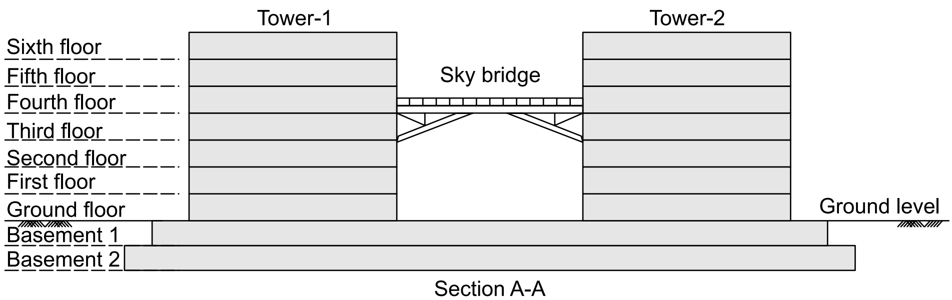


Figure 16.3 Section of the Example Building Project Fig 16.2 Showing Two Symmetrical Towers.

The floor to floor height is 3.5 m including the two basements. All beams are of uniform size: 300 mm width and 450 mm depth. The area for beam formwork can be assumed to be 30% of the plan area. All columns are square columns of 450 mm.side. The lower basement has 142 columns and the upper basement has 135 columns. In the remaining seven stories, there are 80 columns. It is proposed to calculate the formwork area to be mobilized each for column, beam, and slab formwork to complete the entire concrete work in the given duration.

The computation of formwork area for the column, beam, and slab for one tower is shown in Table 16.1. Since the other tower is exactly similar to tower 1, the quantities of formwork for the column, beam, and slab would get doubled.

Table 16.1 Computation of Formwork Area for Column, Beam, and Slab

		Slab area	Beam area@35% of slab area	Column area
1	Lower basement	$72 \times 66 = 4{,}752\ m^2$	$0.35 \times 4{,}752 = 1663.2\ m^2$	$142 \times (0.45 \times 2.90 \times 4) = 741.24\ m^2$
2	Upper basement	$54 \times 42 = 2{,}268\ m^2$	$0.35 \times 2{,}268 = 793.8\ m^2$	$135 \times (0.45 \times 2.90 \times 4) = 704.70\ m^2$
3	Ground floor	$48 \times 38 = 1{,}824\ m^2$	$0.35 \times 1{,}824 = 638.4\ m^2$	$80 \times (0.45 \times 2.90 \times 4) = 417.60\ m^2$
4	First floor	$48 \times 38 = 1{,}824\ m^2$	$0.35 \times 1{,}824 = 638.4\ m^2$	$80 \times (0.45 \times 2.90 \times 4) = 417.60\ m^2$
5	2nd Floor	$48 \times 38 = 1{,}824\ m^2$	$0.35 \times 1{,}824 = 638.4\ m^2$	$80 \times (0.45 \times 2.90 \times 4) = 417.60\ m^2$
6	3rd Floor	$48 \times 38 = 1{,}824\ m^2$	$0.35 \times 1{,}824 = 638.4\ m^2$	$80 \times (0.45 \times 2.90 \times 4) = 417.60\ m^2$
7	4th Floor	$48 \times 38 = 1{,}824\ m^2$	$0.35 \times 1{,}824 = 638.4\ m^2$	$80 \times (0.45 \times 2.90 \times 4) = 417.60\ m^2$
8	5th Floor	$48 \times 38 = 1{,}824\ m^2$	$0.35 \times 1{,}824 = 638.4\ m^2$	$80 \times (0.45 \times 2.90 \times 4) = 417.60\ m^2$
9	6th Floor	$48 \times 38 = 1{,}824\ m^2$	$0.35 \times 1{,}824 = 638.4\ m^2$	$80 \times (0.45 \times 2.90 \times 4) = 417.60\ m^2$

The schedule for the concrete work alone is shown in Fig. 16.4. It can be noticed that the two basements are planned to be completed in one and a half months each, while each of the seven stories is planned to be completed in one month.

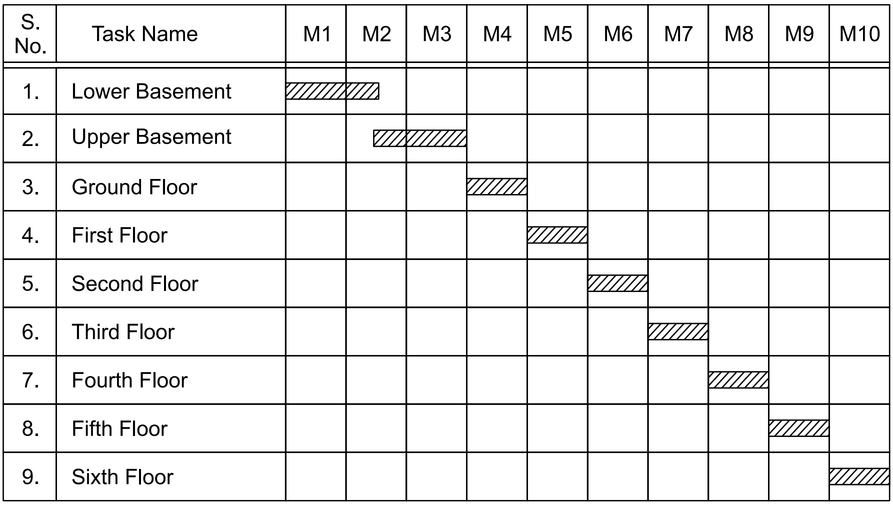


Figure 16.4 The Schedule for Concrete Work.

Lower Basement

Slab area = 4,752 m^2 for one tower. Let's divide the entire floor area in six units as shown in Fig. 16.5.

The construction of formwork normally involves the following operations:

(i) Propping and centering
(ii) Shuttering
(iii) Provision of camber
(iv) Cleaning and surface treatment

It is proposed that the entire concrete work in each unit be completed in one week duration, so that the floor gets completed in 6 weeks i.e., in one and a half months. The number of columns in each unit is equal to 142/6 = 23.6, say 24 sets. Assume that the contractor has prepared 20 sets of columns for this project.

Unit 1	Unit 2	Unit 3
Unit 4	Unit 5	Unit 6

Figure 16.5 Division of Basement Floor Area in Six Units.

Typical floor

The area of slab in a typical floor = 48 m × 38 m = 1,824 m^2. Let's divide the entire floor area in four units as shown in Fig. 16.6. It is proposed that the entire concrete work in each unit be completed in one week duration so that the floor gets completed in 4 weeks, i.e. in one month. The number of columns in each unit is equal to 80/4 = 20. Assume that the contractor has prepared 20 sets of columns for this project. The detailed schedule is given in Fig. 16.7.

Unit 1	Unit 2
Unit 3	Unit 4

Figure 16.6 Division of Typical Floor Area in Four Units.

Beam area = $0.35 \times 1{,}824 = 638.4\ m^2$

Column area = $80 \times (0.45 \times 2.90 \times 4) = 417.60\ m^2$

16.3.3 Preparation and Finalization of Formwork Scheme

The formwork scheme, corresponding to various typical formwork activities, is prepared. The formwork scheme would be dependent on the system of formwork selected for the various activities. The formwork scheme should mention the following features:

- All major design values and loading conditions should be shown on the formwork drawings.
  - values of live load;
  - the compressive strength of concrete for formwork removal and for application of construction loads;
  - rate of placement;
  - temperature;
  - height and drop of concrete;
  - weight of moving equipment;
  - design stresses;
  - camber diagrams;
- Types of materials, size length, and connection details.
- Procedures, sequence and criteria for removal of forms, shores and re-shores.

- Design allowance for construction loads on new slabs should be shown when such allowances would affect the development of the shoring and / or re-shoring schemes.
- Anchors, forms ties, shores, lateral bracing, and horizontal lacing.
- Field adjustment of forms.
- Waterstops, keyways, and inserts.
- Working scaffolds and runways.
- Weepholes or vibrator holes where required.
- Screeds and grade strips.
- Location of external vibrator mountings.
- Crush plates or wrecking plates where stripping may damage the concrete.
- Removal of spreaders or temporary blocking.
- Cleanout holes and inspection openings.
- Construction joints, contraction joints, and expansion joints to conform to design drawings.
- Sequence of concrete placement and minimum elapsed time between adjacent placements.
- Chamfer strips or grade strips for exposed corners and construction joints.
- Camber.
- Mudsills or other foundation provisions for formwork.
- Special provisions such as safety, fire drainage, and protection from ice and debris at water crossings.
- Formwork coatings.

16.3.4 Ensuring Effective and Proper Utilization of Formwork Materials

System formwork consists of a large number of components liable to misuse/abuse. During the execution of formwork, engineers should take interest and give guidance in planning the arrangement and correcting the abuse/misuse of materials. They should also ensure that the materials are utilized to the optimum, and there is neither the under utilization of materials nor unsafe practices. Some common examples of abuses are the use of bracings as crow bars, or as tools for the adjustment of reinforcement or as tools for lifting and transporting materials like bundles of wires, tubes, etc.

Damages to components occur often due to abuse and misuse, and seldom due to repeated proper use. Attention has to be paid to prompt de-shuttering of materials by keeping track of the concreting dates and curing times. This contributes towards material productivity. It is a good practice to write the date on which a concrete element was cast.

Sufficient importance has also to be attached to labor productivity. The size of labor gang has to be kept small (3-4 persons per gang) so that their effectiveness is optimum. Proper tools and accessories have to be employed to improve productivity. The contractor should plan the arrangement of formwork to obtain the optimum usage of materials keeping in view the type of structure under construction. As mentioned elsewhere also, workers' attitude is one of the prime factors determining the reuse of formwork materials and thus it is imperative that the morale of the workers is kept high through different means, such as monitory incentives and so on.

Wherever handling equipments like cranes etc. are used, the size of panels, the method of erecting and dismantling have to be worked out in such a way that the equipment is effectively utilized. Procurement of materials such as plywood, timber, etc. must be monitored on a continuous basis. Care should be taken to see that the material procurement does not exceed the initial planned quantities.

16.3.5 Preparation of Mobilization Schedule of Formwork Materials

It is necessary that the contractor plans the requirement of formwork materials for the entire construction project even if the same is approximate. This can be refined in the course of time as and when the drawings are released or as and when the details become clearer. A contractor normally has four mobilization options: (1) to buy the materials afresh, (2) to get the materials on hire or rent, (3) to get the materials from his own construction site(s) which is (are) under demobilization, and (4) to get the materials from one of the central stores. For large projects, generally the contractor employs a formwork engineer, who is responsible for the mobilization of formwork materials. The formwork engineer has some other responsibilities too which are dealt with elsewhere in the text.

The requirement of formwork is given to the overall formwork-in-charge of the contractor by the concerned formwork engineer stationed at the project site. Then it's the duty of the formwork department to procure the material. The method of mobilization depends on the number of estimated reuses of materials, the cost associated with each of the mobilization alternatives, etc. Also, in the event of the contractor favoring the buying option of material mobilization, the mobilization time is an important consideration. In such cases, the contractor should allow sufficient time of mobilization so that a better price can be obtained from the suppliers of the formwork materials.

16.3.6 Preparation of Mock-up for Various Formwork Systems

Wikipedia defines a *mockup*, or *mock-up*, as a scale or full-size model of a design or device, used for teaching, demonstration, evaluating a design, promotion, and other purposes. A mockup is called a prototype if it provides at least part of the functionality of a system and enables testing of a design.

In the context of formwork, mock-up is used to understand the difficulties that would be faced during the real time implementation of concrete construction. In general, mock-ups are used in the construction of some unusual shapes such as the one shown in Fig. 16.8. In the figure, a mock-up of sloping roof is shown. The need for mock-up arose because of the proposal of a very unconventional kind of roof for a memorial structure. The artist's impression of the proposed facility is shown in Fig. 16.9. The similarity of the sloping roofs can be easily noticed between the two figures.

By constructing the mock-up, the designer, architect and the contractor would be in a position to appreciate the likely difficulties to be faced during the actual construction. Some of the issues that would be looked into would be: whether the reinforcement would be properly tied, whether it is possible to get the required cover for the concrete, whether the concrete would be in a position to flow, whether the finish obtained is acceptable. Once all the bottlenecks confronted during the mock-up stage are sorted out, the go-ahead to real construction is given.

Figure 16.8 Mock Up of Sloping Roof in Progress.

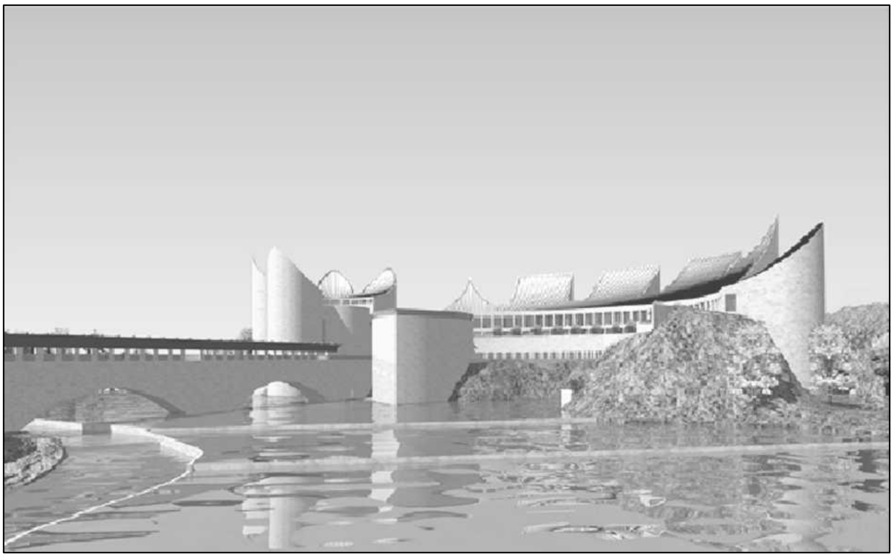


Figure 16.9 Architects Impression of Khalsa Heritage Memorial Complex.

A mock-up can also be prepared for a typical formwork system so that the workers get a fair idea of working with the particular formwork system. Normally such mock-ups are installed at some prominent location at the site and any new carpentry crew engaged at the site is shown these mock-ups so that they take little time to get familiarized with the system. One such mock up is shown in Fig. 16.10. In the figure, mock-ups of wall and column formwork are shown. Any one viewing the mock-up would understand the kind of components used in the system, the connection details and so on.

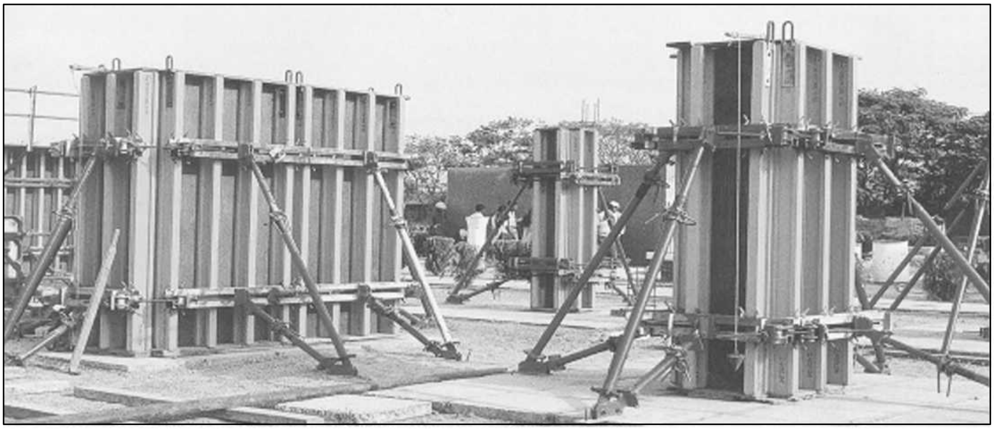


Figure 16.10 Mock-up to Explain the Wall and Column Formwork.

16.3.7 Training Subcontractors and Workers

The role of training cannot be overlooked. It is essential that the knowledge gained pertaining to different formwork applications is disseminated among the concerned staff. This may be in the form of conducting a workshop or a short term training program. The faculty for such programs must be carefully chosen by the contractor. The faculty could be from the contractor's organization itself or the contractor can hire an external expert for this purpose. When buying a new formwork system, provision of providing training to the concerned staff of the contractor by the supplier of the system should also be kept, so that the desired productivity is obtained from the very beginning.

16.3.8 Periodical Reconciliation of Formwork Materials

Apart from losing materials due to abuse and rough handling, there is a loss of accessories and small items due to bad housekeeping and lack of upkeep and maintenance. The contractor may find it economical to even engage a separate labor crew to attend to this aspect alone. It may be worthwhile for the contractor to issue the small accessories such as pins, couplers, bracings, etc. on chargeable basis. This will, in a way, make the subcontractors accountable for the materials which would prevent the misuse and loss of materials. There must be reconciliation and physical counting of the formwork materials at regular intervals. The contractor should also adopt a system of panelizing the individuals/subcontractors found to be misusing the materials.

16.3.9 Monitoring of Formwork Cost

It is absolutely essential that like other activities, the cost of formwork activities is also monitored regularly. In the industry, it is normal to monitor the cost of formwork activities on a monthly basis. Cost codes are assigned to the formwork activities (Table 16.2). The cost of executing the formwork activities is frozen post award of contract.

Table 16.2 Cost Codes for Monitoring Formwork Activities

Sl. No.	Formwork activities	Cost code
1.	Labor charges for making column, beam bottom shutter	2010
2.	Labor charges for making wall and beam side shutter	2020
3.	Labor charges for repairing of shutter	2030
4.	Labor charges for fixing and removing column shutter	2040
5.	Labor charges for fixing and removing beam and slab shutter	2050
6.	Labor charges for scaffolding	2060
7.	Lift for column shuttering for every 3 m	2070
8.	Lift for slab and beam shuttering for every 3 m	2080
9.	Labor charges for shifting of shutter	2090
10.	Labor charges for shifting of scaffolding	2100
11.	Hire charges for scaffolding	2110
12.	Hire charges for steel formwork for beam and slab	2120
13.	Cost of plywood	2130
14.	Cost of wood	2140
15.	Consumables (wire nail, oil, binding wire etc)	2150
16.	Cost for plant and machinery	2160

As soon as the project is awarded to the contracting organization and the letter of intent issued by the client, the contractor appoints a project manager and a planning engineer for the project. The two then go through the entire contract document and prepare the estimates of executing each bid item. They also prepare the estimate for the overheads of the project.

Cost codes are established and the budgeted estimates for each of the cost codes are prepared. These budgeted estimates are scrutinized by the top management of the contracting organization and a mutually agreeable budgeted estimate is agreed upon. This is called the accepted cost estimate (ACE) or zero cost by the contracting organization. The monitoring of formwork activities is performed with respect to the accepted cost estimate (ACE). In Table 16.3, the accepted cost estimate for formwork activities for a project is given. The quantities are drawn from the bill of quantity and in this case it is assumed to be 28,000 m^2.

Table 16.3 The Accepted Cost Estimate for Formwork Activities for a Project

Cost Code	Item Description	Unit	Quantity	Accepted cost estimate	
				Rate (Rs.)	*Amount (Rs.)*
2010	Labor charges for making column, beam bottom shutter	m^2	800	105.00	84,000
2020	Labor charges for making wall and beam side shutter	m^2	200	90.00	18,000
2030	Labor charges for repairing of shutter	m^2	500	50.00	25,000
2040	Labor charges for fixing and removing column shutter	m^2	2,800	75.00	2,10,000
2050	Labor charges for fixing and removing beam and slab shutter	m^2	17,053.4	50.00	8,52,670
2060	Labor charges for scaffolding	m^3	34,106.8	20.00	6,82,136

Cost Code	Item Description	Unit	Quantity	Accepted cost estimate	
				Rate (Rs.)	*Amount (Rs.)*
2070	Lift for column shuttering for every 3 m	m^2	16,800	5.00	84,000
2080	Lift for slab and beam shuttering for every 3 m	m^2	1,02,320.4	5.00	5,11,602
2090	Labor charges for shifting of shutter	m^2	17,053.4	5.00	85,267
2100	Labor charges for shifting of scaffolding	m^2	34,106.8	5.00	1,70,534
2110	Hire charges for scaffolding	m^2	4,872.4	210.00	10,23,204
2120	Hire charges for steel formwork for beam and slab	m^2	3,000	560.00	16,80,000
2130	Cost of plywood	m^2	2,000	1,000.00	20,00,000
2140	Cost of timber	m^3	1,000	750.00	7,50,000
2150	Consumables (wire nail, oil, binding wire etc)	LS	17,053.4	20.00	3,41,068
2160	Cost for plant and machinery	m^2	17,053.4	30.00	5,11,602
	Sub Total				**90,29,083**
	Unit cost of formwork = (90,29,083 ÷ 28,000)				**322.47**

On a monthly basis, the cost against each of the mentioned cost codes is obtained. The cost monitoring has been done for the month of November, December and January, and it is shown in Tables 16.4, 16.5, and 16.6, respectively. It is assumed that formwork quantities executed in each of these months is 4,000 m^2.

Table 16.4 The Details of Cost for the Month of November

Cost Code	Item Description	Unit	Quantity	Cost (November)	
				Rate (Rs.)	*Amount (Rs.)*
2010	Labor charges for making column, beam bottom shutter	m^2	114.3	122.50	14,000
2020	Labor charges for making wall and beam side shutter	m^2	28.6	126.00	3,600
2030	Labor charges for repairing of shutter	m^2	71.4	116.67	8,333
2040	Labor charges for fixing and removing column shutter	m^2	400	105.00	42,000
2050	Labor charges for fixing and removing beam and slab shutter	m^2	2,436.2	58.33	1,42,112
2060	Labor charges for scaffolding	m^3	4,872.4	35.00	1,70,534
2070	Lift for column shuttering for every 3 m	m^2	2,400	7.00	16,800
2080	Lift for slab and beam shuttering for every 3 m	m^2	14,617.2	7.00	1,02,320
2090	Labor charges for shifting of shutter	m^2	2,436.2	7.00	17,053
2100	Labor charges for shifting of scaffolding	m^2	4,872.4	5.83	28,422
2110	Hire charges for scaffolding	m^2	696.1	245.00	1,70,534
2120	Hire charges for steel formwork for beam and slab	m^2	428.6	653.33	2,80,000
2130	Cost of plywood	m^2	285.7	1,000.00	2,85,714
2140	Cost of wood	m^3	142.9	750.00	1,07,143
2150	Consumables (wire nail, oil, binding wire etc)	LS	2,436.2	20.00	48,724
2160	Cost for plant and machinery	m^2	2,436.2	30.00	73,086
	Sub Total				**15,10,376**
	Unit cost of formwork = (15,10,376 ÷ 4,000)				**377.59**

Table 16.5 The Details of Cost for the Month of December

Cost Code	Item Description	Unit	Quantity	Cost (December)	
				Rate (Rs.)	*Amount (Rs.)*
2010	Labor charges for making column, beam bottom shutter	m^2	114.3	147.00	16,800
2020	Labor charges for making wall and beam side shutter	m^2	28.6	126.00	3,600
2030	Labor charges for repairing of shutter	m^2	71.4	58.33	4,167
2040	Labor charges for fixing and removing column shutter	m^2	400	87.50	35,000
2050	Labor charges for fixing and removing beam and slab shutter	m^2	2,436.2	58.33	1,42,112
2060	Labor charges for scaffolding	m^3	4,872.4	28.00	1,36,427
2070	Lift for column shuttering for every 3 m	m^2	2,400	7.00	16,800
2080	Lift for slab and beam shuttering for every 3 m	m^2	14,617.2	7.00	1,02,320
2090	Labor charges for shifting of shutter	m^2	2,436.2	7.00	17,053
2100	Labor charges for shifting of scaffolding	m^2	4,872.4	7.00	34,107
2110	Hire charges for scaffolding	m^2	696.1	294.00	2,04,641
2120	Hire charges for steel formwork for beam and slab	m^2	428.6	653.33	2,80,000
2130	Cost of plywood	m^2	285.7	1,000.00	2,85,714
2140	Cost of wood	m^3	142.9	750.00	1,07,143
2150	Consumables (wire nail, oil, binding wire etc)	LS	2,436.2	20.00	48,724
2160	Cost for plant and machinery	m^2	2,436.2	30.00	73,086
	Sub Total				**15,07,694**
	Unit cost of formwork = (15,07,694 ÷ 4,000)				**376.92**

Table 16.6 The Details of Cost for the Month of January

Cost Code	Item Description	Unit	Quantity	Cost (January)	
				Rate (Rs.)	*Amount (Rs.)*
2010	Labor charges for making column, beam bottom shutter	m^2	114.3	122.50	14,000
2020	Labor charges for making wall and beam side shutter	m^2	28.6	105.00	3,000
2030	Labor charges for repairing of shutter	m^2	71.4	58.33	4,167
2040	Labor charges for fixing and removing column shutter	m^2	400	87.50	35,000
2050	Labor charges for fixing and removing beam and slab shutter	m^2	2,436.2	58.33	1,42,112
2060	Labor charges for scaffolding	m^3	4,872.4	23.33	1,13,689
2070	Lift for column shuttering for every 3 m	m^2	2,400	5.83	14,000
2080	Lift for slab and beam shuttering for every 3 m	m^2	14,617.2	5.83	85,267
2090	Labor charges for shifting of shutter	m^2	2,436.2	5.83	14,211
2100	Labor charges for shifting of scaffolding	m^2	4,872.4	5.83	28,422

Cost Code	Item Description	Unit	Quantity	Cost (January)	
				Rate (Rs.)	*Amount (Rs.)*
2110	Hire charges for scaffolding	m^2	696.1	294.00	2,04,641
2120	Hire charges for steel formwork for beam and slab	m^2	428.6	560.00	2,40,000
2130	Cost of plywood	m^2	285.7	1,000.00	2,85,714
2140	Cost of wood	m^3	142.9	750.00	1,07,143
2150	Consumables (wire nail, oil, binding wire etc)	LS	2,436.2	20.00	48,724
2160	Cost of plant and machinery	m^2	2,436.2	30.00	73,086
	Sub Total				**13,54,707**
	Unit cost of formwork = (13,54,707 ÷ 4,000)				**338.68**

The variation against each of the cost codes is obtained by obtaining the difference between the cost for the month from the accepted cost estimate (Table 16.7). The negative variation indicates trouble areas and needs timely management attention.

Table 16.7 The Variation Statement for the Month of January

Cost Code	Item Description	Unit	Quantity	Variation From November	Variation From Original Estimate
2010	Labor charges for making column, beam bottom shutter	m^2	800	0.00	–17.50
2020	Labor charges for making wall and beam side shutter	m^2	200	–21.00	–15.00
2030	Labor charges for repairing of shutter	m^2	500	–58.33	–8.33
2040	Labor charges for fixing and removing column shutter	m^2	2,800	–17.50	–12.50
2050	Labor charges for fixing and removing beam and slab shutter	m^2	17,053.4	0.00	–8.33
2060	Labor charges for scaffolding	m^3	34,106.8	–11.67	–3.33
2070	Lift for column shuttering for every 3 m	m^2	16,800	–1.17	–0.83
2080	Lift for slab and beam shuttering for every 3 m	m^2	1,02,320.4	–1.17	–0.83
2090	Labor charges for shifting of shutter	m^2	17,053.4	–1.17	–0.83
2100	Labor charges for shifting of scaffolding	m^2	34,106.8	0.00	–0.83
2110	Hire charges for scaffolding	m^2	4,872.4	–35.00	0.00
2120	Hire charges for steel formwork for beam and slab	m^2	3,000	–93.33	0.00
2130	Cost of plywood	m^2	2,000	0.00	0.00
2140	Cost of wood	m^3	1,000	0.00	0.00
2150	Consumables (wire nail, oil, binding wire etc)	LS	17,053.4	0.00	0.00
2160	Cost for plant and machinery	m^2	17,053.4	0.00	0.00

16.3.10 Upkeep/Maintenance of Formwork Materials

Before each use of timber, it should be inspected for any visual damage. The defect such as signs of rot, large cuts on the edges of the timber piece, undue distortion of shapes, splitting, and mechanical damages, etc. tend to reduce the capacity of the timber and it is prudent to discard them.

The metal formwork before each use should also be inspected carefully. Any instance of the concrete or mortar sticking on the metal formwork surfaces should be cleared and only then it should be reused. If the metal formwork is not in use, they should be properly stored. There should be regular cleaning and painting of all the formwork components. The threaded parts need oiling and greasing.

In a large construction company, the formwork materials move from one site to another site. It is the tendency of the releasing site not to take care of the materials after it has been used by them. The releasing site starts considering the formwork materials as a burden to them and would like to get rid of them at the earliest possible instance. However, this should not be encouraged.

The contractor should develop a system of upkeep/maintenance of the formwork material before dispatching it to other sites. A provision for the time taken for the upkeep/maintenance must be kept by the releasing site before communicating the release dates to other sites.

16.3.11 Preparation of Demobilization Schedule of Formwork Materials

Just as the requirement schedule was made in the beginning of the project, the release schedule should be made towards the closing stages of the job. For any large demobilization, the program has to be made at least 2-3 months before the finish of the project and handing over to the formwork department for further action. The formwork-in-charge should take action on the release of material either to a new construction site or to the central stores. If this is not properly monitored, then there is a chance of the material lying unutilized at the site and also leading to extra cost by paying hire charges to the owner. If the material is own material, then this may not be utilized by the other new construction site. Thus, unnecessary procurement of the new material will take place which is an extra cost.

16.4 OTHER COSTS AFFECTED BY FORMWORK PLAN

As mentioned earlier in the text that the primary objective of formwork is to achieve a good concrete quality and safe performance at the lowest overall cost for the project. It is also mentioned earlier that the formwork cost is only a considerable portion of the overall RCC cost. It is important to reduce the formwork cost but we should also try to minimize the overall cost of the project i.e., the formwork cost and the other costs affected by a formwork plan. Some examples of the other costs affected by formwork plan are: crew efficiency, concreting, bar setting, cranes and hoist etc. These are discussed briefly below:

16.4.1 Crew Efficiency

Overall crew efficiency increases only when there is an advanced planning and coordination among formwork, reinforcement and concreting crews. There is no point in one crew working constantly and the other crew sitting idle for most of the times. Plans should be prepared in such a way that same operation is repeated in an orderly manner and each crew knows when they have to perform the work.

16.4.2 Concreting

Formwork plan affects the way concreting activity is to be planned. A planner often faces the task of choosing between high and low lift, large and small form units etc. In such cases, he or she has to tradeoff between formwork crew cost and concrete crew cost to minimize the overall cost.

16.4.3 Bar Setting

A proper coordination between reinforcement crew and formwork crew is needed for overall economy to be maintained. If possible, the same subcontractor should be entrusted for both these tasks. Lack of planning and coordination often results in idling of either the formwork or the reinforcement crew.

16.4.4 Cranes and Hoists

Cranes and hoists are increasingly employed in modern construction these days. Both mobile and fixed cranes are used for a number of operations including formwork material handling, shifting of formwork panels and so on. In order to utilize the cranes in an efficient way, proper scheduling should be prepared to allocate the crane time to different operations. Random and adhoc allocation result in either the crane or a particular crew idling. For efficient crane operation, movement of crane (typically mobile crane) should also be planned. Selection of appropriate crane is also dependent on the weight of formwork panels and the distance to which they are to be handled.

16.5 STRIKING TIME

Striking time is another important factor that decides the number of repetitions available within a specified period. As per clause 10.3 of IS 456-2000, forms can be struck when the concrete attains the strength at least twice the stress to which the concrete may be subjected at the time of removal of formwork. The code has also given recommendations as guidance for cases where no computations are made with regard to the actual stresses while de-shuttering. Many a times, the specifications drawn out stipulate unreasonably long striking times without any consideration of the actual stresses in concrete or the strength attained by the concrete while de-shuttering and thus increase the cost of shuttering and the cost of the project unnecessarily.

The detailing of formwork should be such that the sequence of dismantling of the various components should exactly match with the striking time specified for each component. For example, in the case of a multi-story building, it should be possible to remove the sides of the beam, the soffit of slab and the soffit of beams in a sequential manner to achieve the maximum number of reuses for each of these components. Accelerated methods of curing can also help in reducing the striking time and thus increase the possible number of reuses.

In the case of multi-story buildings where very high speeds of construction are aimed at, it may become necessary for a floor to support not only the self weight but also of the upper floor along with the shores and reshores supporting the upper floor. This should be kept in mind while arriving at the striking time. Complexity of the structure also affects the striking time. A complex structure can result in the blocking of materials in one place for relatively long durations and thus reduce the reuse value of materials.

16.6 FORMWORK ECONOMY DURING CONSTRUCTION STAGE

The greatest economy and efficiency of construction are achieved by using a *minimum number of operations* on site, which includes *minimizing the number of different components* which should be assembled in a repetitive, continuous process, simplifying the field connecting and minimizing the start-stop of any activity. It means each trade should be in control of its own activity without interference. Hence, a proper construction sequence of the different trades, which in turn has an effect on the speed of construction, should be in place.

16.6.1 Economical Form Construction

The economical form construction is an important aspect at the construction stage. More so in the case of forms with limited reuse. Such forms should be built with inexpensive materials that are easy to transport, handle, and shape. For the forms with large numbers of reuses, it is advisable to build them using strong and durable materials. These forms should have features which enable easy assembly and handling.

For large formwork surfaces, it is economical to use large plywood panels as it would require less assembly, and less number of joints requiring finishing. The choice of job-built form and shop-built form also require careful consideration as there is no single solution, and depending on the job condition, sometimes it may be economical to go for the job-built form while in some cases it may be economical to go for the shop-built form. The job-built form can be associated with large waste of materials and almost nil transportation cost, while the shop-built form can be associated with large productivity, less waste, but high transportation cost.

The form construction should address the reusability aspect since greater economy can be achieved only when it is possible that the forms are stripped and re-erected faster. If it can be ensured that during the stripping process, the form is not wrecked, the number of reuses can be further increased. This calls for provisions in the formwork such as metal clamp or special wedge pin connection, etc. to realize easy handling and disassembly. Good formwork construction has the provision of lifting and walkway platforms for ease in erection and stripping.

During form construction, the initial cost alone should not be the deciding factor. For example, in the selection of ties and inserts for form construction, the cost towards labor charges for their fixing and removal should also be considered. The transportation cost also could be quite substantial while transporting form sections from the carpentry shop to the job site, and thus should be carefully considered.

16.6.2 Removal of Forms and Shores

Concrete hardens slowly in the natural process. This calls for forms and their supports to be left in their place for a long time. Shortening this time of release and early reuse are some of the concerns the designers face.

Formwork should be removed as early as possible; however, they should not be removed until the concrete has achieved sufficient strength to carry its own weight and any loads superimposed during the course of further construction. The timing and sequence of form removal should take into account the characteristics of the cement, the mix proportion of the concrete, importance of the structures, type and dimensions of all members, imposed loads to all members, temperature, weather and ventilation, and so on and so forth.

Where it is necessary to impose loads on the structures soon after the removal of forms and shores, the concrete strength, the types of structures, and the characteristics and the values of the imposed loads should be carefully considered in order to avoid harmful cracks and other damages to the structure.

Strength development of the concrete should be the prime criteria for deciding on removal of forms. Strength developed will depend upon the magnitude of the loads and rate of gain in the strength of the concrete. However, in section II of IS:456–2000 on formwork and IS:14687–1999, some guidelines for removal of forms for different concrete elements are presented. These are produced in Table 16.8.

Table 16.8 Removal Times for Different Concrete Elements

Sl.No	Various elements		Time
1.	Vertical formwork to columns, walls, beams		16-24 h
2.	Soffit formwork to slabs(props left under)		3 days
3.	Soffit formwork to beams		7 days
4.	Props to slab	a. Span of 4.5 m	7 days
		b. Span > 4.5 m	14 days
5.	Props to beam and arches	a. Span up to 6 m	14 days
		b. Span > 6 m	21 days

Striking of forms for large span concrete shell structures has to be done as prescribed by the designer in such a way that the load transfer takes place without upsetting the designed stresses. Several failures have taken place due to neglect on this part of the work.

16.7 FORMWORK MANAGEMENT— KEY POSITIONS AND THEIR RESPONSIBILITIES

As pointed out at various locations throughout the text, formwork constitutes a major resource of the contracting organization. For a large organization, the investment in formwork runs into crores of rupees. It is thus imperative that formwork is managed in the same light as any other resource in the organization. A typical organization chart for managing formwork resource in a large contracting organization is shown in Fig. 16.11.

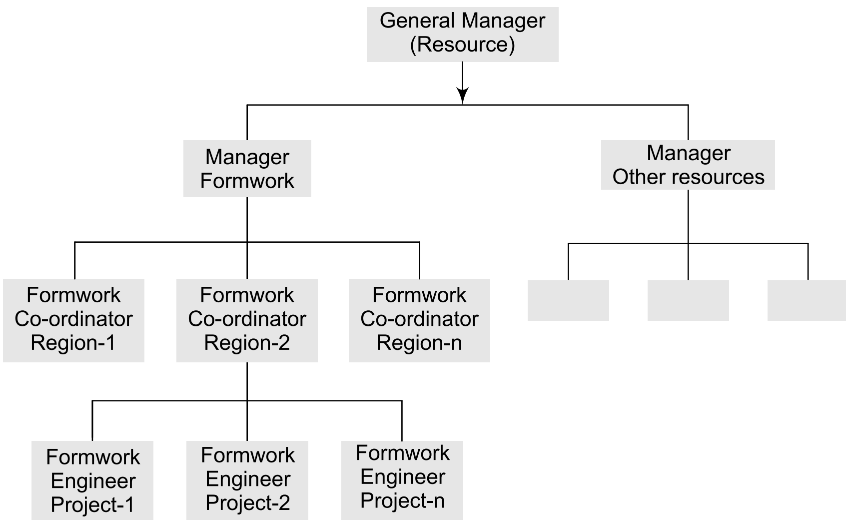


Figure 16.11 The Organization Chart for Managing Formwork in a Large Contracting Organization.

It is assumed here that a large contracting organization has a large number of project sites at any point of time spread over different regions in the country. The organization has a number of regional offices with each regional office having a number of sites to manage. In Fig. 16.11, only three regional offices have been shown.

The organization has a general manager (resources) stationed at the headquarters who is supported by managers responsible for managing different resources. One manager has been shown exclusively for managing the formwork. The manager—formwork is supported by regional formwork coordinators located at various regional offices. The regional formwork coordinator is responsible for coordinating with the site formwork engineer within his region and the formwork manager based at the headquarters of the organization.

The responsibilities of the formwork engineer located at the sites are given below:

1. Preparing formwork schedule in line with the project construction schedule. The schedule would show all the milestones pertaining to the formwork.
2. Preparing the formwork scheme. The formwork engineer prepares the scheme for typical formwork structures on his own while for complicated structures, the formwork engineer may require support from the designers.
3. Based on the formwork scheme, the formwork engineer prepares the bill of materials required for the formwork items and mobilizes with the help of the regional formwork coordinator. This is to be updated at frequent intervals so that the mobilization and demobilization of formwork materials is undertaken smoothly. The formwork engineer has to intimate the receipt and release of any consignment promptly.
4. The formwork engineer is also responsible for the effective utilization of formwork materials and thus he is also entrusted with the responsibility of allocating the formwork materials to the competing site engineers.
5. As large organizations often engage subcontractors for the formwork activities, it is imperative that the subcontractors should also be trained in using a given type of formwork system. Formwork engineer plays a major role in training the subcontractor's workmen. He is also instrumental in installing the mock ups of the various formwork system proposed to be used at the site as ready reference for the workmen.
6. The reader must have noticed that the formwork system contains numerous formwork items. Lack of even a single item/component may render an entire formwork unit useless. Thus it is very essential that each and every component of the formwork system is taken care of, and reconciliation is carried out physically at regular intervals. The formwork site engineer is also entrusted with the responsibility of physical reconciliation of formwork components at regular intervals. He has to have close interactions with persons involved at the stores and in the execution at project site.
7. In some organizations, the formwork engineer is also entrusted with the responsibility of the purchase of some formwork materials such as plywood, timber, etc.
8. The formwork engineer is also responsible for the maintenance of formwork items.

REVIEW QUESTIONS

Q1. True or False

(a) Accepted Cost Estimate (ACE) is also known as zero-cost by the contracting organization.

Q2. Sequence the following in the context of tasks involved in post-contract award formwork management.

(i) Training subcontractors and workers

(ii) Monitoring of formwork cost

(iii) Periodical reconciliation of formwork materials

(iv) Preparation of demobilization schedule of formwork materials

(v) Upkeep/maintenance of formwork materials

(vi) Preparation and finalization of formwork scheme

(vii) Preparation of mock-up for various formwork systems

(viii) Preparation of mobilization schedule for formwork materials

(ix) Ensuring effective and proper utilization of formwork materials

(x) Detailed planning

(xi) Preparation of schedules of formwork activities based on the project schedule

Q3. Discuss the different steps involved in formwork management in post-award stage.

Q4. List out the different features involved in preparation and finalization of formwork scheme.

Q5. List out the various responsibilities of the formwork engineer located at the sites.

Q6. Why is mock-up important in the context of formwork?

Q7. What kind of construction practice (comment only about formwork system consisting of sheathing material, and staging material) would you adopt as far as formwork is concerned under the following circumstances?

(a) Multi-story building with floor to floor height less than about 4 m

(b) A factory shed with floor height 10 m

(c) A cement plant where a number of tall structures such as Silo are present

(d) An airport hangar with folded roof

In case 1, 2, and 4, the repetition of the sheathing material is expected to be about 10-12 due to time constraint, while in case 3, repetitions is expected to be about 100.

Q8. An owner had started work for his new hotel building. He took a lot of time in completing the foundation of the building. So one day he decided to get the remaining job completed through one contractor. Calculate the formwork area (for columns and slab/beam alone) that needs to be mobilized by the contractor in order to complete the RCC work for the entire six-story building (see Fig Q16.8.1) in 6 months time?

Slab and beam formwork material you can repeat every 15 days.

Column formwork material you can repeat every 3 days.

All columns of size 400 × 400 mm

All beams 400 mm wide and 600 mm deep

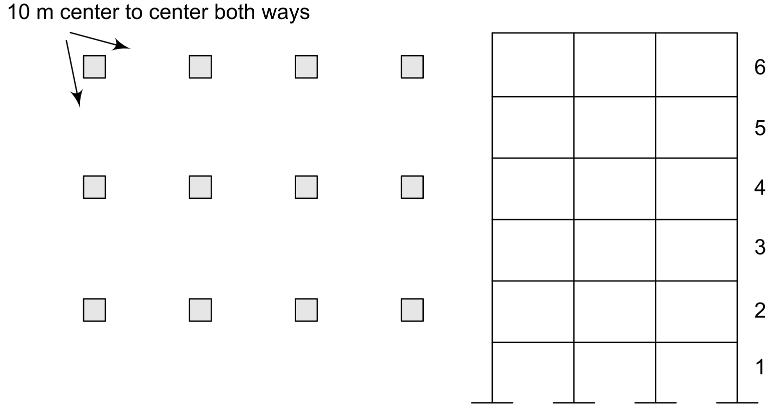


Figure Q 16.8.1 Plan and Section for Question 8

Q9. A casting yard produces pre-stressed concrete beams of size 60 mm width and 200 mm depth and 2.4 m length. Total 5,000 beams are to be cast. Beams can be moved out of the casting bed only after 7 days. The total duration for completion of the beam casting is 180 days. How many casting beds would be required to achieve this construction schedule? Also give typical sketches for the casting yard for such a production. There is no constraint of land availability.

Q10. Formwork is to be designed for a wall 3.5 m and 350 mm thick. The client's specification limits deflection to 1/270 of the span of any formwork member. Through ties are allowed. The concrete will be OPC with admixture but no retarder. The work will take place in April. The temporary works designer has been advised that concrete will be placed by skip, at an assumed volume rate of supply of 9 m^3/h. The concrete temperature for April is assumed to be 12°C. The site crane will lift 2,000 kg at all radii. Wall pour length = 5.7 m, size considered = 3.8 m × 6 m.

Q11. A contractor has to construct a building shown in the following figure. The building consists of two symmetrical towers having two basement and seven stories. The basements dimensions are 72 m × 66 m and 54 m × 42 m. The dimensions of the remaining seven stories are 48 m × 38 m. The concrete work (formwork + reinforcement + concreting) is scheduled to be completed in 10 months time.

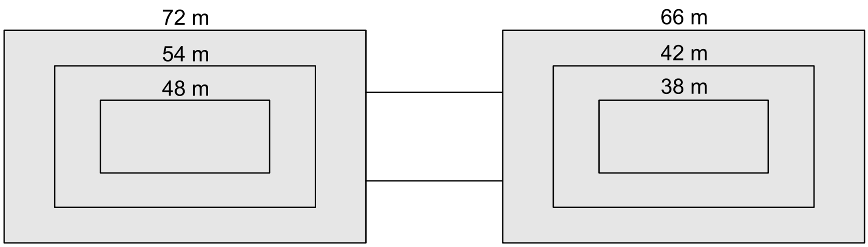


Figure Q 16.11.1 Building Plan for Question 11

Floor to floor height is 3.5 m. All beams are of uniform size: 300 mm width and 450 mm depth. The area for the beam formwork can be assumed to be 30% of the plan area. All columns are square columns of 450 mm. The lower basement has 142 columns and the upper basement has 135 columns. In the remaining seven stories there are 80 columns.

Calculate the formwork area to be mobilized each of column, beam, and slab formwork to complete the entire concrete work in the given duration.

Q12. Estimate the cost per m^2 of the slab formwork system shown in the following figure.

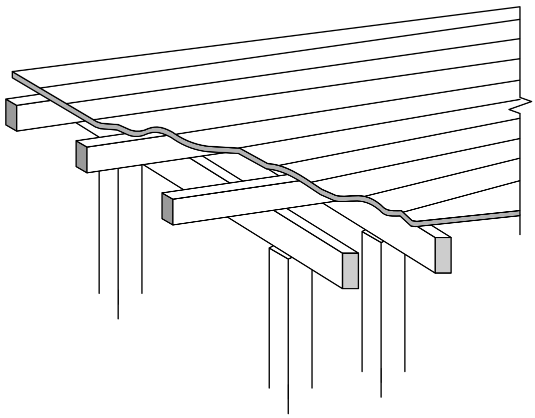


Figure Q 16.12.1 Slab Formwork for Question 12

Q13. Develop thumb rule (norms) for calculating steel requirement for the Doka wall formwork. The norms so developed should be useful for preliminary planning. For the estimation, you may consider a shutter size of 2.4 m × 6.0 m. A sample sheet for estimation is provided in Table 16.13.1.

Table Q 16.13.1 Sample Sheet for Calculating Steel Requirement (Shutter Size of 2.4 m x 6.0 m)

Sl. No.	Item	Nos./length	Unit weight	Total weight
1.	Steel waler	2 nos. 6 m length		
2.	Alignment props (size 3 m)			
3.	Alignment props (size 2m)			
4.	Foot adaptor assembly			
5.	Head adaptor assembly			
6.	Bracket			
7.	Lifting device			
8.	Hand rails			
	Grand total			X

$$\text{Weight per m}^2 = \frac{X}{2.4 \times 6}$$

Q.14. Develop thumb rule (norms) for calculating timber requirement for a conventional wall formwork shutter shown in Fig Q 16.14.1. The norms so developed should be useful for preliminary planning. For the estimation, you may consider a shutter size of 2.4 m × 6.0 m. A sample sheet for estimation is provided in Table Q 16.14.1.

Table Q 16.14.1 Sample Sheet for Calculating Timber Requirement (2.4 m × 6 m)

Sl. No.	Item	Nos./length	Unit volume m^3	Total volume m^3
1.	Timber 50.8 mm × 101.6 mm @ 150 c/c			
2.	Timber 101.6 mm × 101.6 mm (2 nos.) @ 450 c/c			
3.	Timber 152.4 mm × 101.6 mm @ 450 c/c			
4.	--------			
5.	--------			
	Grand total			X

$$\text{Volume per m}^2 \text{ in cubic meter} = \frac{X}{2.4 \times 6}$$

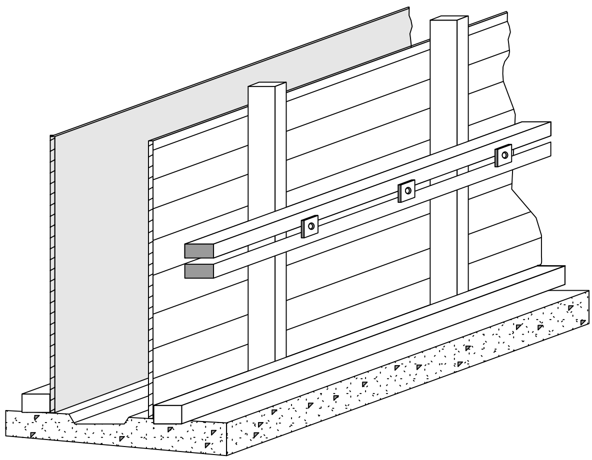


Figure Q 16.14.1 Typical Wall Form with Timber Sheathing.

Q.15 Develop thumb rule (norms) for calculating H-16 requirement for the Doka wall formwork. The norms so developed should be useful for preliminary planning. For the estimation, you may consider a shutter size of 2.4 m × 6.0 m. A sample sheet for estimation is provided in Table Q16.15.1.

Table Q 16.15.1 Sample sheet for calculating H-16 requirement (2.4 m × 6 m)

Sl. No.	Item	Nos./length	Total length m
1	H-16 @ 300 c/c	(6/0.3 + 1)nos. of 2.4 m length	
2	H-16	3 Nos. 6m length	
	Grand total		X

$$\text{H-16 beam in } m \text{ per } m^2 = \frac{X}{2.4 \times 6}$$

Chapter

17

Formwork Failure

Contents: Introduction; Causes of Formwork Failure; Common Deficiencies in Design Leading to Formwork Failure; Case Studies in Formwork Failure; Avoiding Formwork Failure; Recommendations on Safe Practices; Some Suggested Checklists

17.1 INTRODUCTION

Formwork activities constitute an important part of any construction work. Some form of formwork construction is involved in nearly every construction project. Formwork activities are associated with relatively high frequency of disabling injuries and illness. Huang and Hinze (2003) analyzed the OSHA accident reports for 1997 and reported that 5.83% of the falls were attributed to the construction of formwork or the construction of temporary structures, and 21.2% of all struck by accidents involved wood framing or formwork construction.

Ergonomic studies by Har (2002) also suggested that the repetitive activities of lifting, sawing, and hammering commonly performed by the formwork carpenters led to a high frequency of low-severity injuries such as discomfort and persistent pain.

Hallowell and Gambatese (2009) conducted an interesting study in which they identified a total of 13 operations involved with formwork activities. Based on the observations of 11 workers involved with the formwork activities, the exposure of workers in terms of percentage on an 8-h work day was found out.

It may be observed that most formwork failures are not catastrophic and can be detected during concrete placing by vigilant supervisors. When the failure occurs, the concreting is stopped or slowed. Extra bracing may be installed and some remedial action (sawing or clipping) may be required later on the cured concrete. Some signs to look for during the pouring operation to avoid failures are: (1) groaning, creaking and popping noises heard during the pouring, (2) excessive deflection of forms, and (3) concrete quantities not yielding proper coverage.

As it is, very few researches have been conducted in the area of formwork safety and on top of that very few researchers have identified the worker activities required to complete the formwork process. In that sense, the effort of Hallowell and Gambatese (2009) is noteworthy. They created a comprehensive list of activities which became so vital in understanding the risks involved in the formwork activities.

During the observation phase, 11 worker activities were identified and described. The activities and their descriptions were sent to industry professionals for their review and validation. During this review, the activity descriptions were refined and two additional activities were identified. Table

17.1 summarizes the findings of the observation phase of the study and includes activity names, descriptions, and approximate exposure. One should note that the risks defined for the activities described in Table 17.1 apply only to activities that are performed as described.

The exposure values in Table 17.1 represent the average percentage of time spent performing a specific activity during an 8-h work day for the four projects observed. One will note that on an average, 86% of the time was consumed on the 13 listed activities, while 14% of the workers' time was consumed on non-productive activities such as resting, eating, or talking about non-work related topics. Since the researchers calculated the exposures using only four projects, the information presented here should be used for reference only and should not be considered as being representative of the industry as a whole.

Table 17.1 Activity Descriptions and Approximate Exposure (Percentage of an 8-h Work Day)

Sl. No.	Activity name	Description	Approximate exposure
1.	Ascend/Descend ladder	Operations that occur above or below grade typically require workers to ascend or descend the ladders to reach the work site. Ladders may be wooden, metal, or fiberglass and may vary in length from site to site. In many cases, workers may climb up the formwork supports instead of using a ladder.	3%
2.	Static Lift	Workers are often required to temporarily support a portion of the concrete form, while other workers connect materials or components. This activity involves a static lift and may be accompanied by lifting/lowering.	6%
3.	Nail/screw/drill	Nailing or screwing the form components or materials may involve the use of a hammer (typically larger than 0.57 kg), nail gun, electric screwdriver, impact wrench, or staple gun. The worker may be required to repeat this activity for an extended period of time at certain stages of construction.	8%
4.	Motorized transport	Materials may be transported by vehicles such as trucks, skids steers, forklifts, cranes, or scissor lifts when the equipment is readily available or when the site is relatively large.	2%
5.	Crane Materials	When a crane is used to transport materials or form components, workers must accept the materials from the crane and/or load the crane with excess materials or waste. Workers must direct the crane operator as the material is lifted or lowered and they may be required to manually guide the load.	5%
6.	Cut Materials	During most formwork operations, materials such as 50 mm × 100 mm timber plywood, or aluminum must be cut to size. Typically, equipments such as circulating saw or reciprocating or table saw is used to cut materials and the worker operates such equipments and guides materials during cutting/ripping.	9%

Sl. No.	Activity name	Description	Approximate exposure
7.	Inspect/plan	During construction, the workers and crew leaders often take time to inspect their work and plan for subsequent operations or inspect prior work.	16%
8.	Lift/lower materials	Lifting and lowering the materials or equipment involves unassisted vertical transport of construction materials, formwork components, or equipment. The process of forming concrete may require that workers lift materials or concrete from foot-level to a higher or lower grade.	8%
9.	Manual transport	Manual transport may include transporting the equipment and materials of varying weights such as 50 mm × 100 mm timber plywood, form panels, ties, cat heads, and adjustable pipe braces, from one location to another.	11%
10.	Hammer materials	This activity is different from nailing components and materials because heavier tools such as a sledgehammer may be necessary to drive large objects. Such an activity typically requires fewer strikes of larger force than standard nailing.	6%
11.	Plumb/level forms	Leveling and plumbing of forms involves using bodyweight, pry bars or other equipment to shift and adjust the formwork. A screw jack may be used for this activity and some workers may be used for surveying or using hand levels, lasers, or plumb bobs to ensure proper placement.	6%
12.	Excavation	Excavation involves the removal of soil or other materials to access areas below grade. This activity typically involves the use of heavy equipment such as a backhoe or a bulldozer.	2%
13.	Lubrication/preparation	Form lubrication and preparation involves spraying of form oil and/or the curing compound and setting and wetting of curing blankets and expansion materials.	4%

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Using a total of 256 worker-hours (w-h) of field observation, a preliminary primary list of worker activities and the corresponding descriptions was created. This preliminary list was reviewed, augmented, and validated by a group of 8 individuals with an average of approximately 20 years of experience. The result was a final list of 13 distinct and well-defined activities. The safety risk score associated with these activities are given in Table 17.2.

Table 17.2 Comparison of Risk Values among Formwork Construction Activities

Sl. No.	Formwork construction activity	Safety risk score (S/w-h)
1.	Lubrication /Preparation	18.67
2.	Ascend/Descend Ladder	1.86
3.	Crane Materials	0.51
4.	Motorized Transport	0.48

Sl. No.	Formwork construction activity	Safety risk score (S/w-h)
5.	Hammer Materials	0.25
6.	Lift/lower Materials	0.19
7.	Excavation	0.11
8.	Plumb/Level Forms	0.11
9.	Cut Materials	0.05
10.	Manual Transport	0.04
11.	Nail/Screw/drill	0.03
12.	Static Lift	0.03
13.	Inspect/Plan	0.01
	Total	**22.63**

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17.2 CAUSES OF FORMWORK FAILURE

Collapse of the formwork can cause loss of life and serious injuries to crew members, supervisors, and even to third parties. This may also cause property damage, construction delays, and loss of morale of the crew members.

Formwork may fail due to many reasons. The failure primarily occurs at the time of placing of concrete. Some unexpected event may cause one formwork member to fail causing overloading and misalignment of another formwork member ultimately leading to the collapse of the entire formwork system. According to Mosallam and Chen (1992), in general, the primary causes of formwork disasters are: (1) excessive loads, (2) premature removal of forms or shores, and (3) inadequate lateral support for the shoring members.

Some leading causes of formwork failures are discussed in the following sections:

17.2.1 Improper Stripping and Shore Removal

Premature stripping of forms, premature removal of shores, and careless practices in reshoring can produce catastrophic results.

Figure 17.1 shows the partly collapsed floors of a multi-story building. A portion of the fourteenth floor of the tower collapsed due to the collapse of the shoring provided to support the concrete. It appears that the shoring was weak and the reshores were missing. The collapse of this floor resulted into impact loading of the floor directly below it which also failed and collapsed to the lower floor. This has a domino effect and by the impact load from the weight of upper floors debris, lower floors also collapsed subsequently all in a span of less than an hour.

17.2.2 Inadequate Lateral Bracing—Wind, Construction Loads

The more frequent causes of formwork failure, however, are other effects that induce lateral force components or induce displacement of the supporting members. Inadequate cross bracing and horizontal bracing of shores is one of the factors most frequently involved in formwork accidents. Investigations prove that many accidents causing damage worth thousands of rupees could have been prevented only if a few hundred rupees had been spent on diagonal bracing for formwork support. High shoring with heavy load at the top is vulnerable to eccentric or lateral loading.

Diagonal bracing improves the stability of such a structure, as do guys or struts to solid ground or completed structures.

Figure 17.1 Collapse of One Floor Leading to Another.

When a failure occurs in one part, inadequate bracing may permit the collapse to extend to a large portion of the structure and multiply the damage. Suppose a worker accidentally rams a wheelbarrow into some vertical shores and dislodges a couple of them. This may set up a chain sort of reaction that brings down the entire floor. One major objective of bracing is to prevent such a minor accident or failure from becoming a disaster.

The collapse of a segment of *cast-in-situ* waffle slab is shown in Figs. 17.2 and 17.3. The collapse occurred when the concrete pour was about to be completed. Investigation showed that some of the bracings had been removed by the formwork crew for preparing the formwork for another slab segment. The load carrying capacity of the shores was found adequate and it was possibly due to the removal of the bracing only, that resulted into the collapse of the floor segment.

In Fig. 17.4, the bent shores are clearly visible. This may be corresponding to the stage where some of the shores might have given way and the load share of the failed shore are transferred to the shores seen in the picture.

Figure 17.2 View of Collapsed Waffle Slab Segment.

Figure 17.3 Another View of Collapsed Waffle Slab Segment.

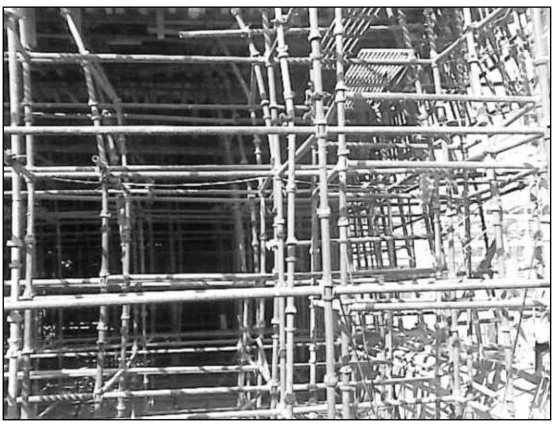

Figure 17.4 View of the Bent Shoring (Due to Excess Load).

17.2.3 Vibration Due to Concrete Placing Equipment

Forms sometimes collapse when their supporting shores or jacks are displaced by the vibration caused by passing traffic, the movement of the workers and the equipment on the formwork, and the effect of vibrating concrete to consolidate it. Diagonal bracing can help prevent failure due to vibration.

Some failures have taken place when the concrete pipe line (used for pouring the concrete) has been supported on the shoring towers without the designers being informed of such an arrangement. The pipe line induces vibration to the shoring towers for which they may not be designed. The vibration sometimes cause struts or supports to loosen leading to the failure of the support system.

17.2.4 Unstable Soils under Mudsills, Shoring not Plumb

Unstable soils under the mudsills can also cause the formwork to fail. The mudsills act as a base for a shore or post in formwork. The mudsills could be a timber plank, a frame, a small footing or pedestals.

Formwork should be safe if it is adequately braced and constructed; so all loads are carried to the solid ground through vertical members. Shores must be set plumb and the ground must be able to carry the load without settling. Shores and mudsills must not rest on the frozen ground; moisture and heat from the concreting operations, or changing air temperatures, may thaw the soil and allow settlement that overloads or shifts the formwork. Site drainage must be adequate to prevent a washout of soil supporting the mudsills.

In Figs. 17.5-17.7, the failure of scaffolding and the deck formwork between the two piers is shown. The failure happened primarily because the shoring for the deck slab gave way. The accident resulted in a large number of major injuries to the workers besides a few casualties. The ground on which the shores were resting, subsided leading to the failure.

Figure 17.5 Collapse of Deck Slab between the Two Piers.

Figure 17.6 Another View of the Accident Site.

Figure 17.7 Shoring Tower in Position for the Deck Slab Adjacent to the Collapsed Slab.

17.2.5 Concrete Placing Techniques— Overloading in One Area/ Inadequate Control of Concrete

The temperature and the rate of vertical placement of concrete are the factors influencing the development of lateral pressure that acts on the forms. If temperature drops during the construction operations, rate of concreting often has to be slowed down to prevent the build-up of lateral pressure overloading the forms. If this is not done, formwork failure may happen. Failure to properly regulate the rate and order of placing concrete on the horizontal surfaces or curved roofs may produce unbalanced loadings and consequent failures of the formwork.

Figures 17.8-17.10 show different views of formwork failure primarily due to overloading of the formwork. During the concrete pouring operation, a large amount of concrete was dumped at a single location. This led to the failure. The buckled (bent) props on account of heavy loading are clearly visible in Figs. 17.8-17.10. The damaged H-16 beams that were used as primary and secondary beams can also be seen in Fig. 17.10.

Figure 17.8 View of the Portion of the Slab which Failed due to Concrete Overloading.

17.2.6 Lack of Attention to Formwork Details

Even when the basic formwork design is soundly conceived, small differences in assembly details may cause local weakness or overstress leading to form failure. This may be as simple as insufficient nailing, or failure to tighten the locking devices on metal shoring. Other details that may cause failure are: inadequate provisions to prevent the rotation of beam forms where slabs frame into them on the side; inadequate anchorage against uplift for sloping form faces; lack of bracing or tying of corners, bulkheads, or other places where unequal pressure is found. Figures 17.11 and 17.12 show another instance of formwork failure due to the above mentioned reasons.

Figure 17.9 Another View of the Portion of the Slab which Failed due to Concrete Overloading (Note the Buckled Props).

Figure 17.10 Another View of the Portion of the Slab which Failed due to Concrete Overloading.

Figure 17.11 View of Portion of Slab and Beam Formwork which Collapsed.

Figure 17.12 Another View of Portion of Slab and Beam Formwork which Collapsed.

17.3 COMMON DEFICIENCIES IN DESIGN LEADING TO FORMWORK FAILURE

The following common design deficiencies can also lead or contribute to formwork failure.

- Lack of allowance in design for such loadings as wind, power buggies placing equipment and temporary material storage;

- Failure to account for loads imposed on anchorages during gap closure in aligning formwork;
- Insufficient allowance for eccentric loading due to placement sequence;
- Inadequate anchorage against uplift due to battered form faces;
- Failure to investigate bearing stresses in members in contact with shores and struts;
- Failure to provide proper lateral bracing or lacing of shoring;
- Failure to investigate the slenderness ratio of compression members;
- Inadequate provisions to tie the corners of intersecting cantilevered forms together;
- Inadequate reshoring; and
- Overstressed reshoring.

Hadipriono and Wang (1986) examined the causes that resulted in 85 major falsework failures over the past 23 years. They identified three causes of failure: triggering, enabling, and procedural causes (Fig. 17.13). In Fig. 17.13, the number of occurrences against each of the causes is given in brackets. Most failures occurred because of the interaction of the triggering and enabling events that were, in many cases, produced by inadequacies in the procedural methods. Their results also revealed that approximately 72% of the failures occurred during concrete placement. The period of concrete placement seems to be the most critical time in the entire life of a building.

Inadequate falsework cross-bracing or placing was the primary source of several falsework accidents, such as the collapses of the Arroyo Seco Bridge in California, the Skyline Center Complex in Virginia, the high-way ramp in East Chicago, and the Coliseum in New York.

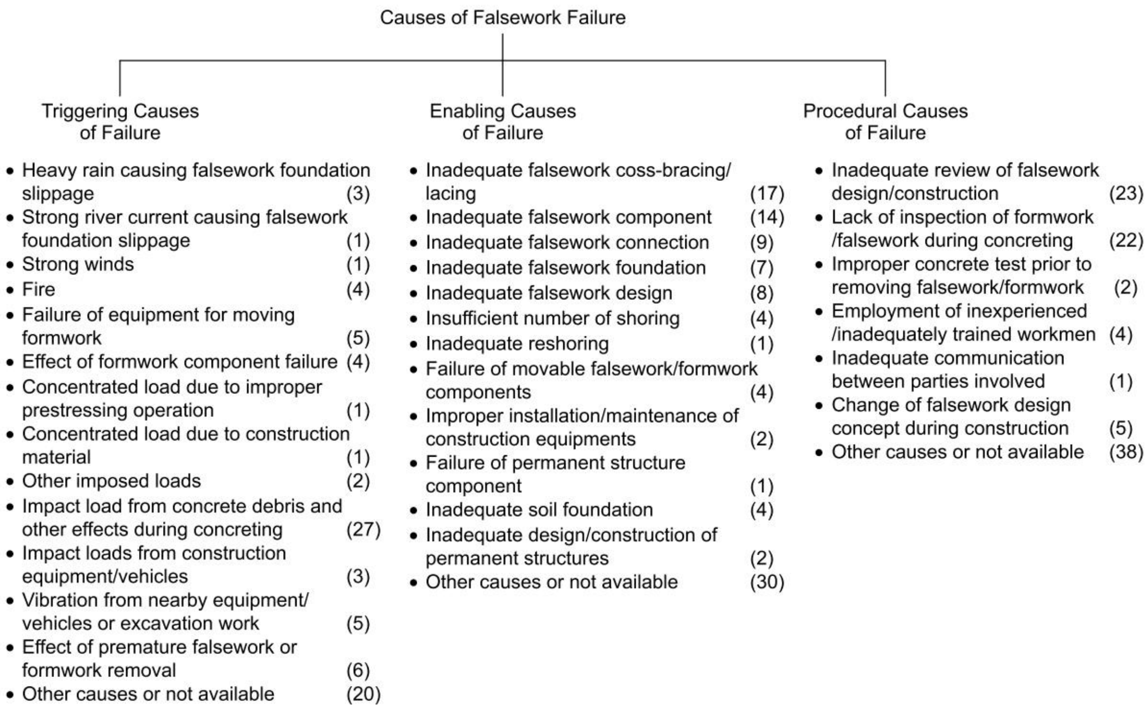

Figure 17.13 Summary of Causes of Formwork Failure (Adapted from Hadipriono and Wang, 1986).

17.4 CASE STUDIES IN FORMWORK FAILURE

The newspaper report for few days would reveal a number of stories on the accidents caused due to formwork/scaffolding/temporary structure failure. These accidents are described by newspapers by headings such as "roof collapse of under construction building", "scaffolding collapse", "formwork collapse", "shuttering collapse", etc. BBC world news quoting UN's International Labor Organization reports that nearly 50,000 Indians die from work-related accidents or illnesses every year. As described earlier, a large chunk of these accidents are caused due to the failure of temporary structures— more specifically the formwork. While carrying the story on the roof collapse of an under construction building, the BBC correspondent writes "India is going through a real estate boom, but there are few safety precautions for workers at the construction sites, and incidents such as wednesday's building collapse are fairly common". This is too insulting for a country which boasts of producing such a large number of engineers and is moving towards becoming a developed country in the near future. Table 17.3 is a compilation of a few randomly selected accidents caused due to formwork and scaffolding collapse. The details of these accidents can be read by following the links given in the 'reference' column of Table 17.3. Some of these accidents result into severe injuries, while some of them result into a number of deaths.

As can be observed from the table, the collapses are reported from different parts of the country every now and then. These collapses occur both in individual house construction using small time contractors and large public projects involving large reputed contractors. The collapses are reported from individual residential houses, multi-story residential towers, school buildings, factory buildings, etc. The collapses have occurred at different point of time such as— at the time of concreting the slab, after the concreting for the slab is over, at the time of removal of shores, and so on.

One of the common events after such accidents is to constitute an enquiry committee. What happens after that is rarely known to the common man. The causes of accidents and the remedial measures are not publicized, with the result that similar accidents repeat frequently. The author is aware of a number of cases where within the same organization, similar types of accidents have occurred repeatedly. This is because people don't like to communicate the accidents, their causes, and the remedial measures. The author is yet to find a news report carrying the causes of the past accident reported by the same newspaper and the recommendations of the expert committee.

The review of failures mentioned in Table 17.3 points out the following as the frequent causes of failures:

- Early shore removal;
- Inadequate reshoring arrangement;
- Rapid delivery of concrete introducing lateral forces at the top of high shoring;
- Inadequate diagonal bracing etc.

17.5 AVOIDING FORMWORK FAILURE

As formwork failure can have catastrophic results, some of the ways to avoid such failures are outlined in the following section.

Table 17.3 Scaffolding Collapse, Form Failure, Roof Collapse, Shuttering Collapse in INDIA

Sl.	Category	Location	Project	Date	Causalities	Reference
1.	Formwork failure	Thiruvanathapuram, Kerala	Construction in VSSC	1 April 2009	1 killed + 34 injured	http://www.thehindu.com/2009/04/01/stories/2009040162050300.htm
2.	Scaffolding collapse	Mumbai, Maharashtra	Mumbai Metro One	15 July 2009	No injuries	http://www.expressindia.com/latest-news/scaffolding-collapses-at-chakala-metro-site/489464/ http://www.highbeam.com/doc/1G1-203806829.html
3.	Scaffolding collapse	Meenambakkam, Chennai, Tamil nadu	Cargo extension, Airport	21 March 2007	3 injured	http://www.hindu.com/2007/03/21/stories/2007032110980100.htm
4.	Scaffolding collapse	Baltana, Zirakpur, Chandigarh	House construction	20 December 2008	2 injured	http://www.tribuneindia.com/2008/20081221/cth2.htm
5.	Iron scaffolding collapse	Hyderabad	Flyover construction	9 September 2007	4 dead, 10 injured	http://www.expressindia.com/news/fullstory.php?newsid=91971 http://www.thehindu.com/2007/09/10/stories/2007091057640100.htm
6.	Scaffolding collapse	Dalmiapuram, Tamil Nadu	Cement factory under construction	29 May, 2005	12 dead, 21 injured	http://www.abc.net.au/news/stories/2005/05/29/1379331.htm
7.	Roof collapse	Whitefield, Bangalore	15 storied building under construction	24 October 2008	4 injured	http://indiahousingbubble.blogspot.com/2008/10/prestige-shanitniketan-roof-collapse.html
8.	Roof collapse	Babausabpalya, Bangalore	Under construction house	29 July 2002	6 injured	http://www.hinduonnet.com/2002/07/29/stories/2002072908010300.htm
9.	Roof collapse	Kurnool district, Andhra Pradesh	House	26 August 2007	1 dead	http://www.hindu.com/2007/08/26/stories/2007082657840300.htm
10.	Roof collapse	Lalru, Chandigarh	School building	15 January 2008	No injuries	http://www.tribuneindia.com/2008/20080116/cth3.htm
11.	Chimney collapse	Chhattisgarh	Under construction power plant	23 September 2009	22 dead	http://news.bbc.co.uk/2/hi/south_asia/8271423.stm
12.	Roof collapse	Jharkhand, Ranchi	School building	10 February 2008	2 dead	http://www.indiaenews.com/india/20080210/96850.htm
13.	Roof collapse	Katni, Madhya Pradesh	Under construction 3 story house	18 September 2009	6 dead	http://visionmp.com/six-killed-in-katni-roof-collapse25411479154/
14.	Shuttering collapse	Faridabad, Delhi	Factory building	12 October 2009	1 dead	http://timesofindia.indiatimes.com/city/delhi/1-die-in-Faridabad-factory-collapse/articleshow/5113078.cms
15.	Shuttering collapse	Kaithal, Chandigarh	Under construction house	15 March 2004	1 dead, 5 injured	http://www.tribuneindia.com/2004/20040316/haryana.htm
16.	Shuttering collapse	Biocon Park, Bangalore	'Biocon Park' construction	7 December 2007	3 dead	http://www.business-standard.com/india/news/biocon-park-building-wall-collapses-three-die/267232/
17.	Shuttering collapse	Rohtak, Haryana	House construction	10 January 2006	2 dead	http://www.tribuneindia.com/2006/20060111/haryana.htm

Proper design and system

All formworks must first be designed even if the design is approximate. Accounting for all the loads that are likely to act on the formwork can ensure safety. The load calculations should include all lateral loads acting on the formwork. The aspect of overturning, if any, must also be considered.

Qualified staff

A properly qualified falsework/formwork coordinator must be appointed for the formwork operations. If any proprietary system is being implemented, guidance from experts in the early years always helps in a better and safe implementation.

Training

Short professional courses should be arranged. Practical training courses on formwork ensure proper implementation of the formwork. Keeping record of the engineers and workmen who have undergone formwork training must also be maintained so that their services can be utilized whenever required.

Reporting

In spite of the above measures, if any collapse occurs, it should be reported and thoroughly investigated by the experts. The causes of the failure must be established. Finally, the recommendation to prevent any future failure must also be suggested by the experts and the report should be given wide publicity so that similar failures are not repeated in future.

17.6 RECOMMENDATIONS ON SAFE PRACTICES

While erecting or dismantling the formwork, it is essential to ensure that the structure is stable and safe at every stage and is adequately braced, temporarily strengthened if necessary, to withstand loads like wind. This should be kept in mind while deciding on the sequence of erection or dismantling.

The accessories should be properly fastened at every stage and there should not be loose materials lying around the place of work. They can fall from heights endangering the safety of men.

Formwork should be continuously watched during and after concreting by a competent person. It is advisable to watch for loosening of nut washers and wedges during vibration.

Rate of rise of concrete in the forms should not exceed that for which they are designed. Limits set by the designer on vibration should also be followed. Reasonable care by the operator is necessary to avoid scarring or roughening of the forms by operating vibrators against them.

The access walkways and working platforms should be sufficiently wide and with adequate safety provisions like proper toe boards and hand rails.

While deshuttering or dismantling the formwork, apart from considering the safety of formwork and staging, the safety of the concrete structure itself should be kept in mind. The removal of form should permit the concrete to take its load gradually and uniformly without any impact or shock.

Improper sequence of stripping can lead to sudden dropping of form components causing injury to work men and also causing damage to the formwork material reducing its reuse value.

OSHA (Occupational Safety and Health Administration) regulations, ACI recommendations, and local code requirements for formwork should be followed. Some of the recommendations are given below:

1. Competent supervision during the erection of formwork;
2. Inspection during concrete placing— tightening wedges, looking for problems;
3. Safe work platforms and access for placing crew;
4. Control of concrete placing;
5. Improving soil quality and bearing capacity;
6. Good design practice and inspection by the designer prior to concrete pouring;
7. Checking shoring and reshoring;
8. Ensuring healthy relationship among the architect, engineer and the contractor;
9. Maintaining and coordinating tolerances; and
10. Preparing a formwork specification.

According to Fuhr and Huston (2000), there were 85 collapses during construction resulting from formwork failure in the last quarter of the 20th century. Formwork failure can result from judgmental errors in design and assembly and deviation from recommended practices. Formwork failure can include: (a) collapses that are not exactly publicized, (b) excessive deflection not documented off the project site, (c) poor quality of formwork often requiring rework involving additional time and money, and (d) structural failures involving overload on account of excessive pressure at the time of failure.

17.7 SOME SUGGESTED CHECKLISTS

The author conducted safety audit of all the 12 packages (consisting of about 500 km) of the Lucknow Muzaffarpur National Highway Project. The safety audit was performed in two stages: review of document and review of field compliance. The review was mainly with respect to the provisions of the contract and the applicable law of the land.

The findings of the safety audit of temporary structures are given in the following sections. Some of the common observations noted in the safety audit of this project can be noticed in other projects as well. It is thus hoped that the following discussions containing common observations, supporting photographs, and recommendations would help the practitioners in avoiding the bad practices and adopting the good ones.

Observation 1

Workers and supervisors without appropriate personal protective equipment (PPE)

It is very common to find complete disregard for personal protective equipment such as hard hat (helmet), safety shoes, safety belts, hand gloves, goggles, ear plugs, both by the workers and by their supervisors. In some instances, even the client and the consultant representatives do not pay proper attention to these life saving gadgets. Figures 17.14-17.16 show some instances where the desired personal protective equipments have not been used by the workers and supervisors.

Figure 17.14 Supervisors without Appropriate PPE.

Figure 17.15 Formwork and Reinforcement Crew without Appropriate PPE.

Figure 17.16 Senior Officials of Contractor, Client, and Consultants without Appropriate PPE.

Observation 2

Substandard arrangement for access/approach

In a number of instances it is found that the contractors do not pay attention to the access/approach to the workers. These are found to be substandard, makeshift type and unsafe. The access/approach is not properly secured and sometimes large gaps exist in between the two elements used to form the ramps. Wherever ladders are used, they are not on firm grounds, slopes are not proper, and sometimes adequate heights are not available. This not only creates unsafe work conditions, the productivity is also compromised. The ladders used are of substandard material and they are also not placed properly. Walkway jallies are not proper. There is no provision of handrails. Figures 17.17-17.25 show all the unsafe accesses/approaches that should never be allowed for any kind of construction.

Figure 17.17 Unsafe Ramp.

Figure 17.18 Makeshift Ramp in Position.

Figure 17.19 View of a Makeshift Ramp.

Figure 17.20 Another Type of Makeshift Ramp.

Figure 17.21 Makeshift Ladder of Inadequate Height and Unsafe.

Figure 17.22 Ladder not Properly Rested.

Figure 17.23 Makeshift Ladder Near Edge of Unstable Ground.

Figure 17.24 Ladder Resting on Pipes which in Turn is Supported on Reinforcement Inserted into Weepholes of the Concrete Wall.

Figure 17.25 Makeshift Walkway jallies Made up of Scrap Reinforcement.

Observation 3

Safety nets not properly installed

Although in some cases, safety nets were in use wherever work at heights was being carried out, these were not fixed properly. The improper use of nets is as good as not having the safety nets at all. An improper safety net installed is shown in Fig. 17.26.

Figure 17.26 Safety Net not Properly Installed.

Observation 4

Shuttering plates used to form ramp for access

At most of the work sites, the ramps for access are prepared using shuttering plates. These plates are reversed (Figs. 17.27 and 17.28) and used for transporting men and materials to heights. Sometimes large gaps are also noticed between the plates (Fig. 17.28) which is a hazard.

Figure 17.27 Access Ramps Made out of Shuttering Plates.

Figure 17.28 Makeshift Ramp Along with Dangerous Gaps (Openings) in between.

Observation 5

Shuttering plates used to form working platforms

At some locations the shutter plates are used as brackets to support the formwork crews and the materials used by them. The bolts used to fix the shutter plates with concrete are not proper. This is extremely hazardous. Figure 17.29 shows this hazardous condition.

Observation 6

Shuttering plates used as pedestals to support scaffolding pipes

Use of shutter plates as mudsills is noticed at a few locations. These plates are placed at loose soils prone to erosion. This may cause uneven settlement of shores leading to failure of the formwork system. Many accidents have happened in past due to such uneven settlements. Figure 17.30 shows such an arrangement made at one of the project sites.

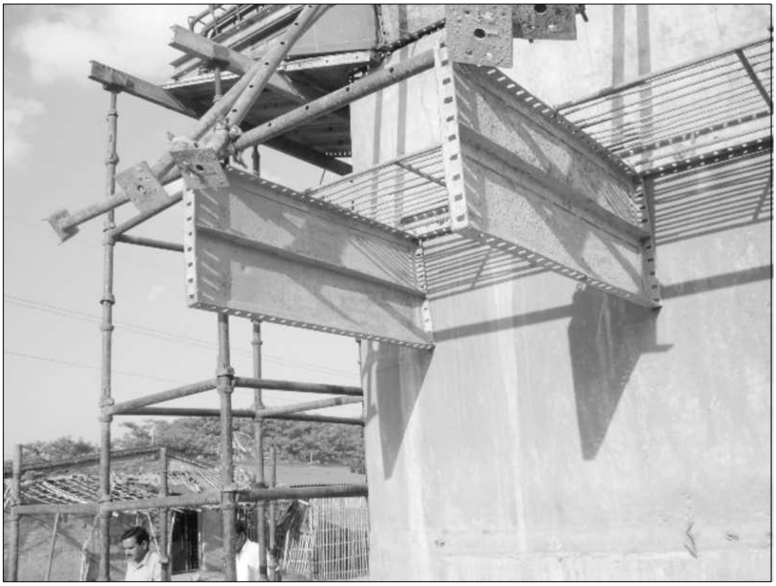

Figure 17.29 Shuttering Plates Being Used to Form Brackets to Support Working Platform.

Figure 17.30 Shuttering Plates Being Used as Pedestal (Even Worse, They Are Kept on Loose Soil).

Observation 7

Proper attention not paid to the design and installation of bracings

Reinforcement bars tied with binding wires are sometimes used as diagonal bracings. Figures 17.31 and 17.32 show the reinforcement bars tied with binding wires being used as horizontal and diagonal bracings. Obviously this arrangement of bracings is not adequate and should not be used.

Figure 17.31 Reinforcement Bars Tied with Binding Wire Used as Horizontal Bracing.

Figure 17.32 Reinforcement Bars Tied With Binding Wires Being Used as Diagonal Bracings.

Observation 8

Shoring towers not placed in level on the concrete pedestal

There is no connection between the cribs and the concrete pedestals. Where scaffolding frames are used, they are without footplates and sometimes not on a level base. Figure 17.33 shows such an arrangement at one of the project sites.

Figure 17.33 Foot Plates not in Proper Contact with Concrete Pedestals.

Observation 9

Work zones not properly barricaded

The work zones are not properly barricaded and the entry of unauthorized persons not connected with the work being performed, is not restricted. Figure 17.34 shows a woman picking up the waste concrete from the work site. Figure 17.35 shows children at the edge of excavation.

Figure 17.34 Unauthorized Entry not Restricted.

Observation 10

Working platforms supported on the ledger pipes passing through the weepholes left in the concrete wall

Figure 17.36 shows such an unsafe arrangement practiced at one of the project sites. In Fig. 17.37, no arrangement of working platforms has been made and workers are seen tying the reinforcement for the wall.

Figure 17.35 Children at the Edge of Excavation. No Barricades.

Figure 17.36 Scaffolding Pipes Are Erected on the Weepholes Made in the RCC.

Figure 17.37 No PPEs. No Platform. No Access/ Approach. No Display of Caution Signs.

Observation 11

Working platforms not proper

The provision of handrails and the arrangement to tie the safety belts are missing at most of the places. The working platforms are not proper.

Figure 17.38 shows the prevailing gaps between the boards. There is no provision of guard rails, knee rails, and toe board above the platform. The boards are not properly secured. Also, the materials stored on the platforms are not properly secured. Further, there is no provision of anchoring safety belts. Figures 17.39 and 17.40 also show similar deficiencies and unsafe practices.

Figure 17.38 Improper Working Platform, No Hand Rails, No Foot Guards.

Figure 17.39 Workers on the Unstable and Weak Working Platform.

Figure 17.40 No Provision of Handrails and Lifeline Support.

Observation 12

Soil not properly compacted

The soil on which the cribs/trestles/scaffolding frames are resting, are not properly compacted and erosion of soil is noticed. Figure 17.41 shows such a practice being adopted at one of the project sites.

Figure 17.41 Erosion of Soil Beneath the Shoring Was Noticed.

Observation 13

The materials used for scaffolding and formwork are sometimes defective.

Observation 14

The approved design and drawings missing

The approved design and drawings are absent in some of the packages. The work in some of the cases is being carried out on advanced copies and provisionally approved copies of the drawings. Sometimes, deviations are noted in the implementation of the formwork from the approved drawings. It appears that the lifting tool and tackles are not inspected, tested, and checked before use. Although the contract mentions vetting of the temporary structure drawings by a third party, in some cases, they are found not implementing this task. Work was performed without trained workmen and supervisor. There was no system of maintenance and checking of form materials before putting them into use.

Based on the observations of the audit of the above projects, the audit team developed a checklist for the works pertaining to the formwork, access/approach, etc. The checklists are given in Tables 17.4-17.8. It contains the description of the checklist points. In order to implement the checklist, the compliance in terms of 'yes' or 'no' are to be entered in the compliance column. Any remarks corresponding to a given checklist point can be entered under the remarks column. Each one of the 'no' entries in the 'compliance' column needs careful consideration before proceeding to the next stage of work. It is hoped that the checklist would be useful to the practitioners.

17.7.1 Checklist for Ladder/Stairways/Ramps

Table 17.4 Checklist for Ladder/Stairways/Ramps

1.0	Ladder/Stairways/Ramps	Compliance	Remarks
1.1	Extension and straight ladders extend 1 m beyond landing?		
1.2	Are ladders placed on level ground?		
1.3	Are ladders positioned at an angle of about 1:4?		
1.4	Are ladders adequately secured?		
1.5	Are ladders safe and inspected regularly?		
1.6	Are existing access ways (stairs, walkways, ladders) etc. left clear?		

17.7.2 Checklist for Scaffolds

Table 17.5 Checklist for Scaffolds

2.0	Scaffolds	Compliance	Remarks
2.1	Is the scaffolding arrangement designed by a competent person?		
2.2	Has it been checked and approved by a competent person?		
2.3	Is an experienced and trained person employed for the erection and dismantling of scaffolds?		

2.0	Scaffolds	Compliance	Remarks
2.4	Is the site engaging suitable/properly trained/experienced workmen for constructing/dismantling/shifting scaffolding works?		
2.5	Are base plates provided? Is the base away from all excavations, drain covers, manholes etc?		
2.6	Is the soil condition proper?		
2.7	Is the draining available for high scaffold?		
2.8	Scaffold in plumbed up (vertical tubes)		
2.9	Entry restriction into effected area		
2.10	Are diagonal bracings secured and are checked with missing elements?		
2.11	Are lock pins in place and secured?		
2.12	Is the scaffold erected in the firm ground?		
2.13	Is it free from defective components?		
2.14	Is scaffold tag system in use?		
2.15	Is there safe access to the working platform?		
2.16	Is the width of the working platform properly maintained?		
2.17	Is there a provision of anchoring full body harness-lanyards to be tied to the life line?		
2.18	Are openings in the working platform kept safely covered/fenced?		
2.19	Whether a competent person is employed to inspect the scaffold condition periodically?		
2.20	Is there a system of inspecting materials of scaffolds on each occasion before erection?		

17.7.3 Checklist for Working at Height

Table 17.6 Checklist for Working at Height

3.0	Working at Height	Compliance	Remarks
3.1	Area below the workplace barricaded, especially below hot-works.		
3.2	Workmen provided with bag/box to carry bolt, nuts and hand tools.		
3.3	Has the arrangement for fastening hand tools made?		
3.4	Fabricated make-shift arrangements are checked for quality and type of material welding, anchoring, etc.		
3.5	Does work at more than one elevation at the same segment restricted?		
3.6	Safety nets are in use and maintained clean where height is more than 6m.		
3.7	Covering of gaps in formwork and edge protection.		
3.8	Is entry restricted into the effected area during the life cycle of the activity.		

17.7.4 Checklist for Formwork — (Shuttering and Deshuttering)

Table 17.7 Checklist for Formwork — (Shuttering and Deshuttering)

4.0	Formwork – (Shuttering and Deshuttering)	Compliance	Remarks
4.1	Whether the approved design, drawing and specifications are available for the formwork system.		
4.2	Whether qualified and experienced supervisors are available.		
4.3	Whether formwork has been inspected by carpentry foreman prior to placing the concrete.		
4.4	Whether the work area has been cordoned of .		
4.5	Whether Signal Men have been deployed to stop unauthorized entry.		
4.6	Whether the provision of access, working platforms, life lines and safety belts are there.		
4.7	Whether proper arrangements have been made for lowering of materials.		
4.8	Whether tag lines have been provided for handling of heavy loads.		
4.9	Whether fall prevention/protection measures provided to all voids and exposed edges.		
4.10	Whether the formwork is firmly supported on the base plate, ground or supporting structure with good holding condition.		
4.11	Whether the working platform of adequate width (min 450 mm) has been provided.		
4.12	Whether appropriate PPE's i.e. safety shoes, helmet and safety belt etc. provided to the workmen.		

17.7.5 Checklist for Working Platform

Table 17.8 Checklist for Working Platform

5.0	Working Platform	Compliance	Remarks
5.1	Is the working platform closely boarded, i.e., no gaps between the boards?		
5.2	Is the working platform at least 450 mm wide?		
5.3	Is a guard-rail, knee-rail and toe board provided above the platform and securely fixed?		
5.4	Are all the materials stored on the platforms properly secured?		
5.5	Are openings in the working platform kept safely covered/fenced?		
5.6	Is there a provision of anchoring safety belts lanyards to be tied to guy ropes?		

REVIEW QUESTIONS

Q1. True or False

(a) Formwork activities are associated with relatively low frequency of disabling injuries and illness.

(b) Signs to look during the pouring operation to avoid failures are:
   (i) Groaning, creaking, and popping noises heard during the pouring.
   (ii) Excessive deflection of forms.
   (iii) Concrete quantities not yielding proper coverage.

(c) Primary causes of formwork disasters are:
   (i) Excessive loads
   (ii) Premature removal of forms or shores
   (iii) Inadequate lateral support for shoring members

(d) The three common causes of formwork failure are: triggering causes, enabling causes, and procedural causes of failure.

(e) Ways to avoid formwork failure are:
   (i) Proper design and system
   (ii) Qualified and trained Staff
   (iii) Reporting

(f) Frequent causes of failure of formwork:
   (i) Early shore removal
   (ii) Inadequate reshoring arrangement
   (iii) Rapid delivery of concrete introducing lateral forces at the top of high shoring
   (iv) Inadequate diagonal bracing

Q2. List out the various activities and the exposure of risk associated with formwork.

Q3. Do the comparative analysis of the risks associated with various construction activities and provide comments.

Q4. List out the various reasons for formwork failure.

Q5. List out the various deficiencies in design leading to formwork failure.

Q6. List out the various recommendations as per OSHA, ACI for safety in formwork.

Q7. List out the various checklists to ascertain safety during formwork/scaffold, etc.

Chapter

18 Formwork Issues in Multi-Story Building Construction

Contents: Introduction; Techniques in Multi-story RC Construction; Distribution of Loads on Shores and Slabs in Multi-story Structures-Simplified Analysis; Load Distribution for Slabs and Shores in One, Two, Three, and Four Levels of Shores; Load Distribution for Slabs and Shores in Two Levels of Shores and One Level of Reshores; Limitations of Simplified Analysis and Discussion on Other Developments; Computation of Strength of Concrete Slab at a Given Point of Time; Illustration

18.1 INTRODUCTION

Rapid urbanization and land scarcity have prompted widespread use of multi-story reinforced concrete (RC) buildings in most countries including India. The country is seeing fast growth of such high-rise buildings in different cities. Builders are forced to complete such projects at a fast pace primarily for economic reasons. Pressures from buyers also sometimes dictate fast construction cycles of the various floors of such buildings. A cycle time of 4 to 7 days per floor which was unheard of in this part of the world is becoming a reality in the modern days.

The reduced cycle time and the tendency of the builders to strip the formwork faster, have resulted in a number of accidents in the United States, the Soviet Union, and Japan in the 1980's, the 1990's, and recently in India. Few such accidents have already been discussed in the previous chapter. The major reason for such accidents is found to be excessive loads imposed onto the supporting slabs, shores, and reshores. It may so happen that the strength of a slab is not enough to support an upper floor when it is cast. The problem is more compounded when the design live load to dead load ratio is small, in the range of 0.5 to 1.0.

The premature removal of shores is another primary reason for the collapse of structures during construction. The shore removal process can induce overloads in slabs which may endanger its safety.

The decrease in cycle time requires a newly placed concrete slab to be temporarily supported on the supporting systems comprising of several previously cast slabs and shoring systems. Depending on a number of factors (explained later), one, two, three, or four levels of shores may be used to support the loads safely during construction. Increasing the number of levels of shores during construction however is not recommended as it increases the number of sets of formworks required thereby increasing the cost. Further, it also increases the ultimate shore loads on the lower floors and interferes in other works to be carried out on a floor.

In order to overcome the problem of large number of shore levels, different construction techniques/procedures/sequences are utilized which can ensure safety. Before going into the techniques, we will discuss the terms shores, reshores, backshores, and preshores which are frequently encountered in multi-story RC building construction.

Shores

Shores are vertical or inclined support members designed to carry the weight of the formwork, the concrete and the construction loads above. Different types of shores have already been discussed in the previous chapters.

Reshores

Reshores are the shores placed snugly under a stripped concrete slab or structural member after the original forms and shores have been removed from a large area, thus requiring the new slab or structural member to deflect and support its own weight and of the existing construction loads applied prior to the installation of the reshores. At the time of installation, reshores carry no significant load. Reshores are like normal shores in design although they are placed with larger spacing between them.

Backshores

Backshores are the shores placed snugly under a stripped concrete slab or structural member after the original forms and shores have been removed from a small area without allowing the slab to deflect or support its own weight or of the existing construction loads from above.

Preshores

Preshores are the refined version of the reshoring method. It is the technique of scheduled reshoring whereby the unsupported slab is reduced and controlled.

A rational judgment on the number of levels of shores and reshores for a given pace of construction is also a matter of concern for a formwork engineer. He/she has to make a right balance between safety and economy aspects. During construction, the safety of a slab depends mainly on the load imposed on the slab and the available strength of the slab. For a given construction load, the load distribution in the slab-shore system is decided by the number of levels of shores and reshores, the stripping sequence, the age of the slabs (which influences the stiffness of slabs), and the load distribution. On the other hand, the available strength of a slab depends on the age of the concrete, the type of cement, the construction temperature, and any admixtures. A formwork engineer has to ensure that the load imposed on slabs during construction is less than their available strength at every construction stage.

In the subsequent sections, we discuss the construction techniques/procedures/sequence in multi-story RC building construction. Subsequently a simplified analysis is presented to find the distribution of loads on the shores and slabs in multi-story structures. The limitations of the simplified analysis and a brief discussion on other methods for the computation of loads on the shores and slabs are discussed. Next, the determination of strength of the concrete slab at different ages is explained. Finally, a large number of examples are provided to illustrate the process of making sure the safety of the slab at each step of construction by adopting different combinations of cycle time, stripping time, construction sequence, concrete mix, and curing temperature, etc.

18.2 TECHNIQUES IN MULTI-STORY RC CONSTRUCTION

The terms construction techniques/procedures/sequences are interchangeably used in the chapter. Shoring, reshoring, backshoring, and preshoring are different construction procedures used to distribute construction loads through the lower floors. Though load distribution analysis is similar, there are significant differences in magnitude, duration, and timing of the floor and shore loads for each procedure. The commonly used construction procedures adopted in multi-story construction are discussed in the following sections.

18.2.1 Shoring

"The typical construction procedure for slabs involves casting the new slab on formwork and shores, and removal of the shores from the lowest level in perhaps four to five days after casting the slab. Shoring is carried to several floors; so the load of the fresh concrete will be shared among several supporting slabs. Increasing the number of shores will delay the occurrence of the peak load, thus benefiting from the increase in strength with time; however, it will increase the maximum slab load and the maximum shore load" (Mosallam and Chen, 1990).

The construction sequence of three levels of shores and the major activities: (a) casting the slab, (b) removing the lowest level of shores, and (c) installing the highest level of shores are shown in the schematic sketches in Fig. 18.1.

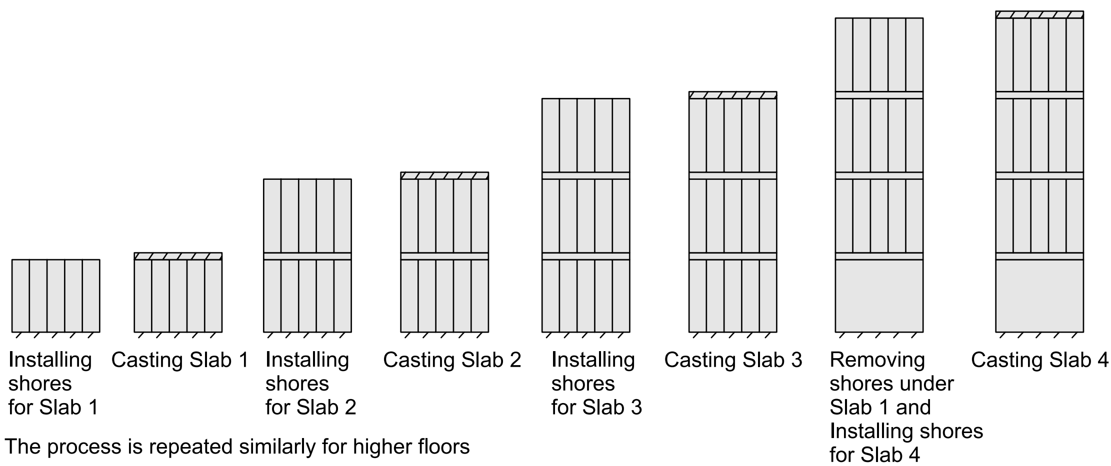

Figure 18.1 Construction Sequence and Activities Involved with Three Levels of Shores.

18.2.2 Shoring and Reshoring

"The shore/reshore technique involves the use of typically only one level of shores and several levels of reshores. Basically, the forms/shores are removed from beneath a slab allowing it to deflect and carry its own weight; reshores are then installed allowing the load during concrete placement to be shared between the various slabs in the systems. The slab loads imposed using reshoring construction are much less than those using shores but the loads are imposed at earlier ages. Reshoring usually requires fewer levels of interconnected slabs, thus freeing more areas for other activities to proceed. Near-capacity loads in slabs usually occur for shorter periods. The disadvantage of the reshoring technique is that, while allowing the slab to deflect reduces the slab maximum loads applied to the slab, the slab deflection can be excessive as the concrete is too young to have developed much stiffness" (Mosallam and Chen, 1990).

The construction sequence of two levels of shores and one level of reshores and the major activities involving (a) casting the slab, (b) removing the lowest level of reshores, (c) removing the lowest level of shores, (d) installing the highest level of reshores, and (e) installing the highest level of shores are shown in the schematic sketches in Fig.18.2.

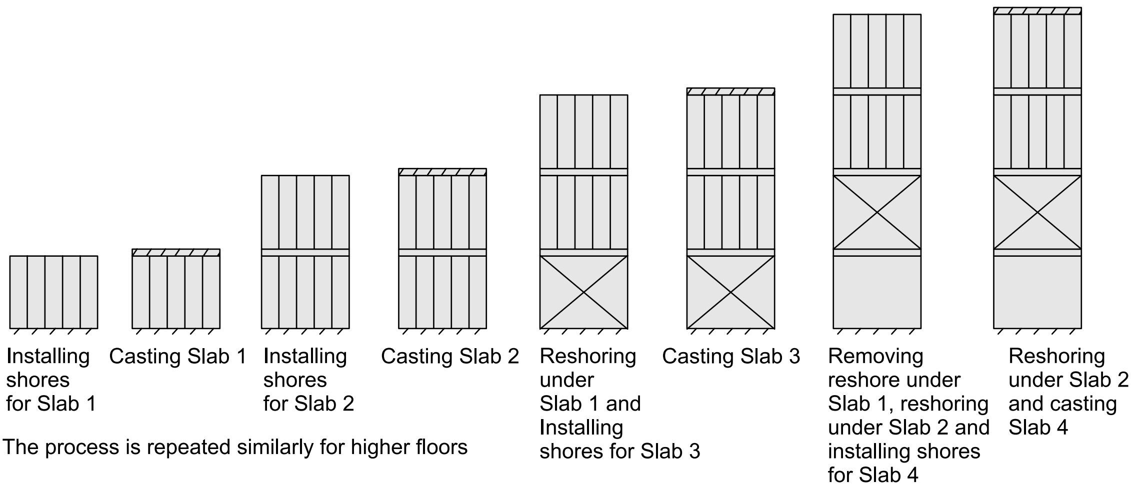

Figure 18.2 Construction Sequence and Activities Involved with Two Levels of Shores and One Level of Reshores.

18.2.3 Shoring and Backshoring

According to the construction dictionary, (http://www.construction-dictionary.com), backshoring is the reinsertion of shores beneath a stripped concrete slab after the original formwork and shoring has been removed from a small section. Unlike reshoring, backshoring keeps the slab from supporting its own weight or the weight of the existing loads above it until the slab attains full strength.

Using backshores, stripping of forms may be accomplished at an early age because large areas of concrete are not required to carry their own weight (Mosallam and Chen, 1990). Backshoring requires knowledgeable supervision and extreme caution. Care must be exercised to ensure that individual shores are not overloaded during stripping. The comparative features of reshoring and backshoring are given in Table 18.1.

Table 18.1 Comparison of Backshoring and Reshoring

S. No.	Feature	Backshoring	Reshoring
1.	Stripping	In backshoring, small areas are stripped at a time.	In reshoring, several bays can be stripped at the same time.
2.	Slab deflection	In backshoring, slab is not allowed to deflect.	In reshoring, slab is allowed to deflect.
3.	Installation of shores	Backshores are installed before any further stripping occurs.	Reshores are installed without removing deflection.
4.	Weight of slab	In backshoring, slab does not carry its own weight.	In reshoring, slabs carry their weight.
5.	Initial load	Backshores have an initial load.	Reshores have no initial load.

18.2.4 Shoring, Preshoring, and Reshoring

In preshoring construction, specified shores in the shore removal process are removed leaving other shores in place; reshores are then installed at the location(s) of the removed shores; the remaining shores are then removed and replaced by reshores (Mosallam and Chen, 1990). The method is illustrated in Fig. 18.3. Figure 18.3a represents the casting of a new floor onto the formwork supported by shores 1, 2 and 3. Figure 18.3b shows shores 1 and 3 removed, allowing the slab to deflect over a span of half the bay. Figure 18.3c represents the reshores placed at 1 and 3 beneath the deflected slab and shore 2 removed. Finally, Fig. 18.3d shows reshores placed at location 2. Generally operations shown in Figs. 18.3b and 18.3c are carried out on the same day.

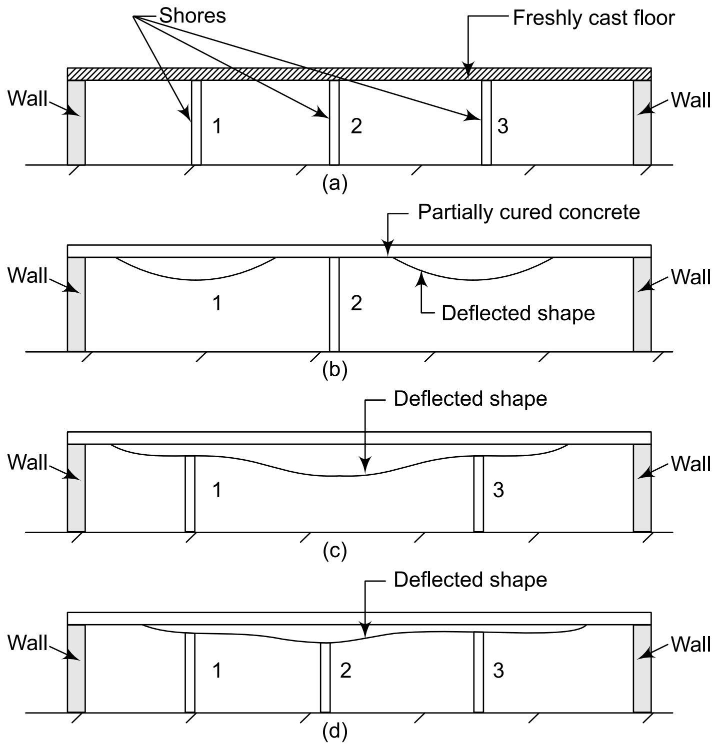

Figure 18.3 Schematic Representation of the Preshoring Technique.

One of the advantages of preshoring is that the unsupported span lengths are reduced, thus reducing slab deflections (Fig. 18.3b). The disadvantages of preshoring are:

(a) Close control of the construction process is needed to realize the benefits.
(b) Preshoring is more complicated to model than reshoring. The analysis in preshoring needs consideration of slab deformations, shore deformation, and static equilibrium of the loads.

18. 3 DISTRIBUTION OF LOADS ON SHORES AND SLABS IN MULTI-STORY STRUCTURES—SIMPLIFIED ANALYSIS

The distribution of loads on shores and slabs in multi-story structures was first studied by Neilsen(1952). He made an extensive theoretical analysis of the interaction between the formwork and the slabs supporting falsework loads under the following assumptions:

1. the slabs behave elastically;
2. shrinkage and creep of the concrete can be neglected;
3. shores are represented by uniform elastic support; and
4. the torsional moments and the shearing forces in the formwork are neglected.

Neilsen's procedures seem lengthy and inconvenient for practical use. Hurd (2005) assumed the following to develop a simplified method for the computation of slab and shore loads.

1. Shores and reshores are infinitely stiff relative to the slabs.
2. Slabs are interconnected by shores; therefore all deflect equally when a new load is added, and carry a share of the added load in proportion to their relative stiffness.
3. Slabs have equal stiffness and added loads are shared equally by the interconnected slabs. Whether one assumes that the slabs are all equally stiff or that stiffness is proportional to the various levels of strength attained by the slabs of different ages, the results differ by relatively small amounts. Therefore, it seems both satisfactory and convenient to follow the simpler assumption.
4. Ground level floor or other base support is rigid.

The load distribution on slabs, shores, and reshores are performed based on the above sets of assumption. The method is illustrated on (a) one, two, and three levels of shores and (b) two levels of shores and one level of reshores for two cases. In one case, only the dead load of slab has been considered while in the second case, other loads such as construction live loads, self-weight of shores, formwork and reshores are considered. The illustrations under (b) are based on the examples provided by Hurd (2005).

18.4 LOAD DISTRIBUTION FOR SLABS AND SHORES IN ONE, TWO, THREE, AND FOUR LEVELS OF SHORES

The following symbols have been used.

Let Si represent the load on slab i, thus $S1$, and $S2$ represent loads on slab 1, and 2 respectively.

Let Pi represent the loads on shores under slab i, thus $P1$ and $P2$ represent loads on shores under slab1 and 2 respectively.

18.4.1 One Level of Shores

Let's assume one level of shores only; and consider only the self weight of slab equal to D. The construction live load and the dead weight of formwork and shores are neglected. Further, assume a cycle time of $T = 7$ days and stripping time of $m = 5$ days.

Let's assume that it takes about 6 days to install the shores, formwork for slabs, and tying of reinforcement; and slab $S1$ is cast on day 7. The loads carried by the slabs are zero and the load carried by shores under slab 1 is $P1 = D$. This arrangement is shown in Fig. 18.4.

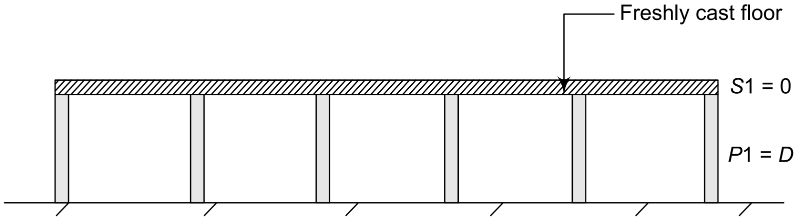

Figure 18.4 Casting of Slab 1 on Day 7.

On day 12, the shores under slab 1 are removed. The shore load of $P1 = D$ is carried by slab 1. Thus, $S1 = D$ and $P1 = 0$ (Fig. 18.5)

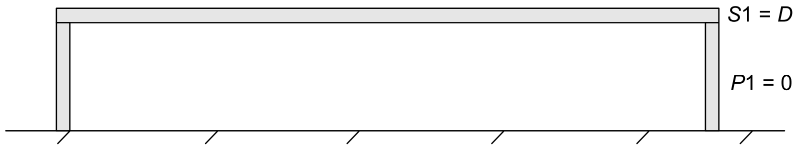

Figure 18.5 Removal of Shores Under Slab 1 on Day 12.

On day 14, slab 2 is cast. The load D of slab 2 is carried by the interconnected slabs 2 and 1; however since slab 2 is freshly cast, it cannot take any load and thus the load is transferred to slab 1. The shores under slab 2 carry that portion of the load above them which is not carried by the slabs. The load carried by slabs and shores at this stage are: $S2 = 0$ (freshly cast slab); $S1$ = initial load + share of new load = $D + D = 2D$; $P2 = 1D$ load above (weight of slab 2) –load carried by slab 2 = $1D - 0 = 1D$ (see Fig. 18.6).

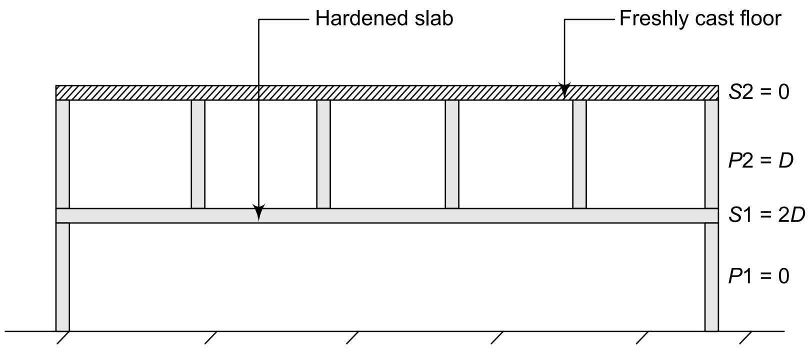

Figure 18.6 Casting of Slab 2 on the Day 14.

On day 19, the shores under slab 2 are removed. The shore load of $P2=D$ is carried by slab 1. Thus, $S2=D$, $P2=0$, $S1=D$, and $P1=0$ (Fig. 18.7).

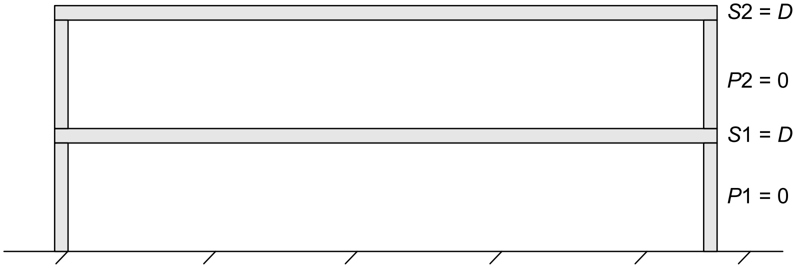

Figure 18.7 Removal of Shores Under Slab 2 on Day 19.

On day 21, slab 3 is cast. The load D of slab 3 is to be carried by the interconnected slabs 3 and 2; however since slab 3 is freshly cast, it cannot take any load and thus the entire load is transferred to slab 2. The shores under slab 3 carry that portion of the load above them not carried by the slabs. The load carried by the slabs and shores at this stage are: $S3= 0$ (freshly cast slab); $S2$ =initial load + share of new load = $D + D = 2D$; $P3=1D$ load above (weight of slab 2) –load carried by slab 2= $1D-0=1D$, $P2=0$, and $S1=D$ (Fig. 18.8).

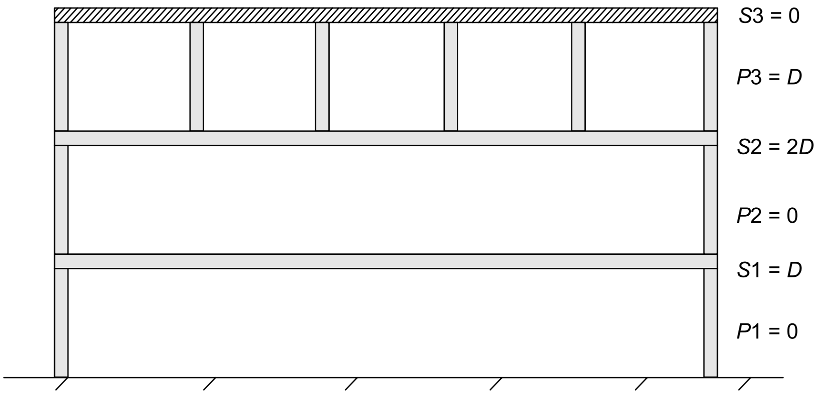

Figure 18.8 Casting of Slab 3 on Day 21.

In a similar manner, the load on slabs and shores can be computed. For easy reference, some more steps are shown in Fig. 18.9 along with the steps explained earlier. As can be seen from Fig. 18.9, after certain steps, the loads on slabs, shores, and reshores converge.

18.4.2 Two Levels of Shores

For this case, let's assume that two levels of shores are provided. Let's assume only the self weight of slab equal to D and neglect the construction live load and the dead weight of formwork and shores. Further, assume a cycle time of $T = 7$ days and stripping time of $m = 5$ days.

Slabs $S1$and $S2$ are cast on day 7 and 14, respectively. The loads carried by the slabs are zero and the load carried by shores under slabs 2 and 1 are $P2= D$ and $P1=2D$, respectively.

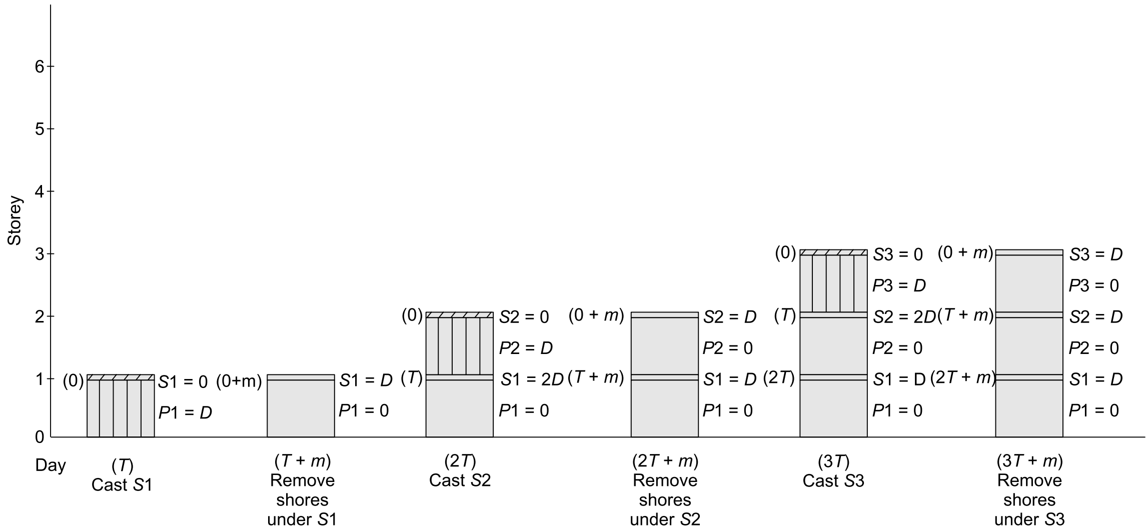

Figure 18.9 Load Ratios Versus Time for One Level of Shores.

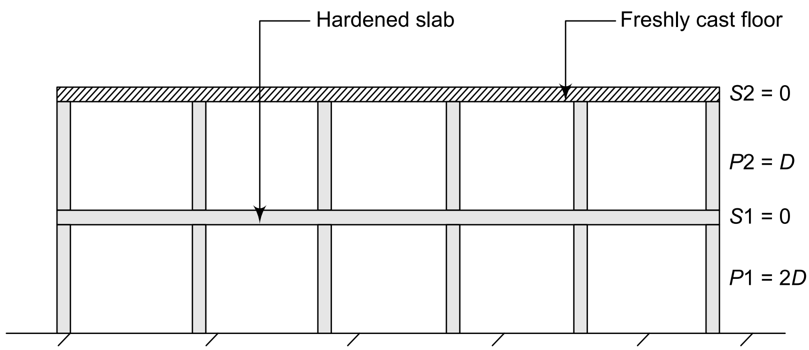

Figure 18.10 Casting of Slab 2 on the Day 14 (Shores Under Slab 1 and 2 are in Position).

Shores under slab 1 are removed on the 19th day ($2T+m$ days, where $T = 7$, and m = stripping period = 5). The shore load of $P1 = 2D$ is carried equally by slabs 1 and 2. Thus, $S2 = D$, $S1 = D$, $P2 = 0$, and $P1 = 0$ (Fig. 18.11).

On day 21, slab 3 is cast. The load D of slab 3 is carried equally among the two interconnected slabs 2 and 1. The shores under slabs 3 and 2 carry that portion of the load above them which is not carried by the slabs. The load carried by slabs and shores at this stage are computed below (also see Fig. 18.12).

$S3$ = initial load + share of new load = 0 (freshly cast slab)

$S2$ = initial load + share of new load = $D + D/2 = 3D/2$,

$S1$ = initial load + share of new load = $1D + D/2 = 3D/2$

$P3 = 1D$ load above (weight of slab 3) –load carried by slab 3 = $1D - 0 = 1D$

$P2 = 2D$ load above (weight of slab 3 + weight of slab 2) – (load carried by slab 3 + load carried by slab 2) = $2D - (0 + 3D/2) = D/2$

$P1 = 3D$ load above (weight of slab 3 + weight of slab 2 + weight of slab 1) – (load carried by slab 3 + load carried by slab 2 + load carried by slab 1) = $3D - (0 + 3D/2 + 3D/2) = 0$ (as expected).

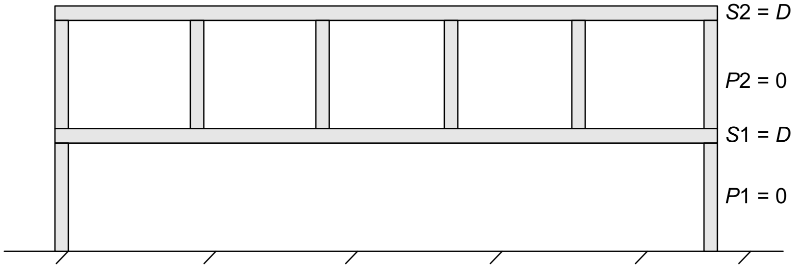

Figure 18.11 Removal of Shores Under Slab 1 on Day 19.

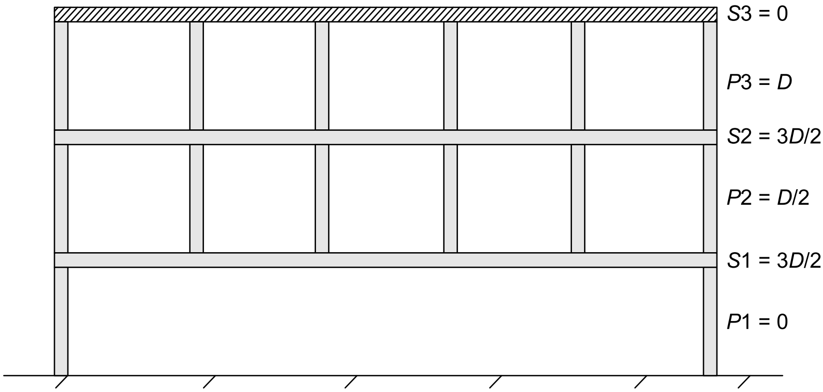

Figure 18.12 Casting of Slab 3 on Day 21.

The shores under slab 2 are removed. The load carried by shores ($P2 = D/2$) under slab 2 is distributed on two interconnected slabs 3 and 2. Thus each of the two slabs would have to carry an additional load of $D/4$. The load on slabs and shores are computed below:

$S3$ = initial load + share of new load = $0 + D/4 = D/4$

$S2$ = initial load + share of new load = $3D/2 + D/4 = 7D/4$,

$S1$ = initial load – load carried by shores under slab 2 = $3D/2 - D/2 = 1D$

$P3 = 1D$ load above (weight of slab 3) – load carried by slab 3 = $1D - D/4 = 3D/4$

$P2 = 2D$ load above (weight of slab 3 + weight of slab 2) – (load carried by slab 3 + load carried by slab 2) = $2D - (D/4 + 7/4\ D) = 0$ (as expected). The load distribution is shown in Fig. 18.13.

On day 28, slab 4 is cast. The load D of slab 4 is carried equally among the two interconnected slabs 3 and 2. The shores under slabs 4 and 3 carry that portion of the load above them which is not carried by the slabs. The load carried by slabs and shores at this stage are computed below (also see Fig. 18.14).

$S4$ = initial load + share of new load = 0 (freshly cast slab)

$S3$ = initial load + share of new load = $D/4 + D/2 = 3D/4$

$S2$ =initial load + share of new load = $7/4D + D/2 = 9D/4$

$P4 = 1D$ load above (weight of slab 4) -load carried by slab 4 = $1\ D - 0 = 1D$

$P3 = 2D$ load above (weight of slab 4 + weight of slab 3) – (load carried by slab 4 + load carried by slab 3 = $2D - (0 + 3D/4) = 5D/4$

$P2 = 3D$ load above (weight of slab 4+weight of slab 3 + weight of slab 2) – (load carried by slab 4 + load carried by slab 3 + load carried by slab 2) = $3D - (0 + 3D/4 + 9D/4) = 0$ (as expected).

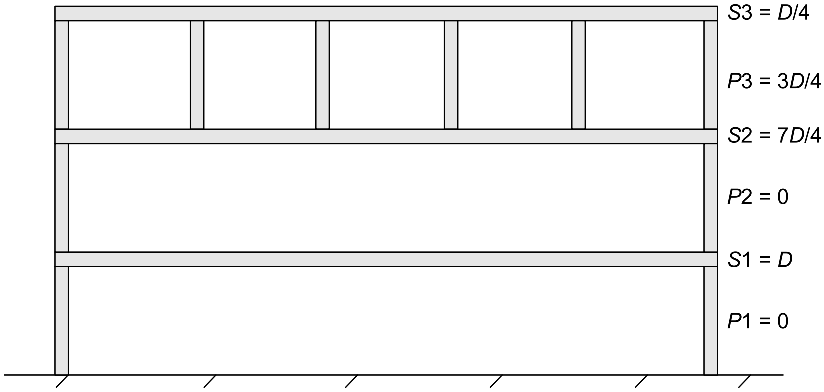

Figure 18.13 Removal of Shores Under Slab 2 on Day 26.

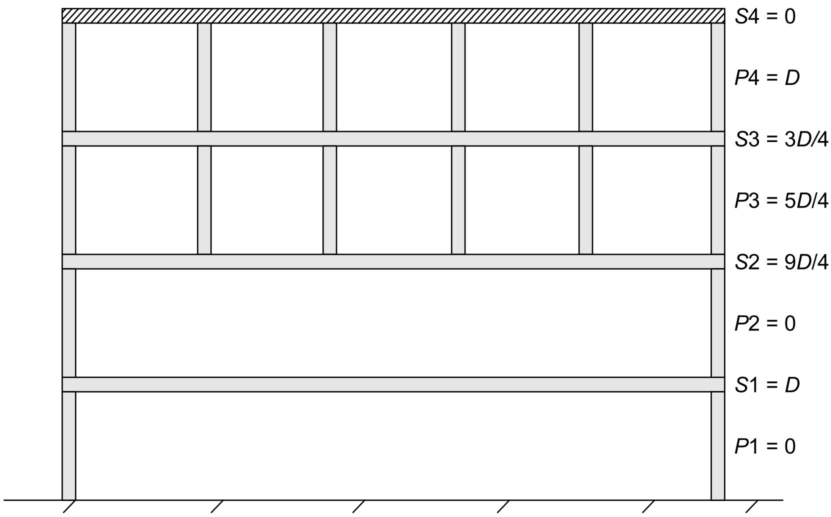

Figure 18.14 Casting of Slab 4 on Day 28.

In a similar manner, the load on slabs and shores can be computed. For easy reference, some more steps are shown in Fig. 18.15 along with the previous steps explained earlier. As can be seen from Fig. 18.15, after certain steps, loads on slabs, shores, and reshores converge.

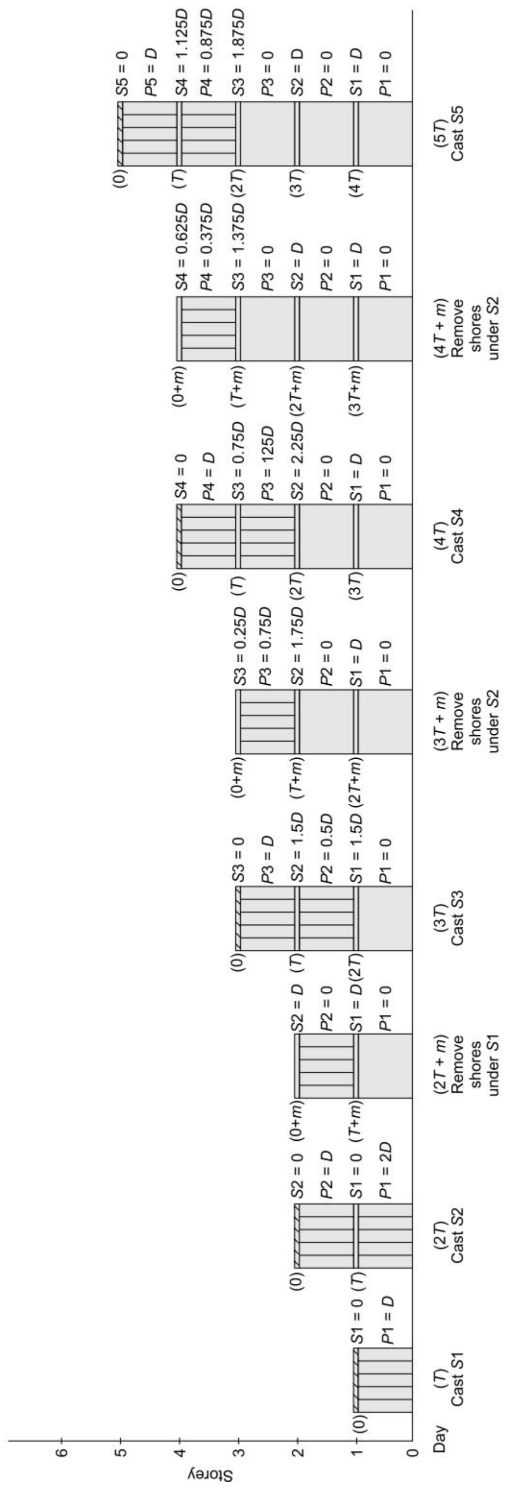

Figure 18.15 Load Ratios Versus Time for Two Levels of Shores.

18.4.3 Three Levels of Shores

A final example is presented wherein three levels of shores are provided and there is no reshoring. Let's assume only the self weight of slab equal to D and neglect the construction live load and the dead weight of formwork, and shores. Further, assume a cycle time of $T = 7$ days and stripping time of $m = 5$ days.

Slabs $S1$, $S2$, and $S3$ are cast on day 7, 14, and 21, respectively. The loads carried by the slabs are zero and the load carried by shores under slabs 3, 2, and 1 are $P3 = D$, $P2 = 2D$, and $P1 = 3D$.

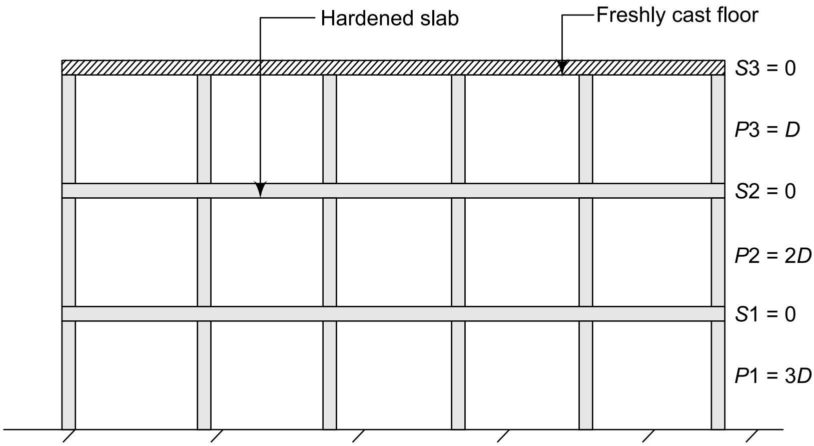

Figure 18.16 Casting of Slab 3 on Day 21.

Shores under slab 1 are removed on the 26th day ($3T + m$ days, where $T = 7$, and m = stripping period = 5). The shore load of $P1 = 3D$ is carried equally by slabs 1, 2, and 3. Thus, $S3 = D$, $S2 = D$, $S1 = D$, and $P3 = 0$, $P2 = 0$, $P1 = 0$ (Fig. 18.17).

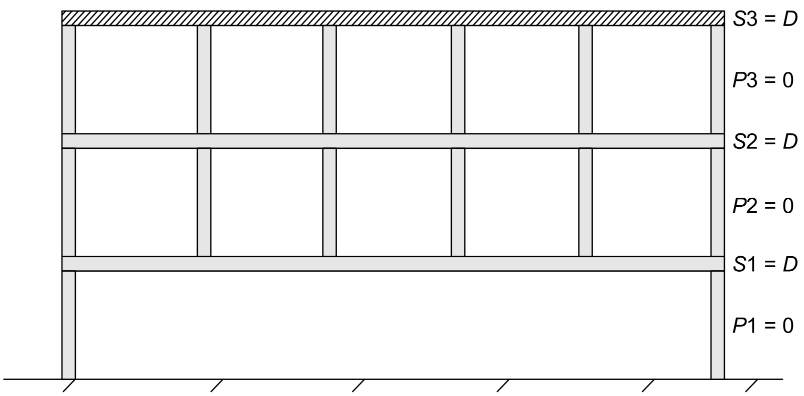

Figure 18.17 Removal of Shores Under Slab 1 on Day 26.

On day 28, slab 4 is cast. The load D of slab 4, is carried equally among the three interconnected slabs 3, 2, and 1. The shores under slabs 4, 3, and 2 carry that portion of the load above them which is not carried by the slabs. The load carried by slabs and shores at this stage are computed below (also see Fig. 18.18).

$S4$ = initial load + share of new load = 0 (freshly cast slab)

$S3$ = initial load + share of new load = $D + D/3 = 4D/3$

$S2$ = initial load + share of new load = $D + D/3 = 4D/3$,

$S1$ = initial load + share of new load = $1D + D/3 = 4D/3$

$P4 = 1D$ load above (weight of slab 4) –load carried by slab 4 = $1D - 0 = 1D$

$P3 = 2D$ load above (weight of slab 4 + weight of slab 3) – (load carried by slab 4 + load carried by slab 3) = $2D - (0 + 4D/3) = 2D/3$

$P2 = 3D$ load above (weight of slab 4 + weight of slab 3 + weight of slab 2) – (load carried by slab 4 + load carried by slab 3 + load carried by slab 2) = $3D - (0 + 4D/3 + 4D/3) = D/3$.

$P1 = 4D$ load above (weight of slab 4 + weight of slab 3 + weight of slab 2 + weight of slab 1) – (load carried by slab 4 + load carried by slab 3 + load carried by slab 2 + load carried by slab 1) = $4D - (0 + 4D/3 + 4D/3 + 4D/3) = 0$ (as expected).

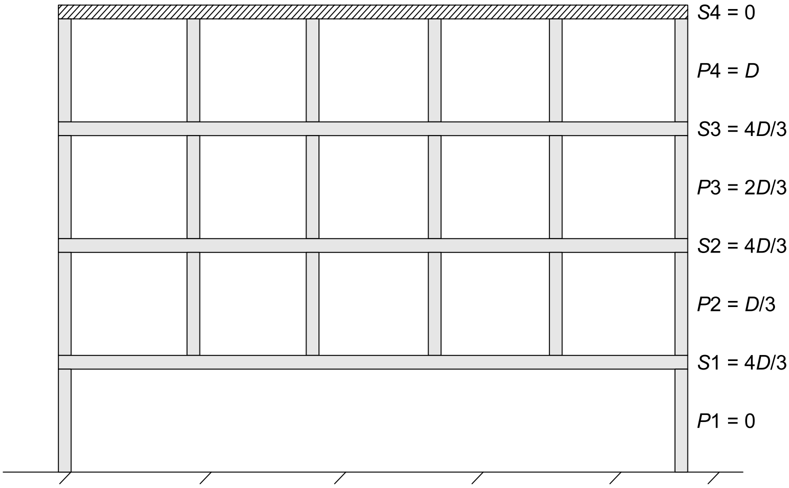

Figure 18.18 Casting of Slab 4 on Day 28.

The shores under slab 2 are removed. The load carried by shores ($P2 = D/3$) under slab 2 is distributed on three interconnected slabs 4, 3, and 2. Thus each of the three slabs would have to carry an additional load of $D/9$. The load on the slabs and shores are computed below:

$S4$ = initial load + share of new load = $0 + D/9 = D/9$

$S3$ = initial load + share of new load= $4D/3 + D/9 = 13D/9$,

$S2$ = initial load + share of new load= $4D/3 + D/9 = 13D/9$

$S1$ = initial load - load carried by shores under slab 2 = $4D/3 - D/3 = 1D$

$P4 = 1D$ load above (weight of slab 4) –load carried by slab 4 = $1D - D/9 = 8D/9$

$P3 = 2D$ load above (weight of slab 4 + weight of slab 3) – (load carried by slab 4 + load carried by slab 3) = $2D - (D/9 + 13D/9) = 4D/9$. The load distribution is shown in Fig. 18.19.

On day 35, slab 5 is cast. The load D of slab 5, is carried equally among the three interconnected slabs 4, 3, and 2. The shores under slabs 5, 4, and 3 carry that portion of the load above them which is not carried by the slabs. The load carried by the slabs and shores at this stage are computed below (also see Fig. 18.20).

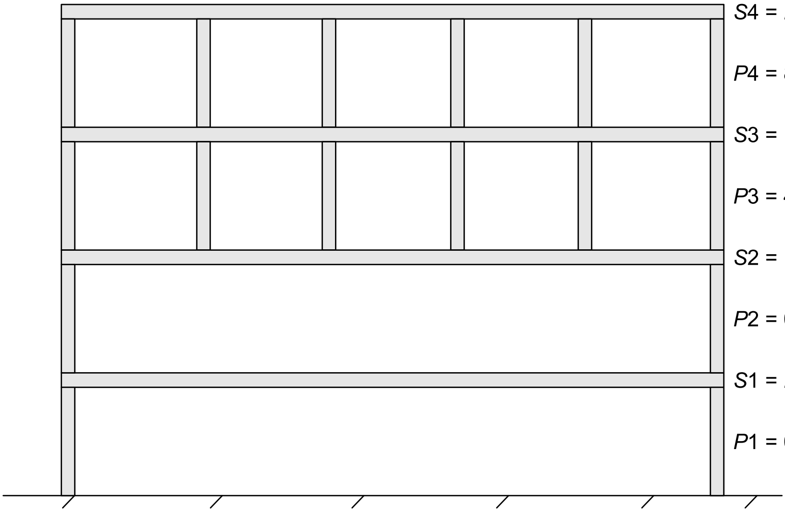

Figure 18.19 Removal of Shores Under Slab 2 on Day 33.

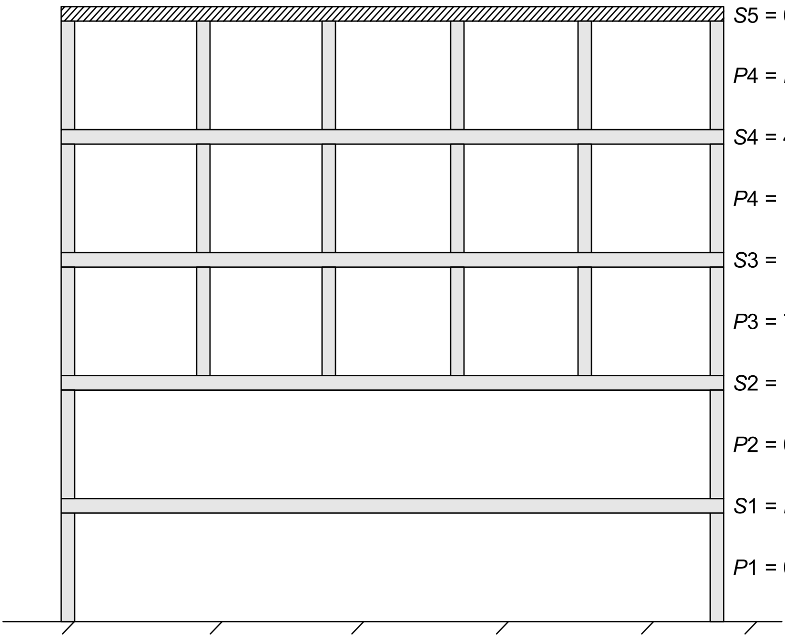

Figure 18.20 Casting of Slab 5 on Day 35.

S5 = initial load + share of new load = 0 (freshly cast slab)
S4 = initial load + share of new load = $D/9 + D/3 = 4D/9$
S3 = initial load + share of new load = $13/9D + D/3 = 16D/9$
S2 =initial load + share of new load= $13/9D + D/3 = 16D/9$
P5 = 1*D* load above (weight of slab 5) –load carried by slab 5 = $1D$-0 =$1D$

$P4$ = $2D$ load above (weight of slab 5 + weight of slab 4) – (load carried by slab 5 + load carried by slab 4 = $2D – (0 + 4D/9) = 14D/9$

$P3$ = $3D$ load above (weight of slab 5+weight of slab 4 + weight of slab 3) – (load carried by slab 5 + load carried by slab 4+ load carried by slab 3) = $3D – (0 + 4D/9 + 16D/9) = 7D/9$

$P2$ = $4D$ load above (weight of slab 5 + weight of slab 4 + weight of slab 3 + weight of slab 2) – (load carried by slab 5 + load carried by slab 4+ load carried by slab 3 + load carried by slab 2) = $4D – (0 + 4D/9 + 16D/9 + 16D/9) = 0$ (as expected).

The shores under slab 3 are removed. The load carried by shores (P3 = $7D/9$) under slab 3 is distributed on three interconnected slabs 5, 4, and 3. Thus each of the three slabs would have to carry an additional load of $7D/27$. The load on slabs and shores are computed below:

$S5$ = initial load + share of new load = $0 + 7D/27 = 7D/27$

$S4$ = initial load + share of new load = $4D/9 + 7D/27 = 19D/27$

$S3$ = initial load + share of new load = $16D/9 + 7D/27 = 55D/27$,

$S2$ = initial load - load carried by shores under slab 3 = $16D/9 - 7D/9 = D$

$P5$ = $1D$ load above (weight of slab 5) –load carried by slab 5 = $1D – 7D/27 = 20D/27$

$P4$ = $2D$ load above (weight of slab 5 + weight of slab 4) – (load carried by slab 5+ load carried by slab 4) = $2D – (7D/27 + 19D/27) = 28D/27$

$P3$ = $3D$ load above (weight of slab 5 + weight of slab 4 + weight of slab 3) – (load carried by slab 5 + load carried by slab 4 + load carried by slab 3) = $3D – (7D/27 + 19D/27 + 55/27\ D) = 0$ (as expected).

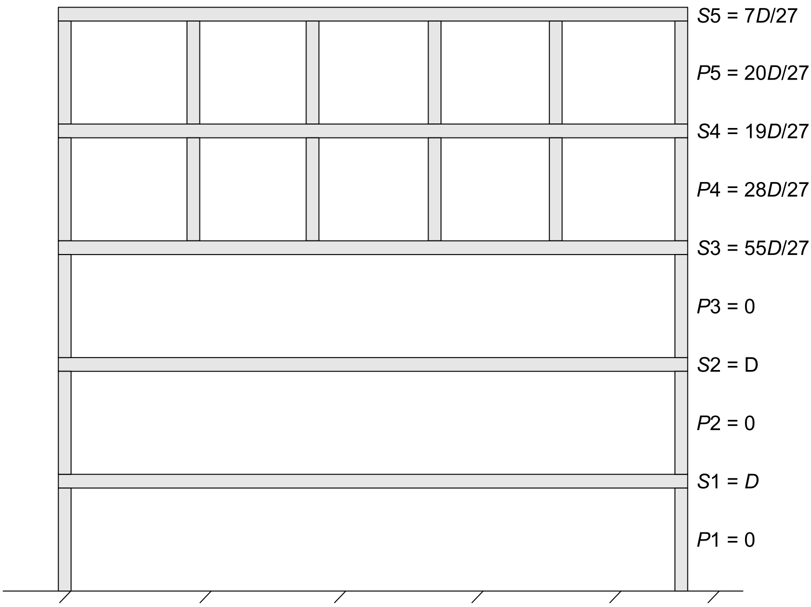

Figure 18.21 Removal of Shores Under Slab 3 on Day 40.

In a similar manner, the load on slabs and shores can be computed. For easy reference, some more steps are shown in Fig. 18.22 along with the steps explained earlier. As can be seen from Fig. 18.22, after certain steps, loads on slabs and shores converge.

In the previous sections, the load distribution on slabs and shores was explained with the help of a simplified method.

The maximum load on the slabs occurs when the shores connecting the supporting assembly with the ground level are removed. However, the load ratio converges for the upper floor levels. This is shown in Figs. 18.23(a), (b), and (c) for one, two, and three levels of shores, respectively.

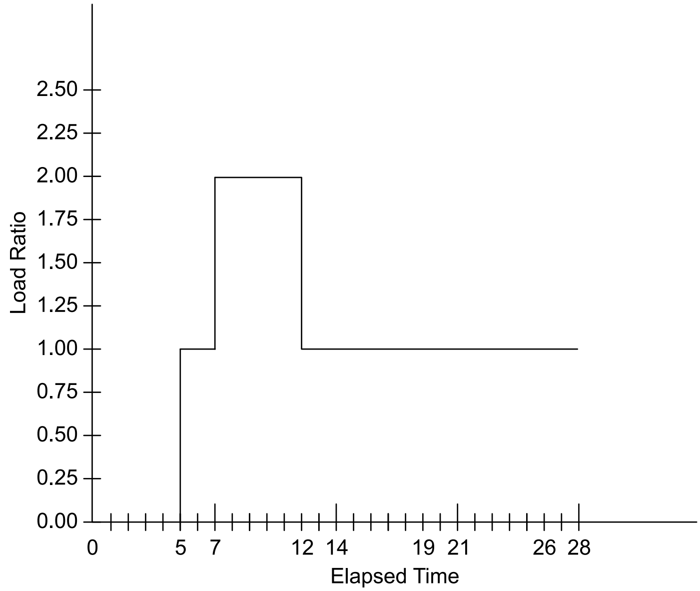

Figure 18.23 (a) Load-Ratio History of the Slabs for One Level of Shores.

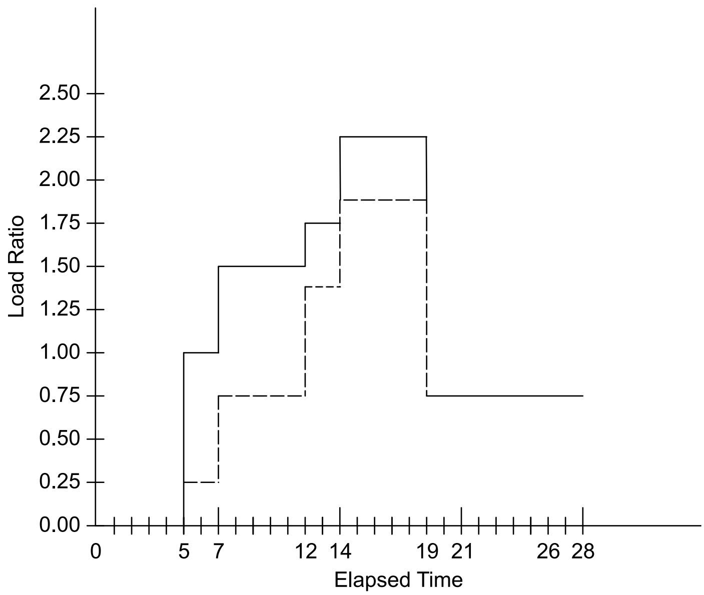

Figure 18.23 (b) Load-Ratio History of the Slabs for Two Level of Shores.

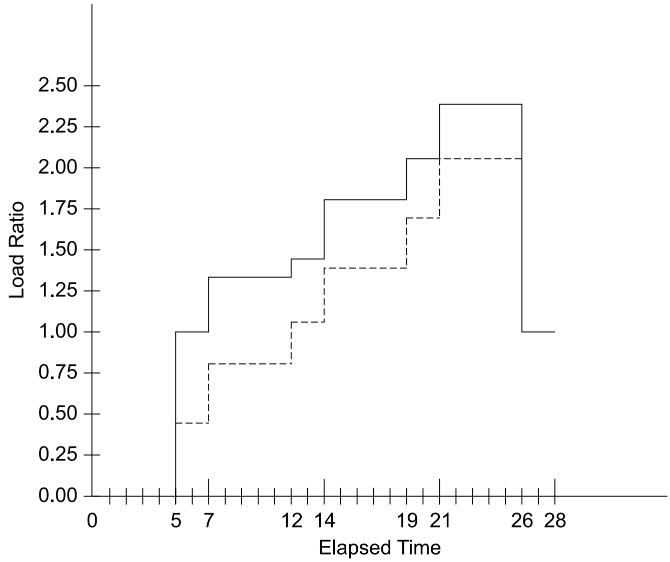

Figure 18.23 (c) Load-Ratio History of the Slabs for Three Level of Shores.

From Fig. 18.23(a), it can be seen that the peak load and the converged load is 2 *D*. From Fig. 18.23(b), it can be observed that the absolute maximum load ratio is 2.25 at 14 days for two levels of shores. In the case of three levels of shores, the simplified analysis showed the maximum load ratio equal to 2.36 while the converged value for upper floor levels is equal to 2.00. The most heavily loaded slab is always the last slab that is supported directly from the foundation. This is shown in Fig. 18.23(c). The absolute maximum load ratio was 2.36 at 42 days.

In the case of four levels of shores, the absolute maximum is 2.43 at 28 days (verify!!!). Whether it is the case of the two or four levels of shores, the maximum loads are always carried by the last slab cast before the shores at ground level are removed. It can also be noticed that above level 4, the loads steadily converge with a maximum load factor of 2.00 in the bottom slab aged 21 days.

In each case (in fact, for any number of shored levels), the loads converge on a cycle in which the bottom slab of the shored system has a load ratio of 2.00.

It can also be observed from the above load distribution that increasing the number of levels shored leads to no reduction at all in the average maximum construction loads on the slabs. The absolute maximum actually increases with an increasing number of levels shored. However, the age of the slab at which these maxima occur also increases. Thus the increase in the concrete strength may offset the extra load as a design consideration.

18.5 LOAD DISTRIBUTION FOR SLABS AND SHORES IN TWO LEVELS OF SHORES AND ONE LEVEL OF RESHORES

18.5.1 Only Dead Load of Slab Considered in the Analysis

The load distribution on slabs, shores, and reshores are demonstrated for two levels of shores and one level of reshores. As before, we consider only the dead weight of slab *D*. The self weight of shores and reshores, and construction live loads exerted at the time of casting the slab are neglected.

It is further assumed that the cycle time T for casting slabs is 7 days and the lowest levels of shores are removed after 5 days ($m = 5$) of casting the top slab. The cycle time T is defined as the period from the beginning of casting a concrete slab to the beginning of casting the next concrete slab. Thus if the first slab is cast on day 7, 2nd, 3rd , and 4th slab will be cast on day 14, 21, and 28, respectively.

The shores under floor 1 will be removed on day 12 and reshores will be kept in place.

Let's assume that work starts on the day 1 and it takes six days to complete the installation of shores, formwork, and the reinforcement for slab 1. On day 7, slab 1 is cast. At this point, the entire weight of slab D is taken by shores under slab 1 and is transmitted to the ground. Thus, the load carried by slab 1 is 0 and the load carried by shores is D.

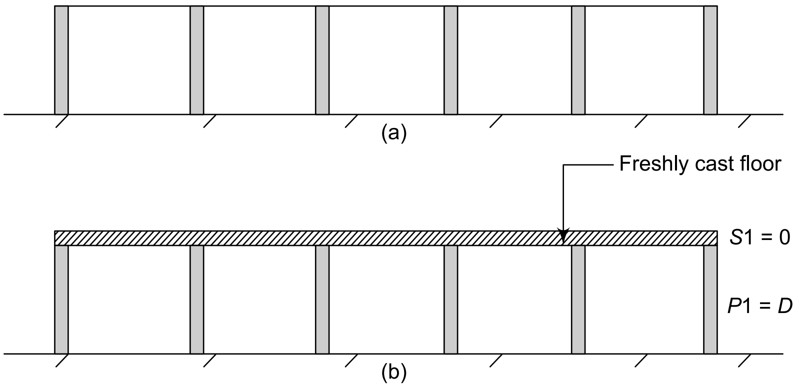

Figure 18.24 (a) Slab Ready to Cast at the End of Day 6, (b) Casting of Slab on Day 7.

Considering the cycle time of seven days, it is assumed that slab 2 is cast on day 14. All loads of slab 1 and slab 2 go through shores to the ground. The load carried by shores under slab 2 is D and the load carried by shores under slab 1 is $2D$. This is indicated as $S2 = 0$, $P2 = 0$, $S1 = 0$, and $P1 = 2D$ in Fig. 18.25.

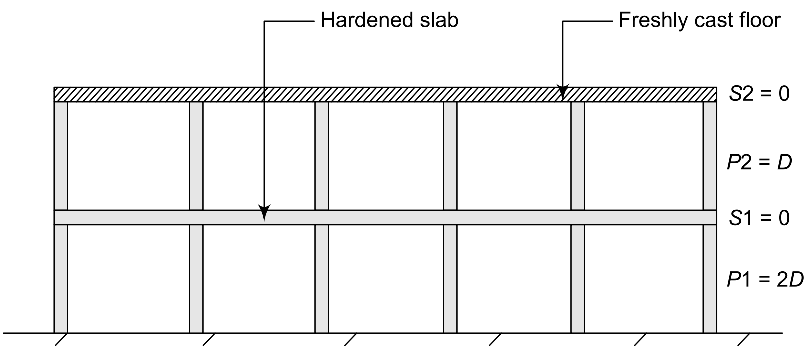

Figure 18.25 Casting of Slab 2 on Day 14.

Shores under slab 1 is removed on the 19th day ($2T + m$ days, where $T = 7$, and m = stripping period = 5). The shore load of $P1 = 2D$ is carried equally by slabs 1 and 2. Thus, $S2 = D$, $P2 = 0$, $S1 = D$, and $P1 = 0$ (Fig. 18.26)

Now the reshores are placed snugly under slab 1. Due to snug placement of the reshores, they are still not carrying any load and thus there is no change in load distribution from the previous case. The slab loads $S2 = D$, $P2 = 0$, $S1 = D$, and $P1 = 0$ remain unchanged (Fig.18. 27).

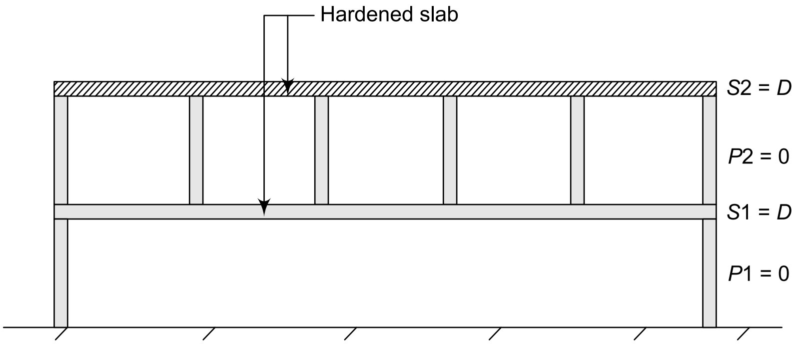

Figure 18.26 Removal of Shores Under Slab 1 on Day 19.

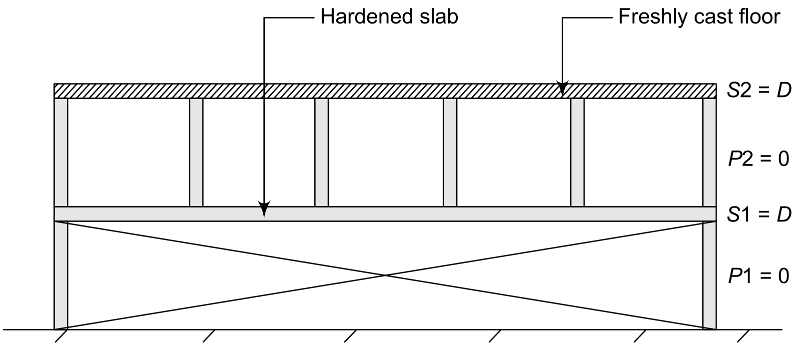

Figure 18.27 Installation of Reshores Under Slab 1 on Day 19.

On day 21, slab 3 is cast. The load D of slab 3 is carried to ground by shores under slabs 3 and 2 and reshores under slab 1. The load carried by slabs and shores at this stage are: $S3 = 0$, $P3 = D$, $S2 = D$, $P2 = D$, $S1 = D$, and $P1 = D$. This is shown in Fig.18.28.

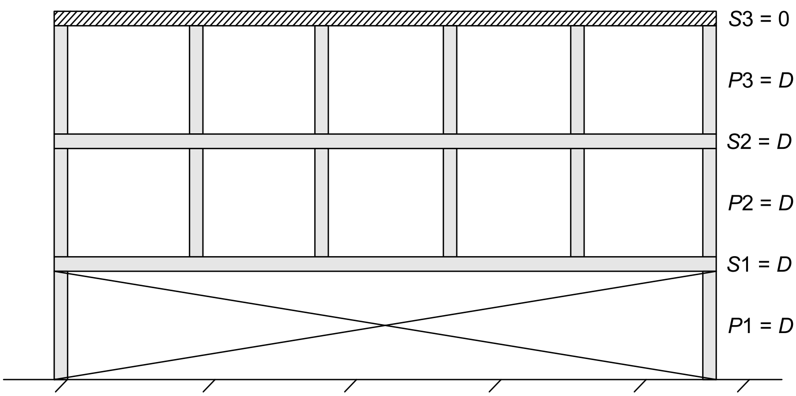

Figure 18.28 Casting of Slab 3 on Day 21.

Reshores under slab 1 are removed. This action will cause all three slabs *S3*, *S2*, and *S1* to deflect together. The reshores load of *P1* = *D* is distributed equally among three interconnected slabs 3, 2, and 1. The shores under slab 3 and 2 carry that portion of the load above them which is not carried by the slabs. The load carried by slabs and shores at this stage are:

S3 = initial load + share of new load = 0 + *D*/3 = *D*/3

S2 = initial load + share of new load = 1*D* + *D*/3 = 4*D*/3,

S1 = initial load + share of new load = 1*D* + *D*/3 = 4*D*/3

P3 = 1*D* load above (weight of slab 3) –load carried by slab 3 = 1*D* – *D*/3 = 2*D*/3

P2 = 2*D* load above (weight of slab 3 + weight of slab 2) – (load carried by slab 3+ load carried by slab 2) = 2*D* – (*D*/3 + 4*D*/3) = *D*/3

P1 = 3*D* load above (weight of slab 3 + weight of slab 2 + weight of slab 1) – (load carried by slab 3 + load carried by slab 2 + load carried by slab 1) = 3*D* – (*D*/3 + 4*D*/3 + 4*D*/3) = 0. This is shown in Fig. 18.29.

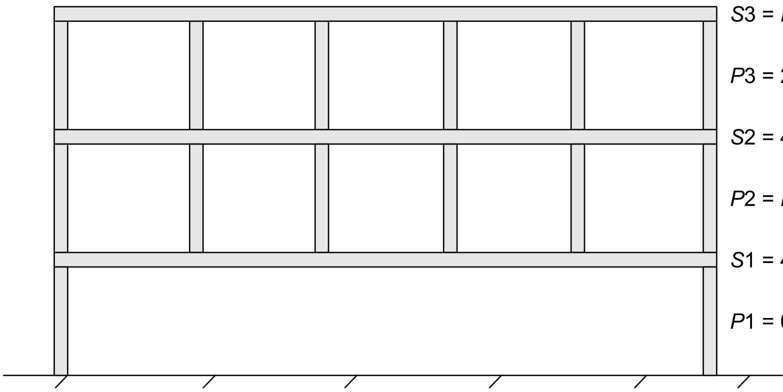

Figure 18.29 Removal of Reshores Under Slab 1 on Day 26.

The shores under slab 2 are removed (Fig. 18.30). The load carried by shores (*P2* = *D*/3) under slab 2 is distributed on two interconnected slabs 3 and 2. Thus each of the two slabs would have to carry an additional load of *D*/6. The load on slabs and shores are computed below:

S3 = initial load + share of new load = *D*/3 + *D*/6 = *D*/2

S2 = initial load + share of new load = 4*D*/3 + *D*/6 = 3/2*D*,

S1 = initial load - load carried by shores under slab 2 = 4*D*/3 – *D*/3 = 1*D*

P3 = 1*D* load above (weight of slab 3) – load carried by slab 3 = 1*D* – *D*/2 = *D*/2

P2 = 2*D* load above (weight of slab 3 + weight of slab 2) – (load carried by slab 3+ load carried by slab 2) = 2*D* – (*D*/2 + 3*D*/2) = 0 (as expected)

The reshores under slab 2 are placed (Fig. 18.31). These reshores act as struts. Since the reshores are snugly placed, there is no change in the load distribution at this stage. The load on slabs and shores are:

S3 = *D*/2, *S2* = 3/2 *D*, *S1* = 1*D*, *P3* = *D*/2, *P2* = 0

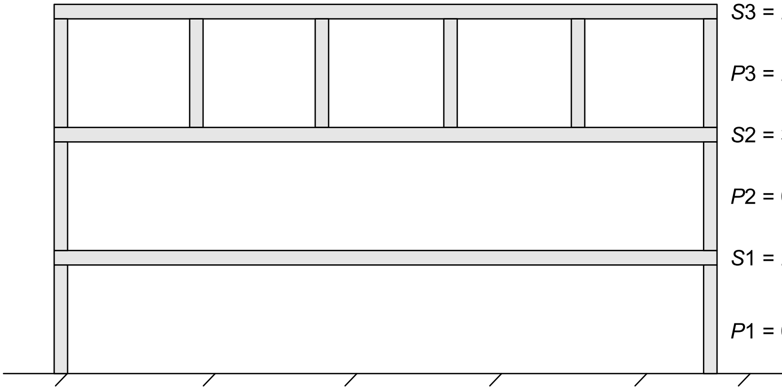

Figure 18.30 Removal of Shores Under Slab 2 on Day 26.

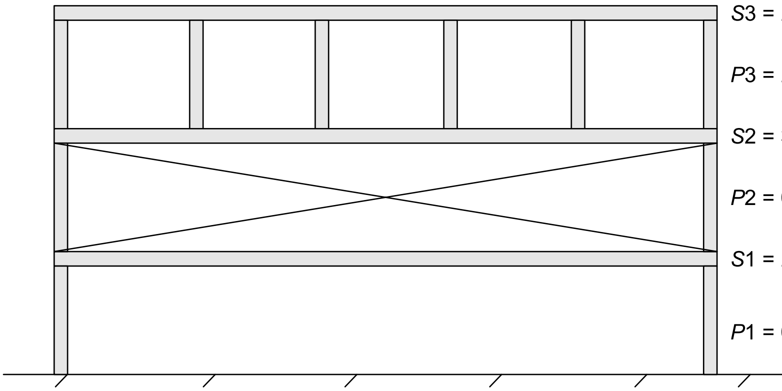

Figure 18.31 Installation of Reshores Under Slab 2 on Day 26.

On day 28, slab 4 is cast. The load D of slab 4 is carried equally among the three interconnected slabs 3, 2, and 1. The shores under slabs 4 and 3, and reshores under slab 2 carry that portion of the load above them not carried by the slabs. The load carried by the slabs and shores at this stage are computed below (also see Fig. 18.32).

$S4$ = initial load + share of new load = 0 (newly cast slab)

$S3$ = initial load + share of new load = $D/2 + D/3 = 5D/6$

$S2$ = initial load + share of new load = $3/2\ D + D/3 = 11D/6$,

$S1$ = initial load + share of new load = $1D + D/3 = 4D/3$

$P4 = 1D$ load above (weight of slab 4) –load carried by slab 4 = $1D - 0 = 1D$

$P3 = 2D$ load above (weight of slab 4 + weight of slab 3) – (load carried by slab 4 + load carried by slab 3) = $2D - (0 + 5D/6) = 7D/6$

$P2 = 3D$ load above (weight of slab 4 + weight of slab 3 + weight of slab 2) – (load carried by slab 4 + load carried by slab 3 + load carried by slab 2) = $3D - (0 + 5D/6 + 11D/6) = D/3$.

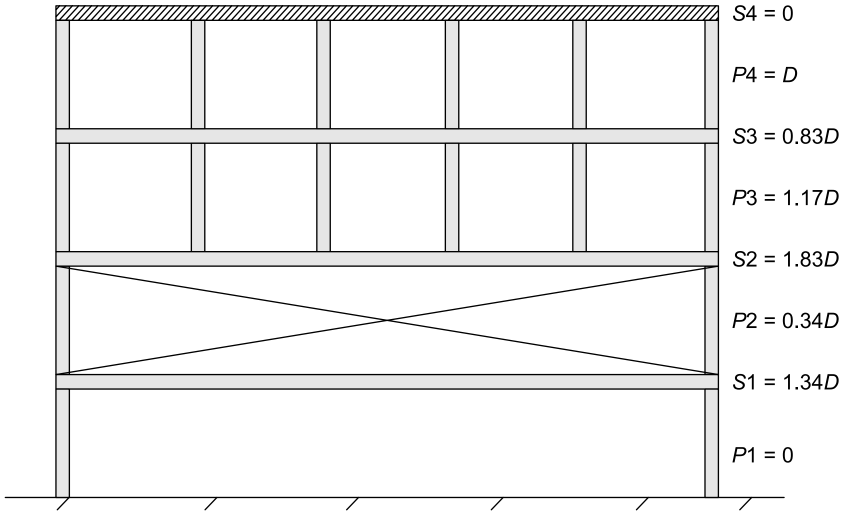

Figure 18.32 Casting of Slab 4 on Day 28.

The reshores under slab 2 are removed (Fig. 18.33). The load carried by shores ($P2 = D/3$) under slab 2 is distributed on three interconnected slabs 4, 3, and 2. Thus each of the three slabs will have to carry an additional load of $D/9$ (one third of $D/3$). The load on the slabs and shores are computed below:

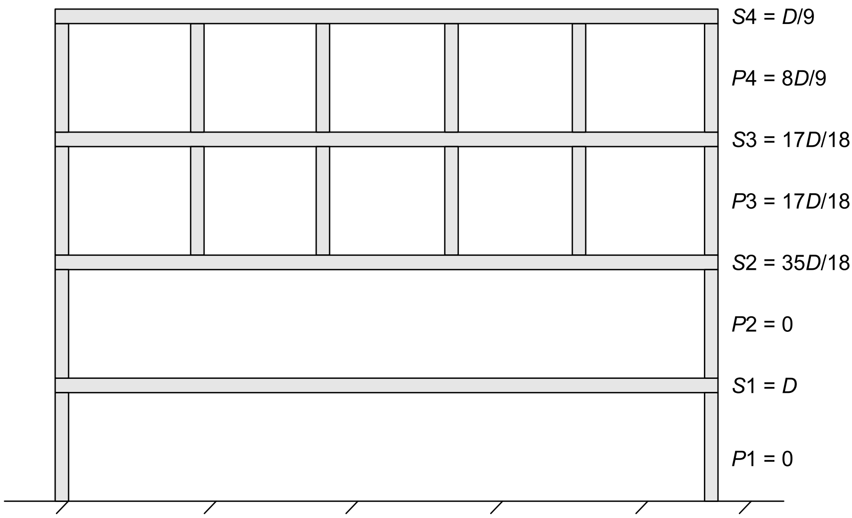

Figure 18.33 Removal of Reshores Under Slab 2 on Day 33.

$S4$ = initial load + share of new load = $0 + D/9 = D/9$

$S3$ = initial load + share of new load = $5D/6 + D/9 = 17D/18$,

$S2$ = initial load + share of new load = $11D/6 + D/9 = 35D/18$

$P4 = 1D$ load above (weight of slab 4) –load carried by slab 4 = $1D - D/9 = 8D/9$

$P3 = 2D$ load above (weight of slab 4 + weight of slab 3) – (load carried by slab 4 + load carried by slab 3) = $2D - (D/9 + 17/18D) = 17D/18$

$P2 = 3D$ load above (weight of slab 4 + weight of slab 3 + weight of slab 2) – (load carried by slab 4 + load carried by slab 3+ load carried by slab 2) = $3D - (D/9 + 17/18D + 35/18D) = 3D - 3D = 0$ (as expected)

The shores under slab 3 are removed (Fig. 18.34). The load carried by shores ($P3 = 17D/18$) under slab 3 is distributed on two interconnected slabs 4 and 3. Thus each of the two slabs would have to carry an additional load of $17D/36$. The load on the slabs and shores are computed below:

$S4$ = initial load + share of new load = $D/9 + 17/36D = 21D/36$

$S3$ = initial load + share of new load = $17/18D + 17/36D = 51D/36$,

$S2$ = initial load – load carried by shores under slab 3 = $35/18D - 17/18D = 1D$

$P4 = 1D$ load above (weight of slab 4) – load carried by slab 4 = $1D - 21/36D = 15D/36$

$P3 = 2D$ load above (weight of slab 4 + weight of slab 3) – (load carried by slab 4 + load carried by slab 3) = $2D - (21/36D + 51/36\ D) = 0$ (as expected)

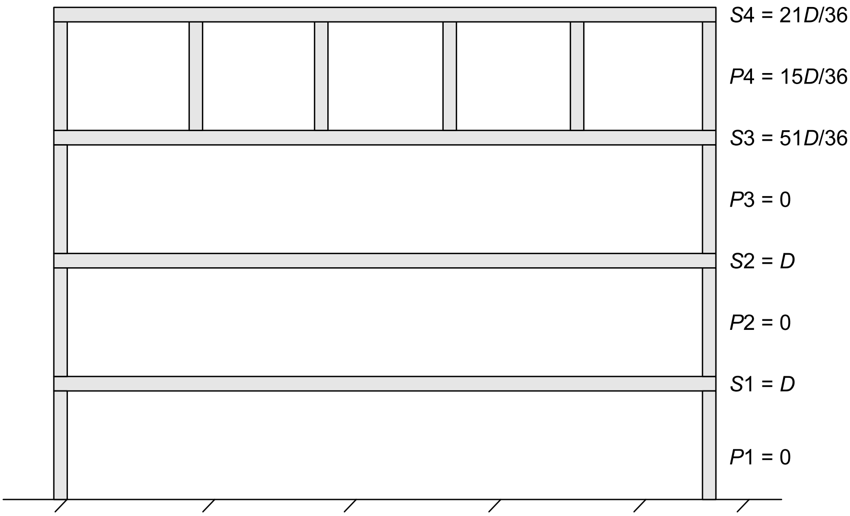

Figure 18.34 Removal of Shores Under Slab 3 on Day 33.

In a similar manner, the load on slabs, shores, and reshores can be computed. For easy reference, some more steps are shown in Fig. 18.35 along with the steps explained earlier. As can be seen from Fig. 18.35, after certain steps, the loads on slabs, shores, and reshores converge.

18.5.2 Two Levels of Shores and One Level of Reshores— Dead Weight of Slab, Self Weight of Shores and Reshores, and Construction Live Loads Considered

Let's assume the following:

Self weight of the slab is D

Self weight of the forms and shores is $D/10 = 0.1D$

Self weight of reshores is $D/20 = 0.05D$

Construction live load is $D/2 = 0.5D$

Let's also assume that live load is removed after the concrete is cast.

As before, let's assume that work starts on day 1 and it takes six days to complete the installation of shores, formwork, and reinforcement for slab 1. On day 7, slab 1 is cast (Fig. 18.36). At this point, the entire weight of slab D, the construction live load $0.5D$, and the self weight of forms and shores $= 0.1D$ is taken by shores under slab 1 and is transmitted to the ground. Thus, the load carried by slab 1 is 0 and the load carried by shores is $1.6D$ (sum of $1D$, $0.1D$, and $0.5D$).

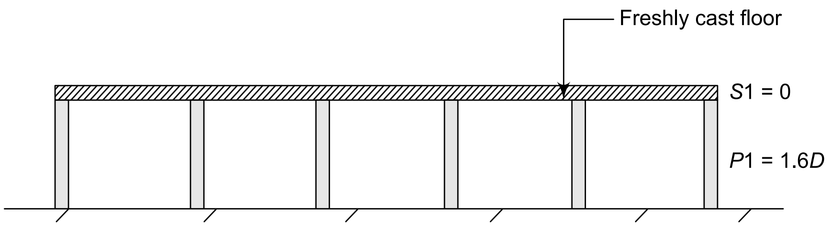

Figure 18.36 Casting of Slab 1 on Day 7.

Considering the cycle time of seven days, it is assumed that slab 2 is cast on day 14. All the loads of slab 1 and slab 2 go through shores to the ground. The load carried by shores under slab 2 is $1.6D$ as before and the load carried by shores under slab 1 is $1.6D + 1.1D = 2.7D$. It may be noted that the construction live load is no longer assumed to be on slab1 and that is why the load on shores under slab 1 is $2.7D$ ($1.6D + 1.1D$) and not $3.2D$ ($1.6D + 1.6D$). The loads on shores and slabs are indicated in Fig. 18.37.

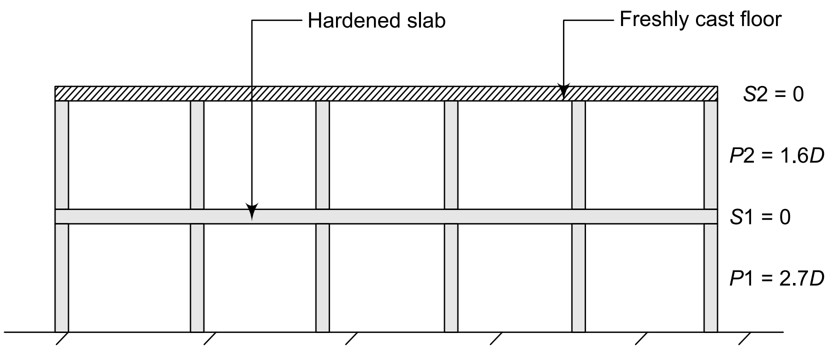

Figure 18.37 Casting of Slab 2 on Day 14.

The construction load is assumed to be removed before proceeding for stripping and reshoring. At this stage, the loads on shores and slabs are as shown in Fig. 18.38. It may be noted that the load on shores under slab 2 is $1.1D$ and not $1.6D$ for the reason explained earlier.

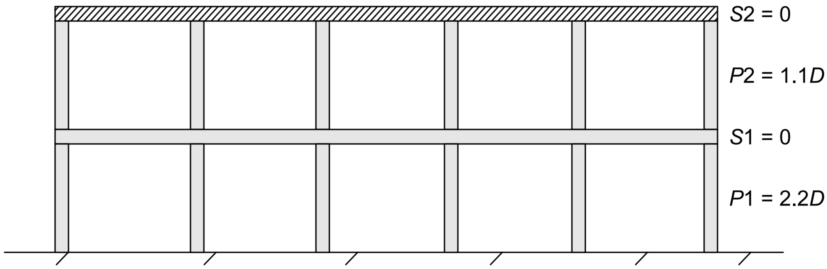

Figure 18.38 Removal of Construction Load After Casting of Slab 2 is Over.

Shores under slab 1 is removed on the 19th day ($2T + m$ days, where $T = 7$, and m = stripping period = 5). Thus $0.1D$ weight of forms and shores are no longer applicable in the system. The revised shore load of $P1 = 2.1D$ ($2.2D$-$0.1D$) will be carried equally by slabs 1 and 2. Thus, $S2 = 1.05D$, $P2 = 1.1D - 1.05D = 0.05D$, $S1 = 1.05D$, and $P1 = 0$ (Fig. 18.39).

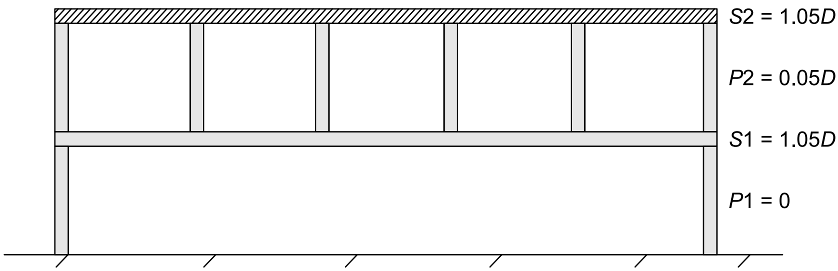

Figure 18.39 Removal of Shores Under Slab 1 on Day 19.

Now the reshores are placed snugly under slab 1. Due to snug placement of reshores, they are still not carrying any load other than their self weight of $0.05D$. The revised load distribution in view of this would be thus as below:

The slab loads $S2 = 1.05D$, $P2 = 0.05D$, $S1 = 1.05D$, and $P1 = 0.05D$ (Fig. 18.40)

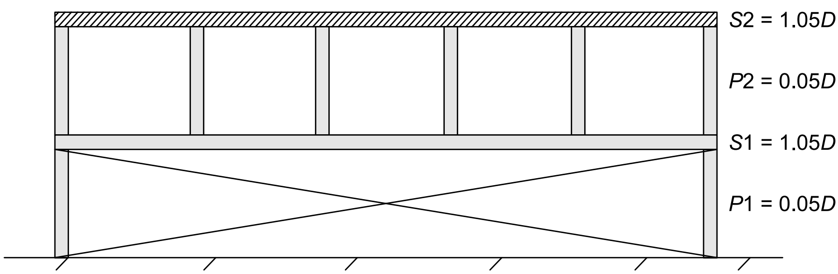

Figure 18.40 Installation of Reshores Under Slab 1 on Day 19.

On day 21, slab 3 is cast. The load $1.6D$ of slab 3 is carried to the ground by shores under slabs 3 and 2 and reshores under slab 1. Since the slabs can't deflect further, they do not carry any new load as of now. The load carried by slabs and shores at this stage are: $S3 = 0$, $P3 = 1.6D$, $S2 = 1.05D$, $P2 = 2.7D\text{-}1.05D = 1.65D$, $S1 = 1.05D$, and $P1 = 3.75D\text{-}0\text{-}1.05D\text{-}1.05D = 1.65D$. This is shown in Fig. 18.41.

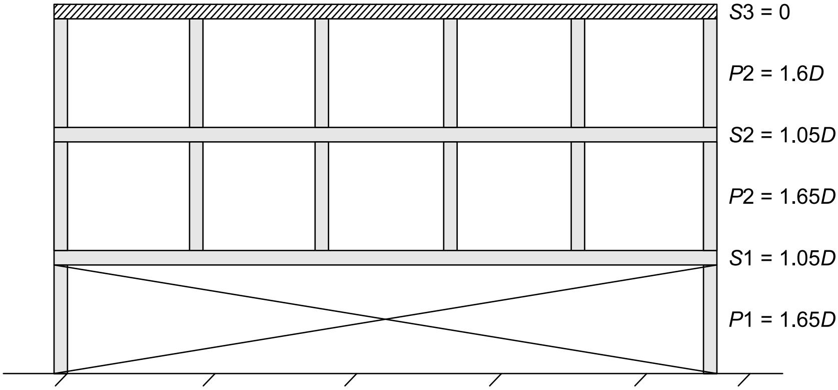

Figure 18.41 Casting of Slab 3 on the Day 21.

As before, the construction load is removed before proceeding for stripping and reshoring. At this stage, the loads on shores and slabs are as shown in Fig. 18.42. It may be noted that the load on shores under slab 3 is 1.1 *D* and not 1.6 *D* for the reason explained earlier. The load carried by slabs and shores at this stage are: $S3 = 0$, $P3 = 1.1D$, $S2 = 1.05D$, $P2 = 2.2D\text{-}1.05D = 1.15D$, $S1 = 1.05D$, and $P1 = (1.1\ D + 1.1\ D + 1D - 0 - 1.05D - 1.05D + 0.05D = 1.15D)$. This is shown in Fig. 18.42.

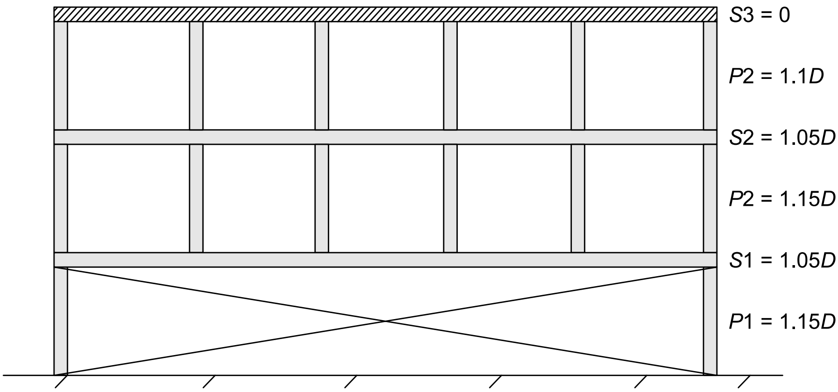

Figure 18.42 Removal of Construction Load on Slab 3 After Casting is Complete.

Reshores under slab 1 are removed (Fig. 18. 43). This action will cause all the three slabs *S3*, *S2*, and *S1* to deflect together. The reshores load of $P1 = 1.15D - 0.05D$ (self weight of reshores) $= 1.1D$ is distributed equally among three interconnected slabs 3, 2, and 1. The shores under slabs 3 and 2 carry that portion of the load above them not carried by the slabs. The load carried by slabs and shores at this stage are:

$S3$ = initial load + share of new load $= 0 + 1.1D/3 = 0.37D$

$S2$ = initial load + share of new load $= 1.05D + 1.1D/3 = 1.42D$

$S1$ = initial load + share of new load $= 1.05D + 1.1D/3 = 1.41D$

$P3 = 1.1D$ load above (weight of slab 3) – load carried by slab 3 $= 1.1D - 0.37D = 0.73D$

P2 = 2.2*D* load above (weight of slab 3 + weight of slab 2) – (load carried by slab 3 + load carried by slab 2) = 2.2*D* – (0.37*D* + 1.42*D*) = 0.41*D*

P1 = 3.3*D* load above (weight of slab 3 + weight of slab 2 + weight of slab 1) – (load carried by slab 3 + load carried by slab 2 + load carried by slab 1) = (1.1*D* + 1.1*D* + 1*D* = 3.2*D*) – (0.37*D* + 1.42*D* + 1.41*D*) = 0 (as expected). This is shown in Fig. 18.43.

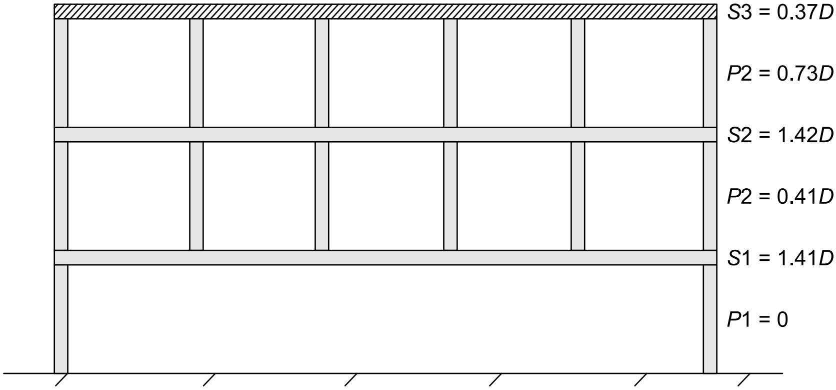

Figure 18.43 Removal of Reshores Under Slab 1 on Day 26.

The shores under slab 2 are removed (Fig. 18.44). The load carried by shores (*P2* = 0.41*D* – 0.1*D* = 0.31*D*) under slab 2 is distributed on two interconnected slabs 3 and 2. Thus each of the two slabs would have to carry an additional load of 0.31*D*/2. The load on slabs and shores are computed below:

S3 = initial load + share of new load = 0.37*D* + 0.31*D*/2 = 0.52*D*

S2 = initial load + share of new load = 1.42D + 0.31D/2 = 1.58 *D*

S1 = initial load - load carried by shores under slab 2 = 1.41*D* – 0.41*D* = 1*D*

P3 = 1.1*D* load above (weight of slab 3) – load carried by slab 3 = 1.1*D* – 0.52*D* = 0.58*D*

P2 = 2.1*D* load above (weight of slab 3 + weight of slab 2) – (load carried by slab 3 + load carried by slab 2) = 2.1*D* – (0.52*D* + 1.58 *D*) = 0 (as expected)

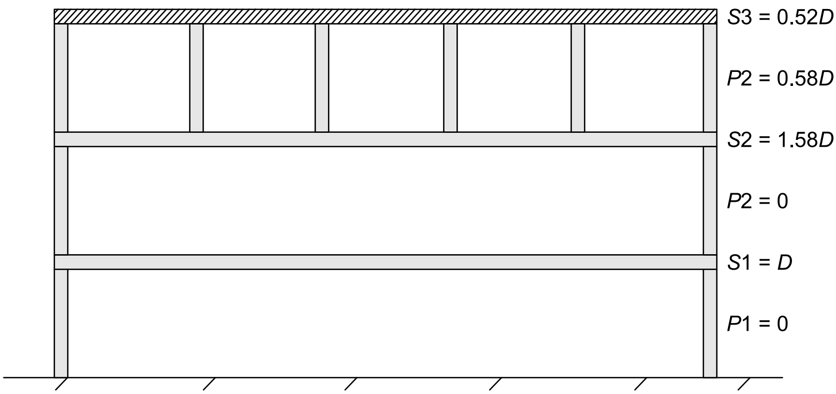

Figure 18.44 Removal of Shores Under Slab 2 on Day 26.

The reshores under slab 2 are placed (Fig. 18.45). These reshores act as struts. Since the reshores are snugly placed, there is no change in the load distribution except for the added weight of reshores. The load on slabs and shores are:

$S3 = 0.52D$; $S2 = 1.58D$; $S1 = 1.05D$; $P3 = 0.58D$; $P2 = 0.05D$

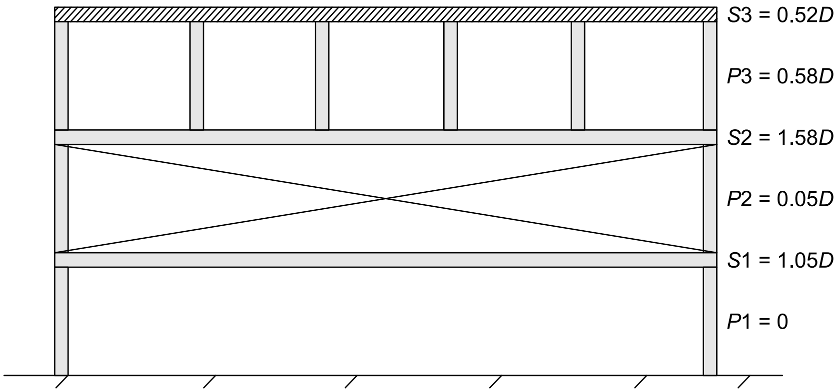

Figure 18.45 Installation of Reshores Under Slab 2 on Day 26.

On day 28, slab 4 is cast. The load $1.6D$ of slab 4 is carried equally among the three interconnected slabs 3, 2, and 1. The shores under slabs 4, and 3 and reshores under slab 2 carry that portion of the load above them not carried by the slabs. The load carried by the slabs and shores at this stage are computed below (also see Fig. 18.46).

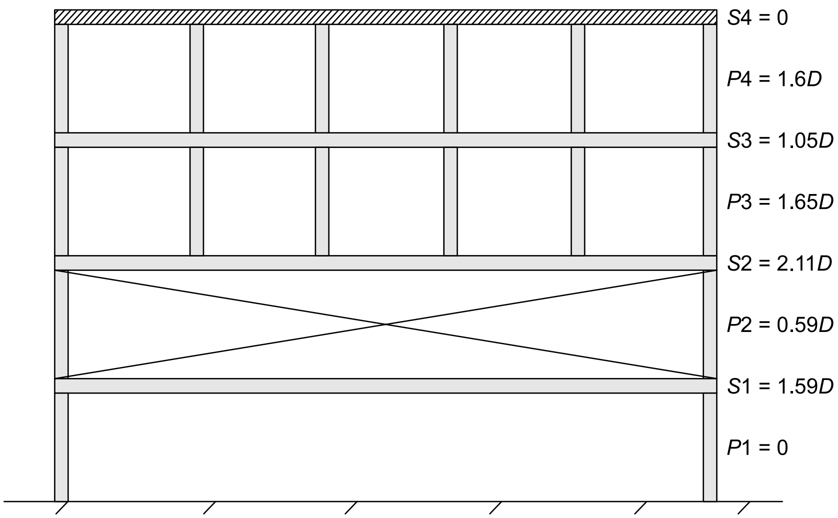

Figure 18.46 Casting of Slab 4 on Day 28.

$S4$ = initial load + share of new load = 0 (Freshly cast slab)

$S3$ = initial load + share of new load = $0.52D/2 + 1.6D/3 = 1.05D$

$S2$ = initial load + share of new load = $1.58D + 1.6D/3 = 2.11D$

$S1$ = initial load + share of new load = $1.05D + 1.6D/3 = 1.59D$

$P4 = 1.6D$ load above (weight of slab 4) – load carried by slab 4 = $1.6D - 0 = 1.6D$

$P3 = 2.7D$ load above (weight of slab 4 + weight of slab 3) – (load carried by slab 4 + load carried by slab 3) = $2.7D - (0 + 1.05D) = 1.65D$

$P2 = 3.7D$ load above (weight of slab 4 + weight of slab 3 + weight of slab 2 = 1.6 + 1.1 + 1.0) + self weight of reshores – (load carried by slab 4 + load carried by slab 3 + load carried by slab 2) = $3.7D + 0.05D - (0 + 1.05D + 2.11D) = 0.59D$.

In a similar manner, the load on slabs, shores, and reshores can be computed. For easy reference, some more steps are shown in Fig. 18.47 along with the steps explained earlier. As can be seen from Fig. 18.47, after certain steps, the loads on slabs, shores, and reshores converge.

18.6 LIMITATIONS OF SIMPLIFIED ANALYSIS AND DISCUSSION ON OTHER DEVELOPMENTS

The simplified load distribution presented in the previous sections was based on single plane analysis and numerous other assumptions. Later, a number of researchers contributed in the computation of loads on slabs, shores, and reshores during construction. In some cases, the analysis was based on two planes at right angles, and in some studies, the analysis was based on 3D (three dimensional) methods. In one study, the shores were treated as concrete columns and an equivalent frame method of analysis was used to predict the load distributions. In another study, the stiffness of shores was considered, while in one of the studies, the structural characteristic parameter approach was used to estimate the load distributions and the maximum slab loads during construction.

Apart from theoretical analysis, some studies utilized extensive experimental set ups and field studies to actually measure the loads on slabs and shores. For example, Agarwal and Gardner (1974) conducted field measurements wherein they found that the field measurements of the construction loads agreed to an acceptable accuracy with those predicted by the simplified method.

Some computer models have also been developed based on the assumptions used for simplified analysis. For example, Liu *et al*. (1988) developed a computer based computational procedure for rapid calculation of construction loads imposed on slabs with shored formwork. The program can also quickly examine the slab safety at any stage of construction. In other words, the program can determine whether the slab loads are greater than the available strength at each step during construction in multi-story buildings. In another study based on two and three dimensional computer models, it was concluded that the simplified method is adequate for predicting the construction step and the location of the maximum slab movements and shore loads. In some studies, modification coefficients have also been suggested for multiplying with loads obtained using the simplified method.

Some studies were conducted to understand the effect of fast construction rate on short- and long-term deflections of RC buildings. It was concluded that a fast construction rate will not affect the short or long-term deflections provided high strength concrete and high early strength cement are used.

Most of the studies have found that the load distribution and the location of the maximum slab load are close to the simplified analysis presented for various construction techniques and thus acceptable for practical applications. A detailed discussion on this subject can be found in Ghosh (1997).

18. 7 COMPUTATION OF STRENGTH OF CONCRETE SLAB AT A GIVEN POINT OF TIME

The available strength depends mainly on the concrete age, the type of cement, and the construction temperature. The strength of the concrete structures designed on the basis of strength, can be determined assuming that allowable load during construction is a function of the ultimate strength capacity of the matured concrete structures. Given the type of cement and the construction temperature, the strength can be expressed as a function of the concrete age alone.

The 28-day design ultimate load carrying capacity of the slabs can be computed as:

$$U_{28} = \gamma_1 \times \text{D.L.} + \gamma_2 \times \text{L.L.} \tag{18.1}$$

where,

D.L. is the slab dead load, weight of building partitions and mechanical equipment,

L.L. is the design live load on the slab.

γ_1 and γ_2 are partial safety factors; the ACI suggested values of γ_1 and γ_2 are 1.4 and 1.7, respectively. The values of γ_1 and γ_2 are equal to 1.5 according to IS: 456–2000.

The strength available at early ages x ($x < 28$ days) is less than the 28-day strength.

The strength at x day is computed by:

$$U_x = \beta \times U_{28} \tag{18.2}$$

where β = modification factor which is some percentage of 28 days strength developed at x days.

The value of β can be determined from Figs. 18.48 and 18.49 for Type I and Type II cements corresponding to different curing temperatures. The value of β can be found out from Fig. 18.50 also which is based on the Indian Standard recommendations.

It may be pointed out here that the Figs. 18.48, 18.49, and 18.50 are for illustration purpose only. For field application, the form designer should use data based on his concrete mix and curing temperature. The rate of strength development varies depending on the concrete mix and curing temperature.

The estimated ultimate load capacity so derived, is divided by a factor of safety to compute the allowable load during construction. Not much work has been carried out to establish the factor of safety for a concrete building under construction. However some researchers suggest the factor of safety to be 1.3 or 1.4.

Thus the allowable load at x days is given by:

$$\textit{Allowable load at x days} = \frac{U_x}{\textit{Factor of safety}} \tag{18.3}$$

The allowable load is compared with the expected load coming on the slabs for a particular construction method and it should be made sure that the latter is always less than the former. In case the allowable load is exceeded, suitable adjustments can be made. For example, changes can be made in the following parameters:

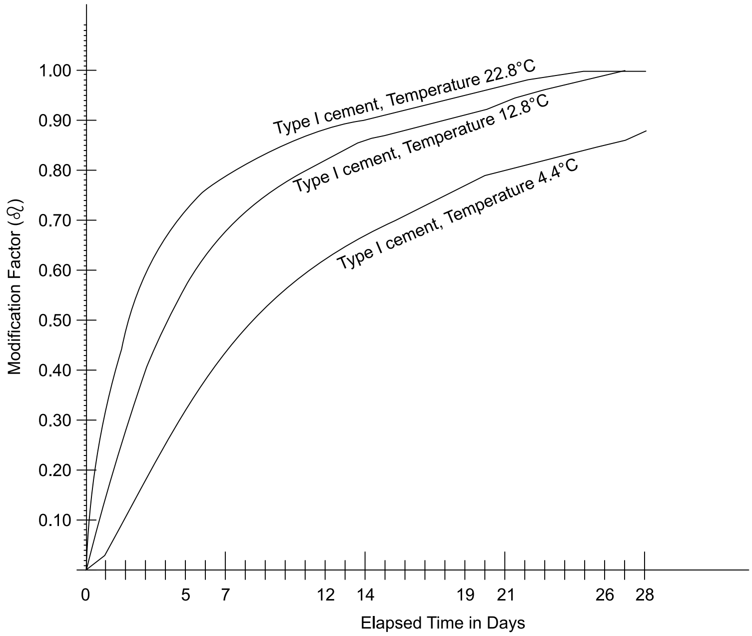

Figure 18.48 Development of Concrete Strength with Time (Type I Cement).

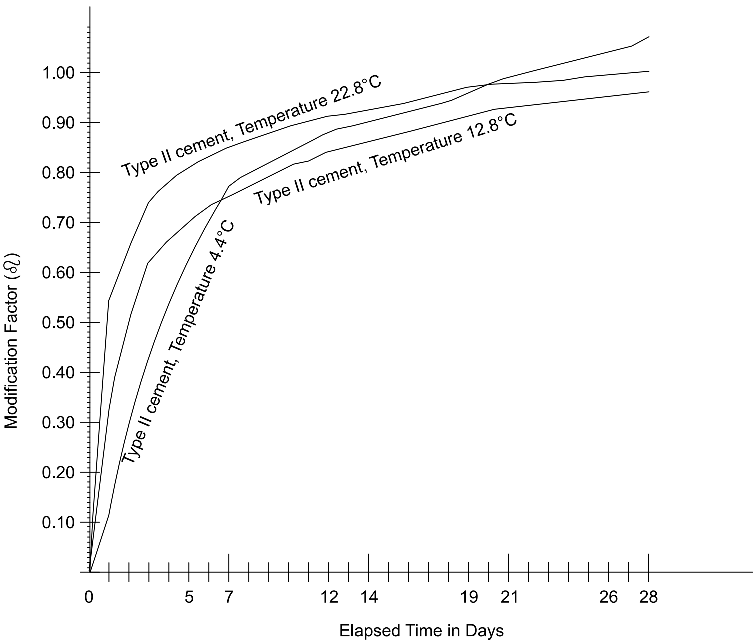

Figure 18.49 Development of Concrete Strength with Time (Type II Cement).

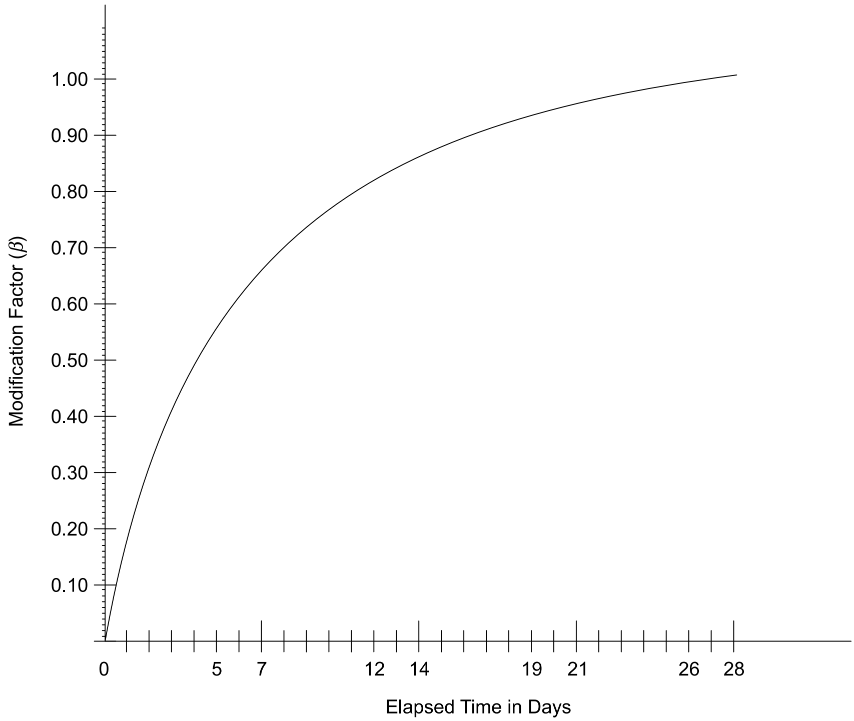

Figure 18.50 Development of Concrete Strength with Time Based on IS Code.

1. the construction cycle;
2. the period for shore and reshore removal;
3. number of levels of shores and reshores, etc.

The changes can be made individually or in combination of the mentioned parameters.

18.8 ILLUSTRATION

Examples are provided to illustrate the process of ensure the safety of the slab at each step of construction. The procedure illustrated can be used to answer questions such as: whether the estimated loads based on a construction technique are adequate and the loads exerted on the slab are below the allowable load. If not, how to ensure that the loads exerted on a slab are less than the allowable loads at all stages.

Example 1: Slab thickness = 200 mm; it is assumed that Type I cement is used and concrete curing temperature = 12.8°C, The concrete is designed to carry a live load of 2.4 kN/m^2 and 0.72 kN/m^2 for mechanical systems and partitions.

As before, let's assume the weight of slab = 1.00D

Weight of forms and shores = 0.10D

Weight of reshores = 0.10D

$$\text{Construction live load} = 0.50D$$
$$N = 15$$

Assume γ_1 and γ_2 are 1.4 and 1.7 respectively.

Construction technique: two floors of shoring and one floor of reshoring

Check the safety of slabs in both cases: (a) factor of safety = 1.3 for concrete buildings under construction and (b) factor of safety = 1.4 for concrete building under construction if a cycle time of 4 days is considered. Check the safety with a revised cycle time of 8 days. Suggest suitable cycle time to be considered if it is desired to work without changing the setup of concrete mix, concrete curing temperature and the construction technique.

Such problems are solved in three broad steps.

In step 1, for the assumed construction technique, load distribution on slabs, shores, and reshores are established using the simplified method. In step 2, the allowable load on slab is worked out at different days. In step 3, comparison is made between the values obtained in step 1 and step 2 and the various parameters are decided. The stepwise solution is provided for the above problem.

Step 1: Load distribution for two levels of shores and one level of reshores

The estimated slab load using two levels of shores and one level of reshores are taken from Fig. 18.47-(9)(Left), which was derived considering the slab dead weight as D, construction live load of 0.5 D, shore and form weight of 0.1 D, and reshore weight of 0.05 D.

Considering a cycle time of T= 4 days, the estimated load at 0, 4, 8, 12, ….. days in terms of D is shown in Table 18.2.

Table 18.2 Estimated loads on slabs at different ages

Slab number	Age of slab (days)	Estimated load
Slab 4	0	0
Slab 3	4	1.05D
Slab 2	8	2.11D
Slab 1	12	1.59D

Step 2: Allowable loads on the slabs

The ultimate design load capacity of the slab at 28 days is:

$$U_{28} = \gamma_1 \times D.L. + \gamma_2 \times L.L.$$
$$D.L. = 24 \times 0.2 + 0.72 = 5.52 \text{ kN/m}^2$$
$$D = \text{self weight of slab} = 24 \times 0.2 = 4.8 \text{ kN/m}^2$$
$$L.L. = 2.4 \text{ kN/m}^2$$
$$\gamma_1 = 1.4,\ \gamma_2 = 1.7$$

Thus,

$$U_{28} = 1.4 \times 5.52 + 1.7 \times 2.4 = 11.808 \text{ kN/m}^2$$

Expressed in terms of the self weight of 200 mm thick slab,

$$U_{28} = \frac{11.808}{4.8} D = 2.46D$$

The strength of the slab at = 0, 4, 8, 12, …… days can be estimated using,

$$U_x = \beta \times U_{28}$$

The values of β at x = 0, 4, 8, 12 …. days are taken from Fig. 18.48, corresponding to the given curing temperature of 12.8°C. The values are shown in Table 18.3 along with the estimated ultimate load capacity at x days (U_x) and the estimated allowable load during construction based on the factor of safety (U_x/Factor of safety) values 1.3 and 1.4.

Step 3: Comparison between the values obtained in step 1 and step 2

It can be noted from Table 18.3, that the estimated allowable load at 4, 8, 12, and 16 days are 0.93*D*, 1.36*D*, 1.55*D*, and 1.66*D*, respectively, for a factor of safety of 1.3. Comparing these values with 0, 1.05*D*, 2.11*D*, and 1.59*D* (the estimated load obtained from Table 18.2), it can be inferred that the estimated allowable load has been exceeded for all the slabs, and thus, 4-day cycle is not possible with the given sets of parameters.

Table 18.3 Comarison of allowable and estimated loads (Cycle time 4 days, Factor of safety = 1.3)

x	β	$U_x = \beta \times U_{28}$	$\frac{U_x}{\text{factor of safety}} = \frac{U_x}{1.3}$	Remarks (Compare with values obtained from Table 18.2)
0	0	$U_0 = 0 \times 2.46D = 0$	0	
4	0.49	$U_4 = 0.49 \times 2.46D = 1.21D$	$1.21/1.3 = 0.93D$	Less than 1.05*D*, thus unsafe
8	0.72	$U_8 = 0.72 \times 2.46D = 1.77D$	$1.77D/1.3 = 1.36D$	Less than 2.11*D*, thus unsafe
12	0.82	$U_{12} = 0.82 \times 2.46D = 2.02D$	$2.02D/1.3 = 1.55D$	Less than 1.59*D*, thus unsafe
16	0.88	$U_{16} = 0.88 \times 2.46D = 2.16D$	$2.16D/1.3 = 1.66D$	
20	0.92	$U_{20} = 0.92 \times 2.46D = 2.26D$	$2.26D/1.3 = 1.74D$	
24	0.97	$U_{24} = 0.97 \times 2.46D = 2.39D$	$2.39D/1.3 = 1.84D$	
28	1.00	$U_{28} = 1.00 \times 2.46D = 2.46D$	$2.46D/1.3 = 1.89D$	

Similar observations can be made for the factor of safety of 1.4. This is clearly mentioned in the last column of Table 18.4.

Table 18.4 Comarison of allowable and estimated loads (Cycle time 4 days, Factor of safety = 1.4)

x	β	$U_x = \beta \times U_{28}$	$\frac{U_x}{\text{factor of safety}} = \frac{U_x}{1.4}$	Remarks (Compare with values obtained from Table 18.2)
0	0	$U_0 = 0 \times 2.46D = 0$	0	
4	0.49	$U_4 = 0.49 \times 2.46D = 1.21D$	$1.21/1.4 = 0.86D$	Less than 1.05*D*, thus unsafe
8	0.72	$U_8 = 0.72 \times 2.46D = 1.77D$	$1.77D/1.4 = 1.26D$	Less than 2.11*D*, thus unsafe
12	0.82	$U_{12} = 0.82 \times 2.46D = 2.02D$	$2.02D/1.4 = 1.44D$	Less than 1.59*D*, thus unsafe
16	0.88	$U_{16} = 0.88 \times 2.46D = 2.16D$	$2.16D/1.4 = 1.54D$	
20	0.92	$U_{20} = 0.92 \times 2.46D = 2.26D$	$1.61D$	
24	0.97	$U_{24} = 0.97 \times 2.46D = 2.39D$	$1.71D$	
28	1.00	$U_{28} - 1.00 \times 2.46D = 2.46D$	$1.76D$	

Thus, for both cases, we find that the structure is not safe to carry the estimated loads. There is a need to change some parameter. The change could be made in the construction technique, cycle time, either one at a time or in combination. To illustrate the change, let's assume a revised cycle time of 8 days. The estimated loads on slabs remain unchanged with the same construction technique of two levels of shores and one level of reshores. The estimated allowable load will however change. This can be determined from Fig.18.48, and is produced in Table 18.5. Let's perform the calculation for a factor of safety of 1.3 alone.

Table 18.5 Comarison of allowable and estimated loads (Cycle time 8 days, Factor of safety = 1.3)

x	β	$U_2 = \beta \times U_{28}$	$\dfrac{U_x}{\text{factor of safety}} = \dfrac{U_x}{1.3}$	Remarks (Compare with values obtained from Table 18.2)
0	0	$U_0 = 0 \times 2.46D = 0$	0	0 (Slab 4)
8	0.72	$U_8 = 0.72 \times 2.46D = 1.77D$	$1.77D/1.3 = 1.36D$	More than 1.05D thus safe (Slab 3)
16	0.88	$U_{16} = 0.88 \times 2.46D = 2.16D$	$2.16D/1.3 = 1.66D$	**Less than 2.11D thus unsafe (Slab 2)**
24	0.97	$U_{24} = 0.97 \times 2.46D = 2.39D$	$2.39D/1.3 = 1.84D$	More than 1.59D thus safe (Slab 1)
28	1.00	$U_{28} = 1.00 \times 2.46D = 2.46D$	$2.46D/1.3 = 1.89D$	

Table 18.5 shows that the slabs, especially 3 and 1 are now better equipped to sustain the estimated loads on them; however, slab 2 is still overloaded (overload is 2.11 D-1.66D = 0.45D).

A careful examination of Fig. 18.47 shows a maximum estimated slab load of 2.07D (Fig. 18.47-17), but the maximum load allowed on any slab during construction never exceeds its allowable 1.76D (see Table 18.4 last row fourth column) based on 28-day strength adjusted to allow a factor of safety of 1.4 and 1.89D (see Table 18.5 last row fourth column) to allow a factor of safety of 1.3. Therefore the assumed structure cannot be built with these safety factors, using two levels of shores and one level of reshores. Measures such as: increasing more levels of shores and reshores, further increase in cycle time, higher curing temperature, improved concrete mix with enhanced early strength development, etc. can be adopted either in isolation or in different combinations of the stated parameters.

REVIEW QUESTIONS

Q1. Let's assume the weight of slab = 1.00D, Weight of forms and shores = 0.10D

Weight of reshores = 0.10D, Construction live load = 0.50D, N =15

Factor of safety = 1.3 for concrete buildings under construction. It is assumed that Type I cement is used, Temperature = 22.8°C. Construction technique: two floors of shoring and two floors of reshoring. Check the safety and recommend the suitable cycle time for the given condition.

Hint: For a construction technique of two floors of shoring and two floors of reshoring, cycle time of 7 days, casting period of each concrete slab = 1 day, and concrete curing period of 5 days, it can be observed that slabs 4 through 13 will be in danger during construction. To avoid the failure, the number of floors of shores or reshores or the construction cycle must be changed.

For a construction technique of two floors of shoring and three floors of reshoring, cycle time of 7 days, casting period of each concrete slab = 1 day, and concrete curing period of 5 days, it will be noticed that the loads on slabs would not be exceeded beyond the available strength of the slab and thus the failures can be avoided at the time of construction. The slabs would still be safe if the following changes are made: cycle time = 14 days, casting period of each concrete slab = 2 days, and concrete curing period = 10 days.

Q2. A construction technique using two levels of shores and one level of reshores is proposed. Concrete curing temperature of $T = 22.8°C$ are used. Two construction cycles of 4 and 7 days per floor are proposed. Check the safety and recommend the suitable cycle time for the given conditions.

Hint: It will be observed that the 4-day cycle is not acceptable and that a casting cycle of 7 days per floor is required.

Q3. A construction technique using two levels of shores and one level of reshores and a casting cycle of 7 days is proposed. Concrete curing temperatures of 22.8°C and 12.8°C are proposed. Which one would you recommend?

Hint: If the concrete can be maintained at only 12.8°C, the slab load will exceed the capacity at the age of 14 days.

Q4. Construction techniques using one, two or three levels of shores and one level of reshores are being proposed. A casting cycle of 5 days at a temperature of 15.6°C is planned. Which construction scheme should be recommended for the given conditions?

Hint: As can be seen, if three levels of shores are used, the maximum slab load, while larger than that for the other two techniques, is less than the predicted slab capacity.

Glossary of Formwork Related Terms

1.	Accessories	Accessories are small items other than the main items, such as frames, braces, or post shores. These are used to facilitate the construction of scaffold and shoring.
2.	Access Door (Access Trap or Inspection Door or Porthole or Trap Door)	Access door is a removable panel in the formwork for a high lift. This is also known as access trap, inspection door, or porthole, or trap door. This is provided to give (a) access for inspection or (b) for placing and compacting concrete.
3.	Adjustable Base Plate	Adjustable base plate is a base plate with provisions for vertical adjustment.
4.	Adjuster	Adjuster is a mechanical device provided for assuring correct line or level or both for formwork.
5.	Anchor	An anchor is a device for providing a fixing to a concrete surface.
6.	Anchor Bolt	An anchor bolt is a bolt which passes through the member to be anchored, and engages with the anchor.
7.	Anchor Plate	An anchor plate is a plate on the embedded end of an anchor. This increases the resistance of an anchor to being pulled out of the concrete.
8.	Angle Fillet (Corner Mould)	An angle fillet is a strip which is used to form an internal or external intersection which is to be other than a sharp angle.
9.	Arris	An arris is a sharp edge or protruding corner formed by meeting of two surfaces. The surfaces can be plane or curved. An arris is applied to edges in moldings and edges separating fluting.
10.	Back Form	Back form is a form for a concrete surface which will be unseen in the finished structure. For example, the outer form for a foundation wall.
11.	Back Propping	Propping provided to cast concrete floors to support formwork for higher floors.
12.	Ballies	Ballies are thin poles in the round usually without bark. These were very commonly used for supporting and aligning formwork in conventional construction.
13.	Bar Spacer	A bar spacer is a device for maintaining the correct position of reinforcement within the formwork.
14.	Base Plate	A base plate is a plate for distributing the load from an upright or raker.

15. Batter Boards	Batter boards are pairs of horizontal boards which are nailed to wood stakes adjoining an excavation. They act as a temporary reference markers to retain the location of the building lines while the foundation is being excavated. These are used as (a) a guide to elevations and (b) to outline the building.
16. Beam Clamp (Beam Cramp or Clamp, or Cramp)	Beam clamp is also known as beam cramp, clamp, or cramp. This is a yoke or other device for holding a beam box tightly against the pressure of freshly placed concrete.
17. Beam Hanger (Form Hanger or Beam Saddle)	Beam Hanger also known as Form Hanger or Beam Saddle is a support to the soffit form for the encasement to a steel joist where the form is hung from the joist itself. This is also defined as a wire, strap, or other hardware device that supports formwork from structural members.
18. Beam Sides	Beam sides are vertical side panels or parts of a beam form.
19. Bevel	Bevel is the intersection of one plane surface with another plane surfaces at an angle other than a right angle. By doing so, the panels or boards can easily be struck.
20. Bind	Bind is the adherence of forms to the concrete when the supports are removed and striking is attempted.
21. Block (Blocking Piece)	Block, also referred to as Blocking Piece, is a piece of wood or other material to pack out or separate, or; when glued, to stiffen other members used.
22. Blowhole	Blowhole is a small hole across in a concrete face caused by an air pocket.
23. Board	Board is a piece of square-sawn softwood/timber under 50 mm thick and 100 mm depth over wide. This is manufactured in rigid or semi-rigid sheets, such as laminated board, wood chipboard and other particle board or hard-board.
24. Box	Box is a formwork for a beam or column.
25. Box-Out	Box-out is the form for a pocket or aperture in concrete.
26. Bracing	Bracing is the system of members which stiffens a frame against deformation. This is usually diagonal, and acts in compression or tension.
27. Bridle	Bridle is a horizontal tube slung between putlogs for the purpose of supporting intermediate putlogs. This is required where due to window openings and the like it is impossible to support a putlog in the wall.
28. Bulkhead	Bulkhead is a vertical partition within the forms which blocks fresh concrete from a section of forms. At construction joints bulkhead closes the end of a form.
29. Camber	Camber is the intentional curvature of a beam or formwork. This is either formed initially to compensate for subsequent deflection under load or produced as a permanent effect for aesthetic reasons.
30. Carcassing Timber (Framing Timber)	Carcassing timber, also referred to as framing timber, is the timber used for any structural purpose in the support of the forms. This is normally not in contact with the concrete.
31. Cast-in-Socket	Cast-in-socket is an anchor which consists of a female-threaded metal socket cast into concrete as a fixing for a bolt.
32. Castor	Castor is a swiveling wheel attached to the lower end of a tubular column. This is provided for moving and supporting scaffolding.

33. Chamfer	Chamfer is the surface produced in concrete work by placing a three-corner piece of wood in the form corner. The surface produced is usually symmetrically, of an external edge.
34. Chase	Chase is a long groove or recess formed in the concrete surface.
35. Chase Form	Chase form is a form for molding a chase which is often a wooden molding planted on the face of the main form.
36. Check	Check is a small piece of timber fixed to the face of a form to indicate the top of a concrete lift and to form a clean line at the joint with the next lift.
37. Check Out	Check out is a piece of wood or other material fixed to the face of a form to form a recess in concrete.
38. Cleanout Trap (Cleanout Hole)	Cleanout trap or cleanout hole is a removable section at the base of the column or wall formwork. This allows rubbish to be removed before concreting from the inside of formwork.
39. Cleat	Cleat is a block fixed to a main member to provide a bearing or to resist a thrust. A small board used to connect formwork members or used as a brace is also referred as cleat.
40. Climbing Formwork (Moving Formwork) -	Climbing formwork also referred to as moving formwork is a formwork for vertical or near vertical structures which are constructed in successive lifts and which is supported by the previously poured lift.
41. Collapsible Forms	Collapsible forms are forms which get activated by mechanical means to reduce their volume or surface area in order to permit striking.
42. Column Box (Column Casing)	Column box also referred to as column casing is the assembled forms for a column.
43. Column Clamp (Column Cramp)	Column clamp also referred to as a column cramp is a yoke or other device which holds a column box tightly close against the pressure of freshly placed concrete.
44. Column Guard	Column guard is a length of metal angle cast into a corner of a column. This is provided to protect the corner of column against damage.
45. Cone	A cone, as the name indicates, is a cone-shaped piece of wood, rubber or other material. This is placed over the end of a form tie to form a neat hole in the surface of the concrete.
46. Coupler	Coupler is a fitting by which a grip is applied to the external surfaces of two tubes. This is provided to hold the two tubes together. A right angled coupler connects tubes at right angle. Swivel coupler is used for connecting two tubes at any angle other than a right angle. Putlog coupler is a non load bearing coupler used for fixing a putlog or transom to a ledger. Sleeve coupler is used to connect two tubes end to end.
47. Cradling	Cradling is light timber framing in formwork.
48. Cross Brace	Cross brace is a pair of diagonal braces.
49. Dead Shore	Dead shore is a support to hardening concrete designed to be left in place when the soffit form is struck.
50. Deal	Deal is a piece of square-sawn softwood timber of thickness varying between 50 to 100 mm and width varying between 230 to 280 mm.

51.	Deck or Decking	Deck or decking is the form upon which concrete for a slab is placed. This is the sheeting to a soffit form.
52.	Distance Piece (Spacer, Spreader)	Distance piece also referred to as spacer or spreader is a short piece of timber or other material. This is used to hold parallel forms for walls or beams at the correct spacing.
53.	Double-Headed Nail	Double-headed nail is a round-wire nail with a second head formed in the shank, just below the head struck by the hammer. The provision of the second head makes for easy withdrawal.
54.	Dovetailed Anchor (Anchor Slot or Slot Anchor)	Dovetailed anchor also referred as anchor slot or slot anchor is a device made from sheet steel or other metal. This is cast into the concrete surface to produce a chase which is narrower at the surface than at its base into which shaped metal tongues are inserted to form an anchor.
55.	Dowel	Dowel is a cylindrical piece of wood. This is used for positioning and fixing one member to another.
56.	Dowel Bar (Dowel Pin)	Dowel bar also referred as dowel pin is a short metal rod or bar cast into concrete with part of its length left projecting as a fixing or as a means of transferring forces acting in the plane of a joint from one member or slab to another.
57.	Edge Form	Edge form is the formwork to the edge of a road or other slab.
58.	Fair Face	Fair face is a plain concrete finish which is better than that produced from rough formwork.
59.	False work	False work is the temporary structure erected to support work in the process of construction. It is composed of shores, formwork for beams or slabs (or both), and lateral bracing. This is also defined as that part of formwork which supports the forms usually for a large structure, such as a bridge.
60.	Feather Edge	Feather edge is the sharp edge produced when two surfaces meet at an acute angle.
61.	Fillet	Fillet is a piece of timber of triangular cross-section. This is fixed at the intersection of two forms to produce a chamfer on the concrete.
62.	Fin	Fin is an undesirable projection from the face of the concrete caused by grout escaping into a gap along a joint in the form.
63.	Floor Centre (Telescopic Centre)	Floor centre, also known as telescopic centre, is a beam of adjustable length, usually a metal-lattice or sheet-metal box beam. This is used to support decking for a floor slab. Some manufacturers also term this as telescopic beam span.
64.	Folding Wedges (Easing Wedges)	Folding wedges also referred to as easing wedges are timber or metal wedges used in pairs. They are used (a) for leveling and adjusting formwork during erection and or (b) to release loads on formwork prior to striking, and (c) to tighten or slacken connections between formwork members.
65.	Form (Shutter)	Form or shutter is that part of formwork which consists of the sheeting and its immediate supporting or stiffening members. This is also defined as a temporary structure or mould for the support of concrete while it is settling and gaining sufficient strength to be self supporting.

66.	Form Anchor	Form anchor is a device normally embedded in the concrete during placement. This is used in the securing of formwork to previously placed concrete of adequate strength.
67.	Form Plucking (Form Scabbing)	Form plucking or scabbing is removal of the surface of the form due to adhesion of the form to the concrete.
68.	Form Tie (Wall Tie or Tie)	Form tie also referred to as wall tie or tie is a device for holding the opposing faces of wall, beam or other forms at the correct distance apart against the pressure of fresh concrete. This is also defined as a tensile unit adapted to holding concrete forms against the active pressure of freshly placed plastic concrete.
69.	Formwork (Shuttering)	Formwork or shuttering is defined as a complete system of temporary structure built to contain fresh concrete so as to form it to the required shape and dimensions, and to support it until it hardens sufficiently to become self supporting. Formwork includes the surface in contact with the concrete and all necessary supporting structure.
70.	Gang Mould	Gang mould is a series of moulds arranged in such a manner that a number of identical precast units can be formed on the same base.
71.	Ganged Form	Ganged form is a number of panels fixed together and stiffened with waling, strongbacks or soldiers, or combinations of these.
72.	Grillage	Grillage is an assemblage of timber or steel members placed parallel to each other under a sill to spread the load from the sill. For heavier loads a grillage may consist of two or more layers of parallel members placed across each other at right angles.
73.	Guard Rail or Handrail	Guard rail or handrail is a member incorporated in the structure at all points from where an operative may fall.
74.	Hanger	Hanger is a vertical member giving support from above.
75.	Hanging Formwork	Hanging formwork is a formwork which is supported from above.
76.	Hollow Form	Hollow form is a form for producing weight-reducing recesses in the slit of a suspended slab.
77.	Insulation (Lagging)	Insulation or lagging is a cover of insulating material intended to reduce loss of heat. It is used on forms in cold weather to reduce loss of heat from the concrete.
78.	Jack	Jack is a mechanical device for raising heavy loads.
79.	Jack Rod	Jack rod is a steel rod upon which jacks act in order to raise slipforms.
80.	Joint Pin	Joint pin is an internal fitting for jointing two tubes end to end.
81.	Jois	Joist is a horizontal or sloping timber beam, which carries decking for a suspended concrete.
82.	Lacing	Lacing is a horizontal member which holds together and positions props or other vertical supports.
83.	Lagging	Lagging is narrow timber fixed to a shaped frame for forming curved surfaces.
84.	Lap	Lap is the part of the length or height of a form which covers the previously placed concrete when the form is fixed in place for the next lift.

85.	Leapfrog Form	Leapfrog form is a form for a method of casting vertical surfaces in which two or more sets of forms are used so that the form for the last poured lift may be left fixed in place while that from a lower lift is struck and re-erected on top of it, the process being repeated, if necessary.
86.	Ledge (Cleat)	Ledge or cleat is a member nailed across a number of boards to hold them together.
87.	Ledger (Runner)	Ledger or runner is a horizontal timber supported on posts or hangers and, carrying joists. Other definitions of ledger are: (a) A timber fixed to the side of a beam box, or elsewhere to support the ends of joists carrying decking. (b) A longitudinal member spanning across a number of support members to lace them together.
88.	Lift	Lift is that height of concrete which is poured in one continuous operation.
89.	Loose Tongue	Loose tongue is a strip of wood which is inserted into grooves in the edges of adjacent boards where these are to be joined edge to edge.
90.	Mould	Mould is a form for casting precast concrete units.
91.	Mould Oil	Mould oil is oil or emulsion applied to the face of forms. This acts primarily as a release agent.
92.	Nogging Piece	Nogging piece is a short wooden strut fixed between and at right angles to cleats, joists, studs, walings, etc. in order to stiffen them.
93.	Pallet	Pallet is a flat timber or metal plate on which precast concrete units are cast and handled until they have hardened.
94.	Pan	Pan is a form of stiffened steel sheet which is a component of a system of formwork. A number of pans may be fixed together to construct a larger area.
95.	Panel	Panel is a prefabricated form of limited size, designed for repeated re-use, a number of which may be fixed together to form a larger surface.
96.	Permanent Form (Permanent Shutter)	Permanent form or shutter is a form permanently left in place to provide a facing to the concrete.
97.	Placing Rate	Placing rate is the rate at which the free surface of the concrete rises in the forms during placing.
98.	Pocket	A recess formed in a concrete surface is known as pocket.
99.	Post	A post is a vertical support.
100.	Profiled Form (Profiled Shutter)	Profiled form or shutter is a form for casting a concrete surface which is curved or unusual in shape.
101.	Prop	Prop is a strut which is light enough to be man-handled. A prop can be adjustable in length.
102.	Raking Strut (Raker)	Raking strut or raker is an inclined strut.
103.	Random Board Forms	Random board form is a form, the sheeting of which is made with softwood boards of random widths or lengths or both.
104.	Release Agent	Release agent is a substance, usually applied to the form face, to prevent adhesion of the concrete to the form and thus facilitate stripping.

105.	Re-Propping (Re-Shoring)	Re-propping or re-shoring is the application of posts or props to the soffit of a concrete slab or beam to enable it to carry an excess superimposed load, or to carry superimposed loads when the concrete is not fully mature.
106.	Ribbon	Ribbon is a runner connecting beam bottom support members to prevent spreading of the lower edges of the side of a beam box.
107.	Road Form	Road form is an edge form used in the construction of a road or other ground slab. The term is normally used to describe specially designed proprietary steel products.
108.	Rough Formwork	Rough formwork is formwork for concrete where no special surface finish is required.
109.	Sawn Timber	Timber which has not been planed on any surface
110.	Scaffold (Scaffolding)	Scaffold or scaffolding is a temporary structure for gaining access to higher levels of the permanent structure during construction.
111.	Scaffold Board	Scaffold board is a softwood board used with similar boards to form walkways and as toe boards on a scaffold.
112.	Sheeting (Sheathing)	Sheeting or sheathing is that part of the form which is in contact with the concrete.
113.	Side Form	Side form is a form for the side of a concrete member where the height of the member is not great in proportion to the width of its cross-section.
114.	Sill (Sole Plate)	Sill or sole plate is a horizontal timber under the foot of a jack or post used to spread the load from the member above.
115.	Slip Form (Moving Formwork, or Sliding Formwork)	Slipform, also referred to as moving or sliding formwork, is a form which moves, usually continuously, during placing of the concrete. The movement may be either horizontal or vertical.
116.	Snap Tie	Snap tie is a form tie which is designed to be broken off beneath the concrete surface after use.
117.	Soffit	Soffit is the under surface of concrete, for example, of a concrete arch, or suspended beam or slab.
118.	Soffit Form	Soffit form is the form to a soffit.
119.	Soldier	Soldier is a vertical member, acting as a beam or cantilever, and used in conjunction with form ties or struts to support and prevent movement of forms.
120.	Square-Edged Boards	Square-edged boards are boards of rectangular cross-section with plain edges suitable for butt jointing.
121.	Staging	Staging is a temporary structure on which persons work. Staging is formed from prefabricated frames which are sometimes mounted on casters.
122.	Standard or Upright	Standard or upright is a tube used as a vertical support or column in the construction of a scaffold and transmitting a load to the ground or a base plate.
123.	Stop End	Stop end is the form for a construction joint in the vertical plane.
124.	Striking Piece (Stripping Piece)	Striking or stripping piece is a narrow, often splayed, member intended to facilitate striking in a confined space.
125.	Striking Time (Stripping Time)	Striking or stripping time is the time specified for the earliest removal of forms and other support from the concrete.

126.	Strongback (Counter-Waling)	Strongback or counter-waling is a soldier, often of framed construction, for heavy duty application.
127.	Strut	A strut is a member in compression.
128.	Stud	A stud is a vertical or horizontal stiffener to the back of the form sheeting.
129.	Table Form	Table form is a formwork for suspended floors which are built up into the form of a table. It stands on the floor previously cast and is lowered and moved in one piece.
130.	Telltale	A telltale is any device designed to indicate movement of formwork.
131.	Templet Profile or Template	Templet profile, also known as template, is a guide or pattern used for accurately locating bolts, etc. These are made of timber or metal and used for building or testing the accuracy of surfaces which are curved or otherwise unusual in shape.
132.	Through Tie	Through tie is a form tie passing through the concrete. The tie is withdrawn when the forms are struck.
133.	Tie Tube	Tie tube is a tube used for connecting a scaffold to a reveal tie or other rigid anchorage.
134.	Tilting Table	Tilting table is a table on which a vertical concrete component is cast horizontally, and which is tilted into a vertical or near vertical position to allow the component to be lifted in its natural position.
135.	Tilt-Up Form	Tilt-up form is a wall form which is built on the ground beside the wall to be cast and then rotated into the vertical plane.
136.	Toe Board Clip	Toe board clip is a clip for attaching toe boards to scaffolding members.
137.	Top Form	Top form is a form to the top surface of the concrete. This is required where the slope of concrete to be cast is very steep and cannot be cast without containment.
138.	Transom	Transom is a tube spanning across ledgers to tie a scaffold transversely and which may also support a working platform.
139.	Travelling Formwork (Mobile Form or Moving Formwork)	Travelling, mobile, or moving formwork is a formwork carried on wheels or rollers so that it may be struck and moved for re-use without dismantling. This is used for walls and tunnel linings.
140.	Trestle	Trestle is a structure for temporary or permanent support which gains its stability by having its main members at an inclination to the vertical.
141.	Tripod	Tripod is a self supporting metal stand with three legs for supporting one end of a horizontal beam on which a working platform may be laid.
142.	Trough Form	Trough form is a form which produces an inverted-trough shaped concrete soffit.
143.	Veneer	Veneer is a thin sheet of wood. This is produced usually by rotary cutting or slicing.
144.	Void Box	Void box is a trapped form. This is used to create a completely enclosed space within the concrete.

145.	Waling	Waling is a long horizontal member acting as a beam and used in conjunction with form ties, struts or strong backs to support and prevent movement of forms. These are also referred to as wales or walers.
146.	Wall Clamp (Wall Cramp or Clamp or Cramp) -	Wall clamp also known as wall cramp, clamp, or cramp is like an adjustable yoke to support wall forms in such a way as to avoid the use of form ties through the concrete.
147.	Wedge	Wedge is a piece of wood or metal tapering to a thin edge. This is used to adjust elevation and tightening of formwork.
148.	Wrecking Strip	Wrecking strip is a striking piece which is intended to be destroyed in the course of formwork striking.
149.	Yoke	Yoke is an assembly of members which restrains forms from movement by encircling, or nearly encircling, them. In case of a column form, yoke acts as a tie or clamping device around column forms to keep them from spreading because of the lateral pressure of concrete. In the case of a vertical slipform, a yoke is in the form of an inverted 'U'. A yoke in the case of slipform carries the wall forms. Yokes themselves are carried and raised by a jack in case of slipform.

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3. IS 399 : 1963, Reaffirmed 2005, *Classification of Commercial Timbers and their Zonal Distribution*.
4. IS 456 : 2000 *Plain and Reinforced Concrete - Code of Practice*.
5. IS 800 : 2007 *General Construction in Steel - Code of Practice*.
6. IS:806:1968, Reaffirmed 2002, *Code of Practice for Use of Steel Tubes in General Building Construction.*
7. IS 816 : 1969, Reaffirmed 1998, *Code of Practice for use of Metal Arc Welding for General Construction in Mild Steel*.
8. IS 875 : Part 1 : 1987, Reaffirmed 2003, *Code of Practice for Design Loads (Other than Earthquake) for Buildings and Structures Part 1: Dead Loads - Unit Weights of Building Material and Stored Materials* (Incorporating IS:1911-1967).
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12. IS:1161, (1998), Reaffirmed 2003, *Steel Tubes for Structural Purposes— Specification.*
13. IS 1597 : Part 1 : 1992, Reaffirmed 2002, *Construction of Stone Masonry — Code of Practice - Part 1 : Rubble Stone Masonry*.
14. IS 1734 : Parts 1 to 20 : 1983, Reaffirmed 2003, *Methods of Test for Plywood*.
15. IS 1786 : 2008 *High Strength Deformed Steel Bars and Wires for Concrete Reinforcement – Specification*.
16. IS 1905 : 1987, Reaffirmed 2002, *Code of Practice for Structural use of Unreinforced Masonry*.
17. IS 1977 : 1996, Reaffirmed 2001, *Low Tensile Structural Steels – Specification*.
18. IS 2062 : 2006, *Hot Rolled Low, Medium and High Tensile Structural Steel*.
19. IS 2212 : 1991, Reaffirmed 2005, *Code of Practice for Brickwork*.
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28. IS:4891, (1988), Reaffirmed 2005, *Specification for Preferred Cut Sizes of Structural Timber.*

29. IS:4990, (1993), Reaffirmed 2003, *Plywood for Concrete Shuttering Work— Specification.*

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31. IS 8112 : 1989, Reaffirmed 2005, *Specification for 43 Grade Ordinary Portland Cement*.

32. IS 8500 : 1991, Reaffirmed 2000, *Structural Steel - Microalloyed (Medium and High Strength Qualities) – Specification*.

33. IS 12269 : 1987, Reaffirmed 2004, *Specification for 53 Grade Ordinary Portland Cement*.

34. IS:14687, (1999), Reaffirmed 2005, *Falsework for Concrete Structures— Guidelines.*

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Index

M

P

S. no	Activity Description	Time in days (01–32)
1.	Cutting, bending and shifting of reinforcement for 20 column sets	
2.	Formwork for 20 sets of column unit 1, braces and shore	
3.	Tying of verticals and rings in column	
4.	Fixing of column shutter	
5.	Inspection and casting of column up to beam bottom	
6.	De-shuttering of column	
7.	Scaffolding for beam bottom	
8.	Placing of beam bottom form work	
9.	Inspection of beam bottom level, alignment, and tying of beam reinforcement	
10.	Inspection of beam reinforcement and fixing of beam inner side form work	
11.	Scaffolding for slab bottom	
12.	Fixing of formwork for slab bottom	
13.	Inspection of slab bottom and tying of slab reinforcement	
14.	Closing of outer side of beam	
15.	Concreting of slab	
16.	Tying of verticals and rings in column in unit 2	
17.	Fixing of column shutter	
18.	Inspection and casting of column up to beam bottom	
19.	De-shuttering of column	
20.	Scaffolding for beam bottom	
21.	Placing of beam bottom form work	
22.	Inspection of beam bottom level, alignment, and tying of beam reinforcement	
23.	Inspection of beam reinforcement and fixing of beam inner side form work	
24.	Scaffolding for slab bottom	
25.	Fixing of formwork for slab bottom	
26.	Inspection of slab bottom and tying of slab reinforcement	
27.	Closing of outer side of beam	
28.	Concreting of slab unit 2	
29.	De-shuttering of slab and beam of unit 1	
30.	Tying of verticals and rings in column in unit 3	
31.	Fixing of column shutter	
32.	Inspection and casting of column up to beam bottom	
33.	De-shuttering of column	
34.	Scaffolding for beam bottom	
35.	Placing of beam bottom form work	
36.	Inspection of beam bottom level, alignment, and tying of beam reinforcement	
37.	Inspection of beam reinforcement and fixing of beam inner side form work	
38.	Scaffolding for slab bottom	
39.	Fixing of formwork for slab bottom	
40.	Inspection of slab bottom and tying of slab reinforcement	
41.	Closing of outer side of beam	
42.	Concreting of slab unit 3	
43.	De-shuttering of slab and beam of unit 2	
44.	Tying of verticals and rings in column in unit 4	
45.	Fixing of column shutter	
46.	Inspection and casting of column up to beam bottom	
47.	De-shuttering of column	
48.	Scaffolding for beam bottom	
49.	Placing of beam bottom form work	
50.	Inspection of beam bottom level, alignment, and tying of beam reinforcement	
51.	Inspection of beam reinforcement and fixing of beam inner side form work	
52.	Scaffolding for slab bottom	
53.	Fixing of formwork for slab bottom	
54.	Inspection of slab bottom and tying of slab reinforcement	
55.	Closing of outer side of beam	
56.	Concreting of slab unit 4	

Figure 16.7 Detailed Schedule for the Example Project.

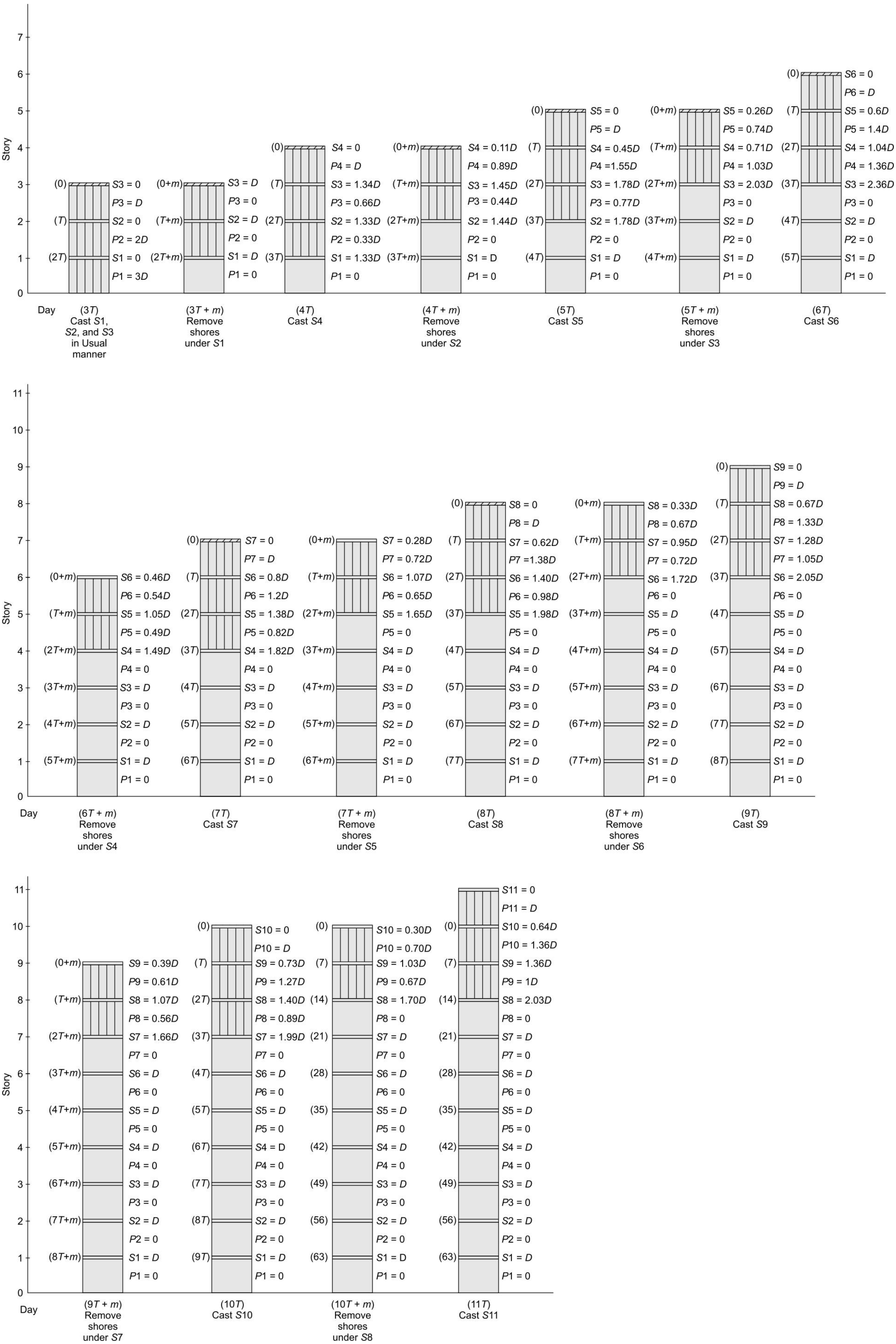

Figure 18.22 Load Ratios Versus Time for Three Levels of Shores.

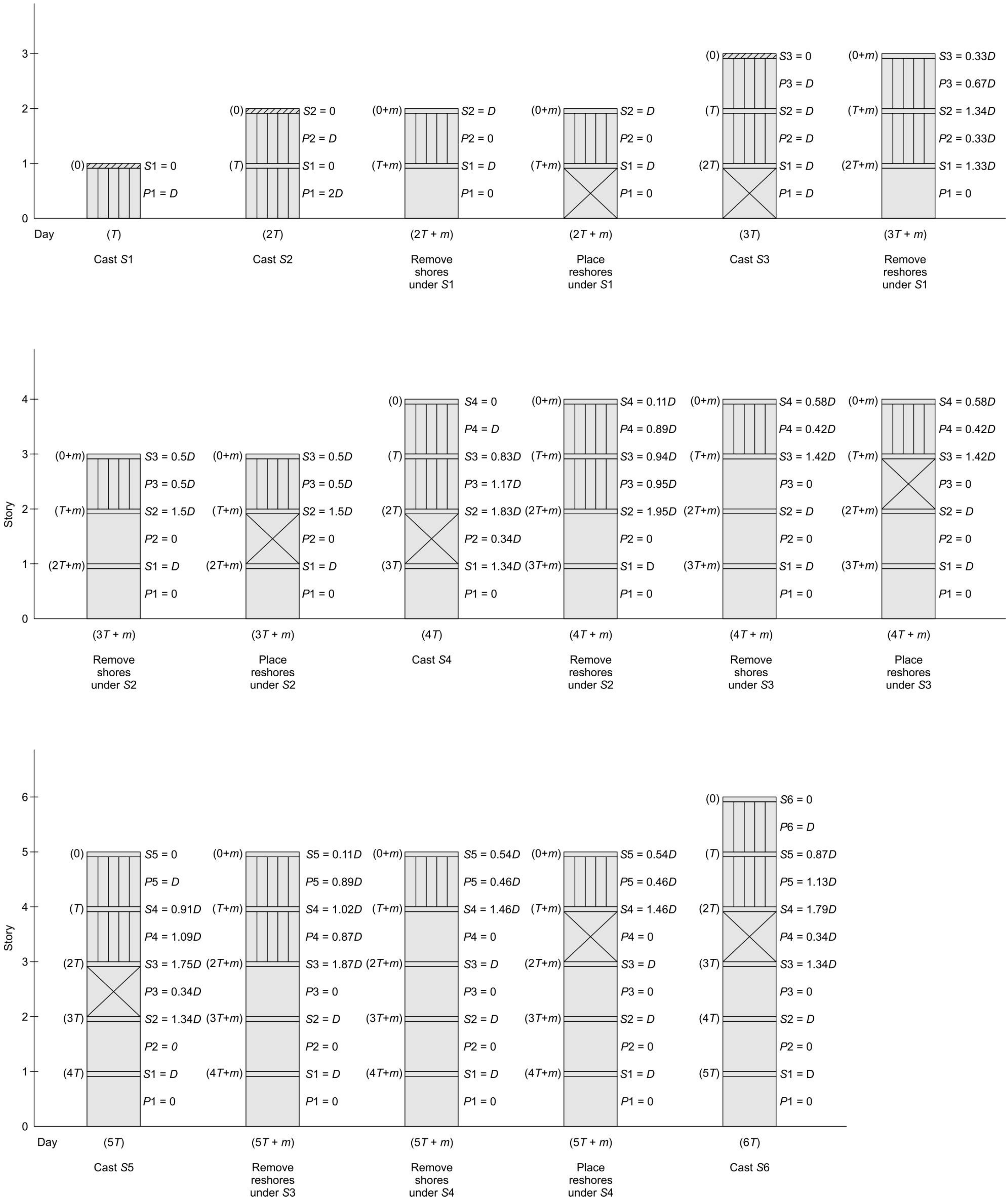

Figure 18.35 Load Ratios Versus Time for Two Levels of Shores and One Level of Reshores (Only Dead Weight of Slab Considered).

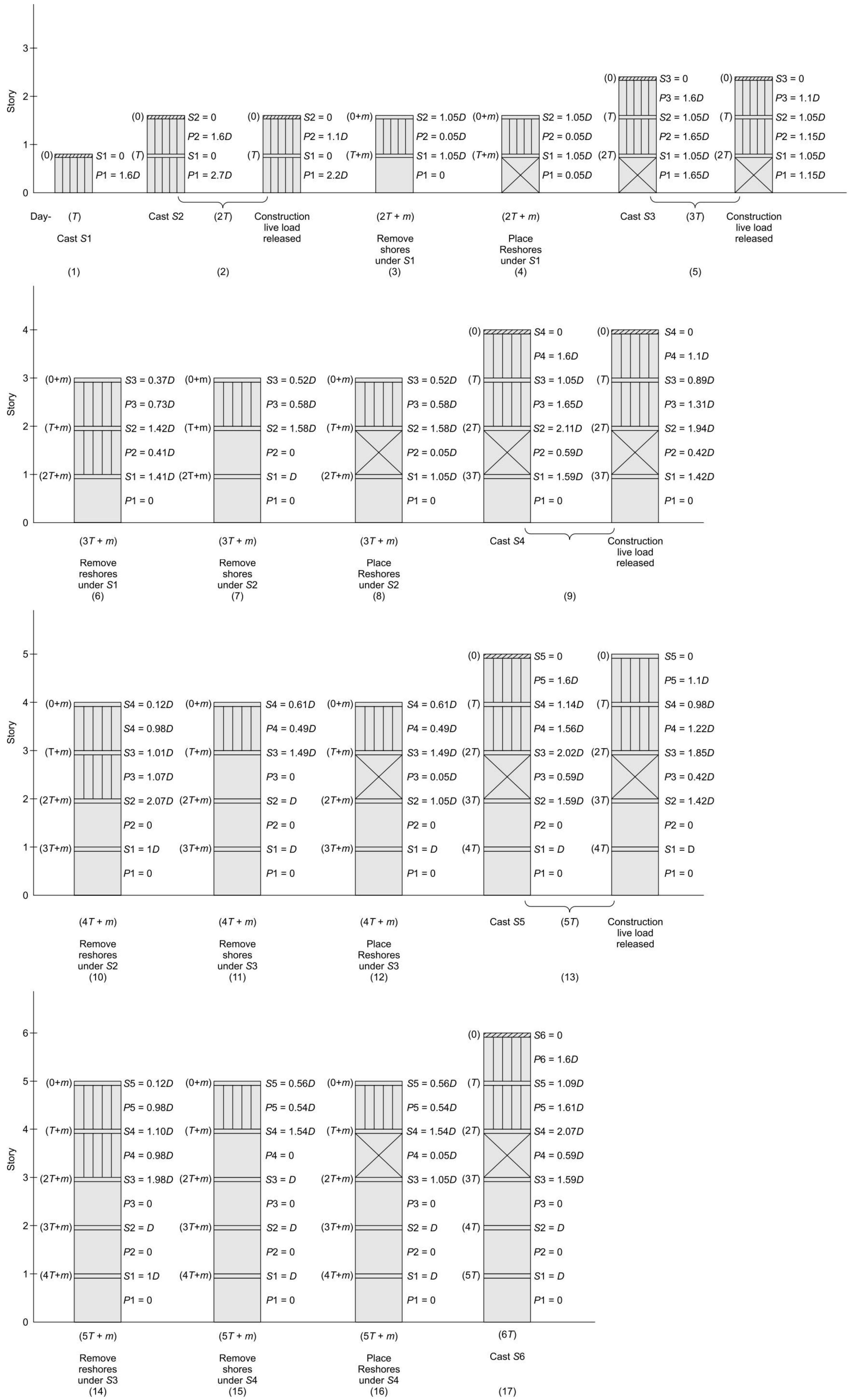

Figure 18.47 Load Ratios Versus Time For Two Levels of Shores and One Level of Reshores (Dead Weight of Slab, Self Weight of Shores and Reshores, and Construction Live Loads Considered).

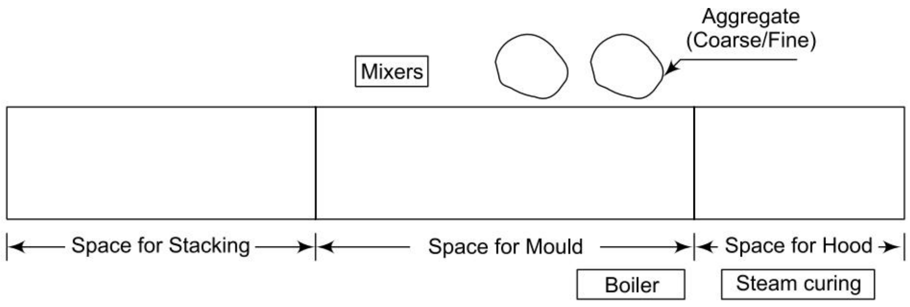

Figure 14.5 Typical Layout of a Precast Yard.

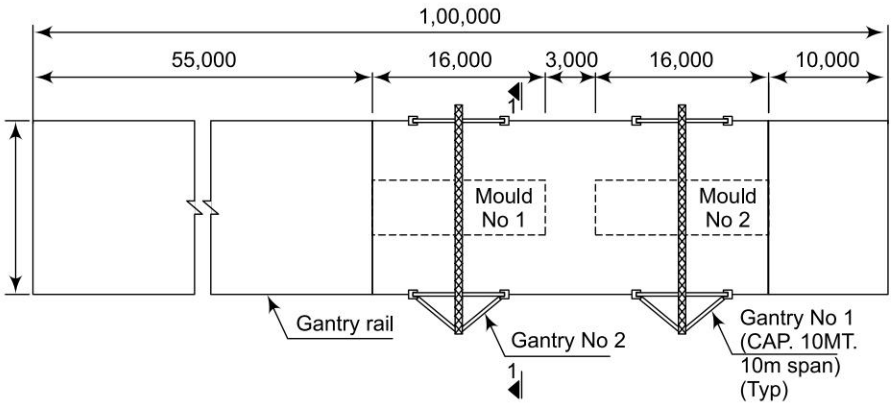

Figure 14.6 Details of Space for Mould Shown in Fig. 14.5.

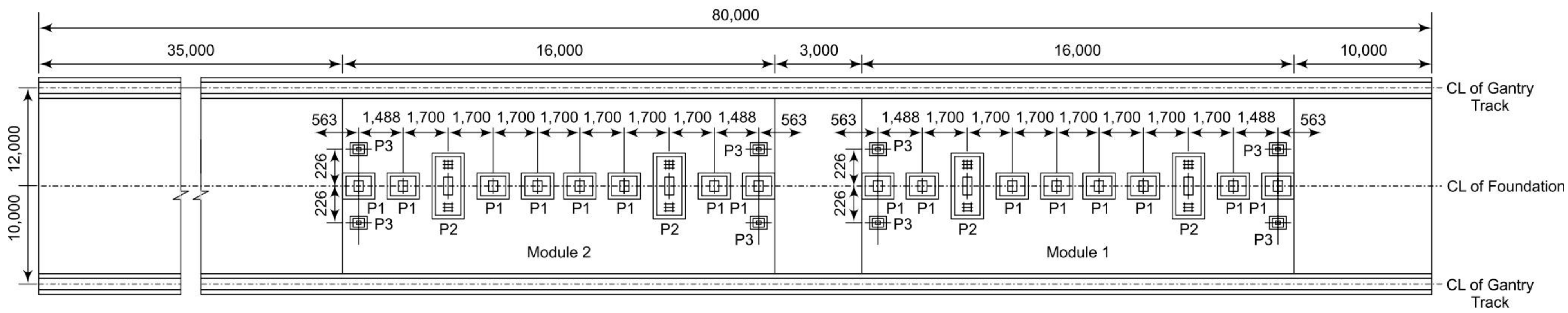

Figure 14.7 Further Blow Up of Space for Mould Shown in Fig. 14.6.

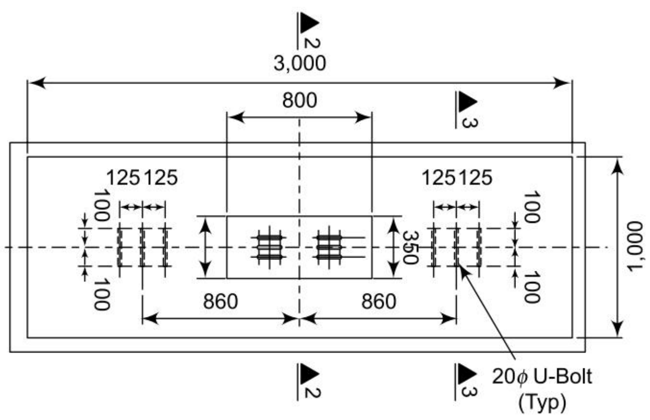

Figure 14.8 Plan of Footing P-2.

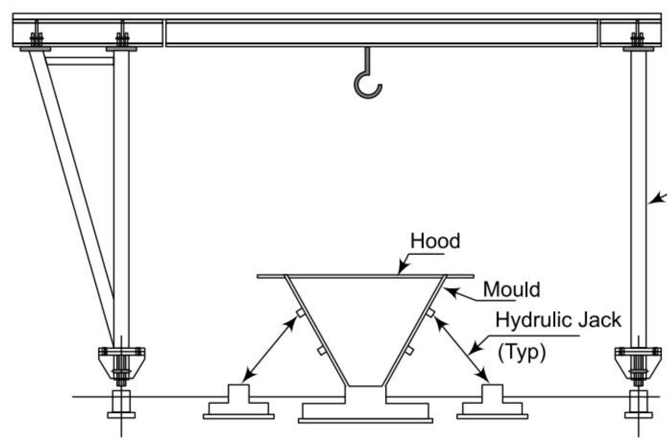

Figure 14.9 Section 1-1 of Figure 14.6.

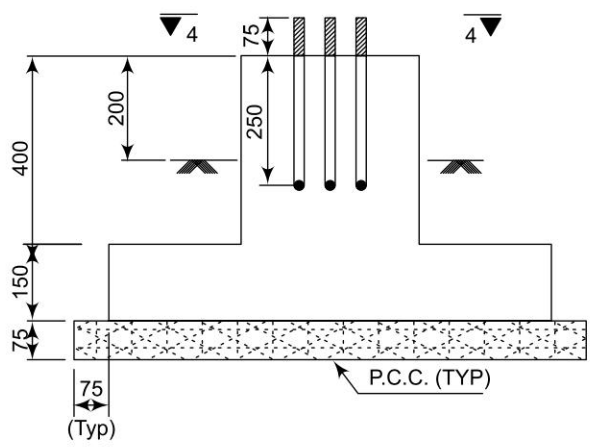

Figure 14.10 Section 2-2 of Figure 14.8.

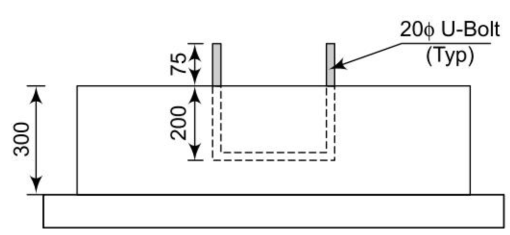

Figure 14.11 Section 3-3 of Figure 14.8.

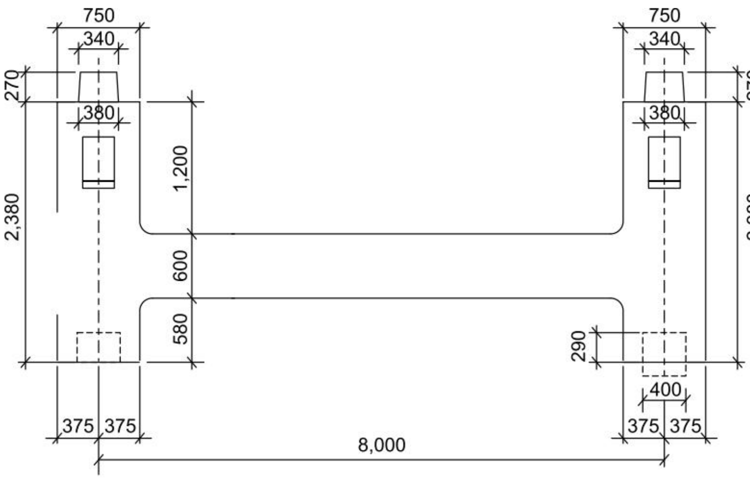

Figure 14.16 Elevation of PB-1.

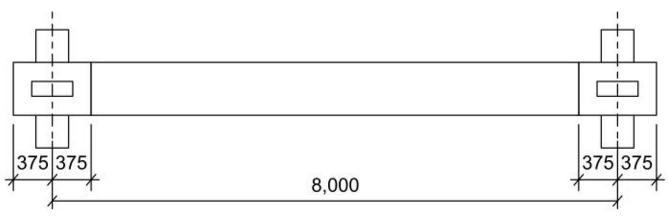

Figure 14.17 Plan of PB-1.

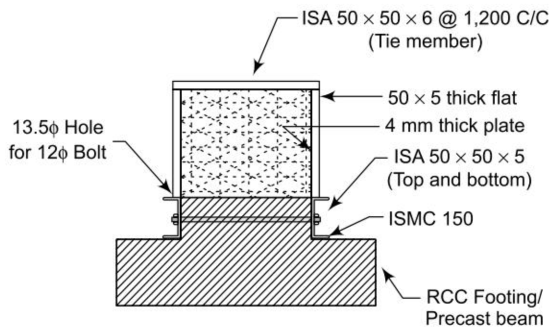

Figure 14.18 Typical Detail of a Shutter.

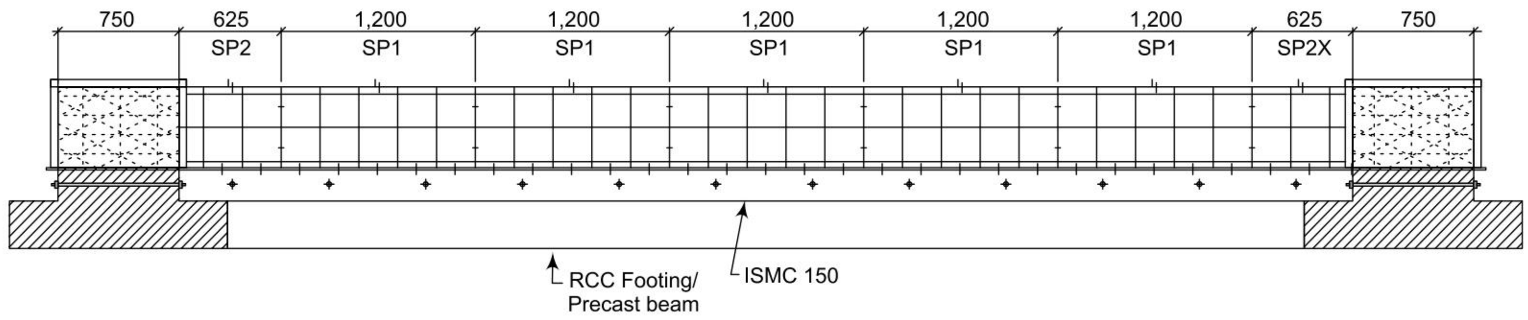

Figure 14.19 Arrangement for Precast Beam.

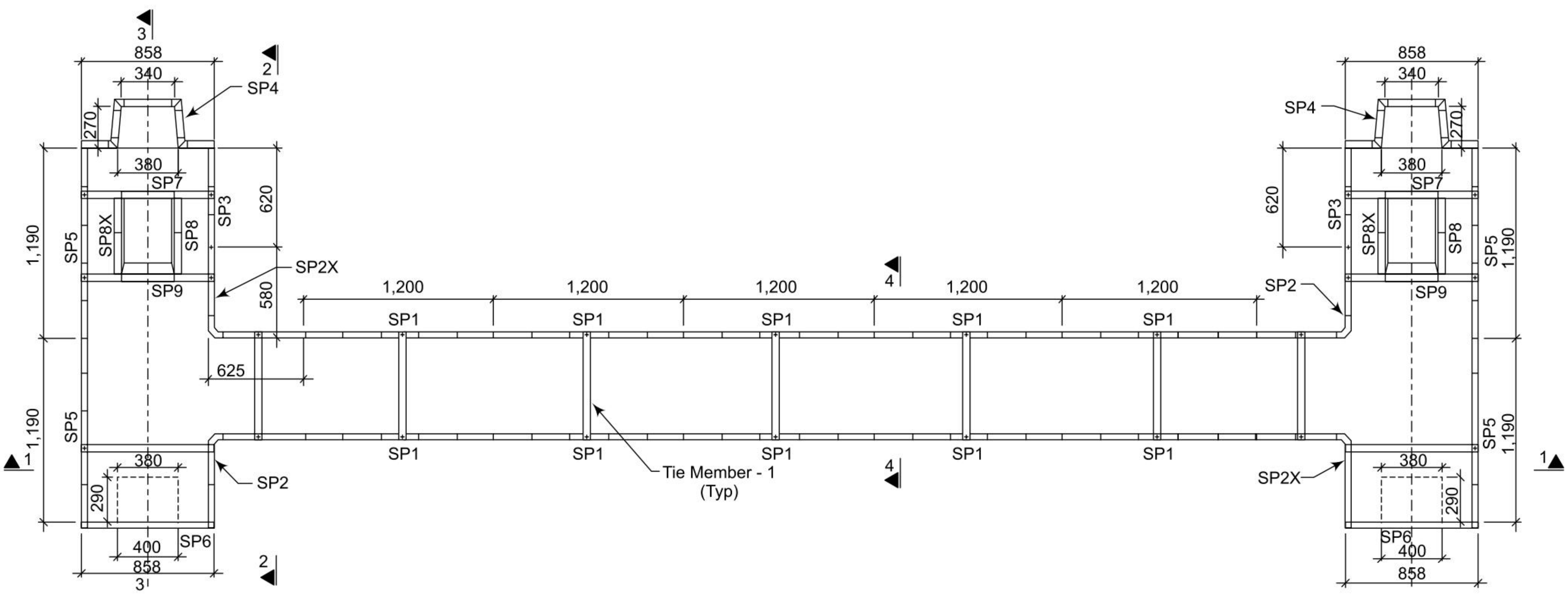

Figure 14.20 Layout Showing Shutter Panel Marking for PB-1.

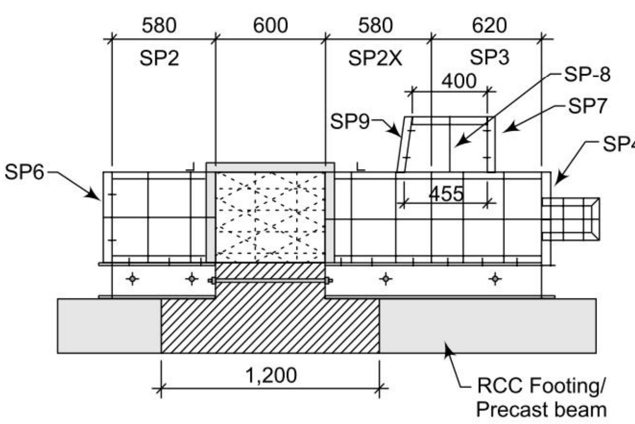

Figure 14.21 Section 2-2 of Figure 14.20.

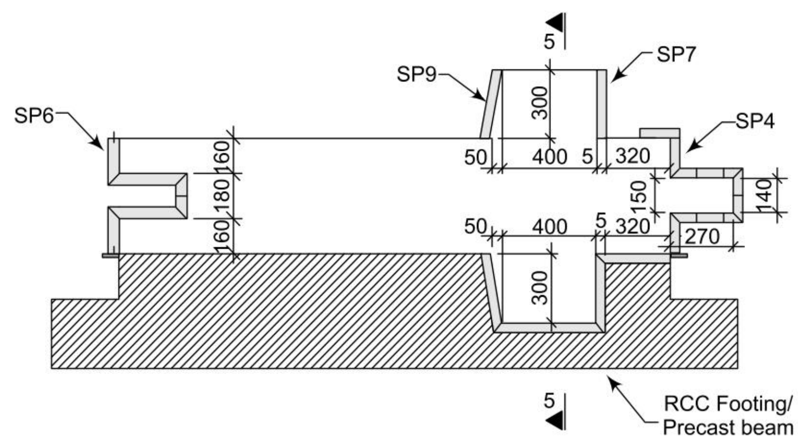

Figure 14.22 Section 3-3 of Figure 14.20.

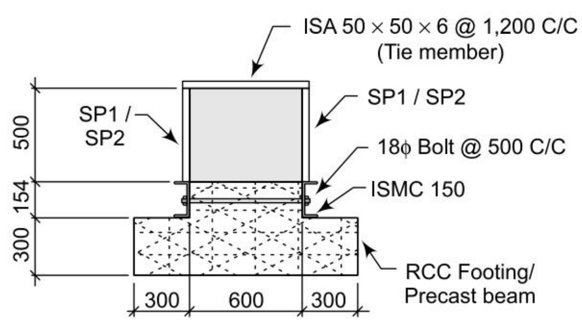

Figure 14.23 Section 4-4 of Figure 14.20.

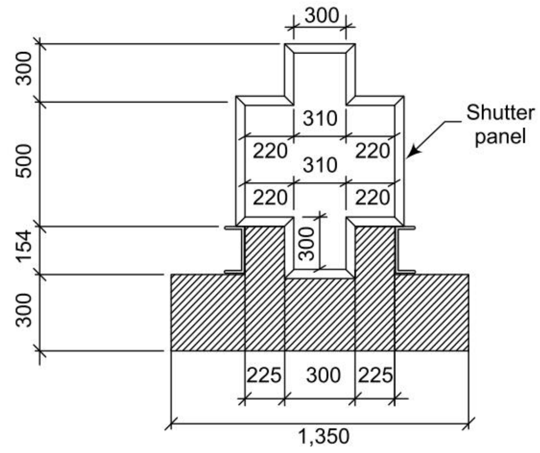

Figure 14.24 Section 5-5 of Figure 14.22

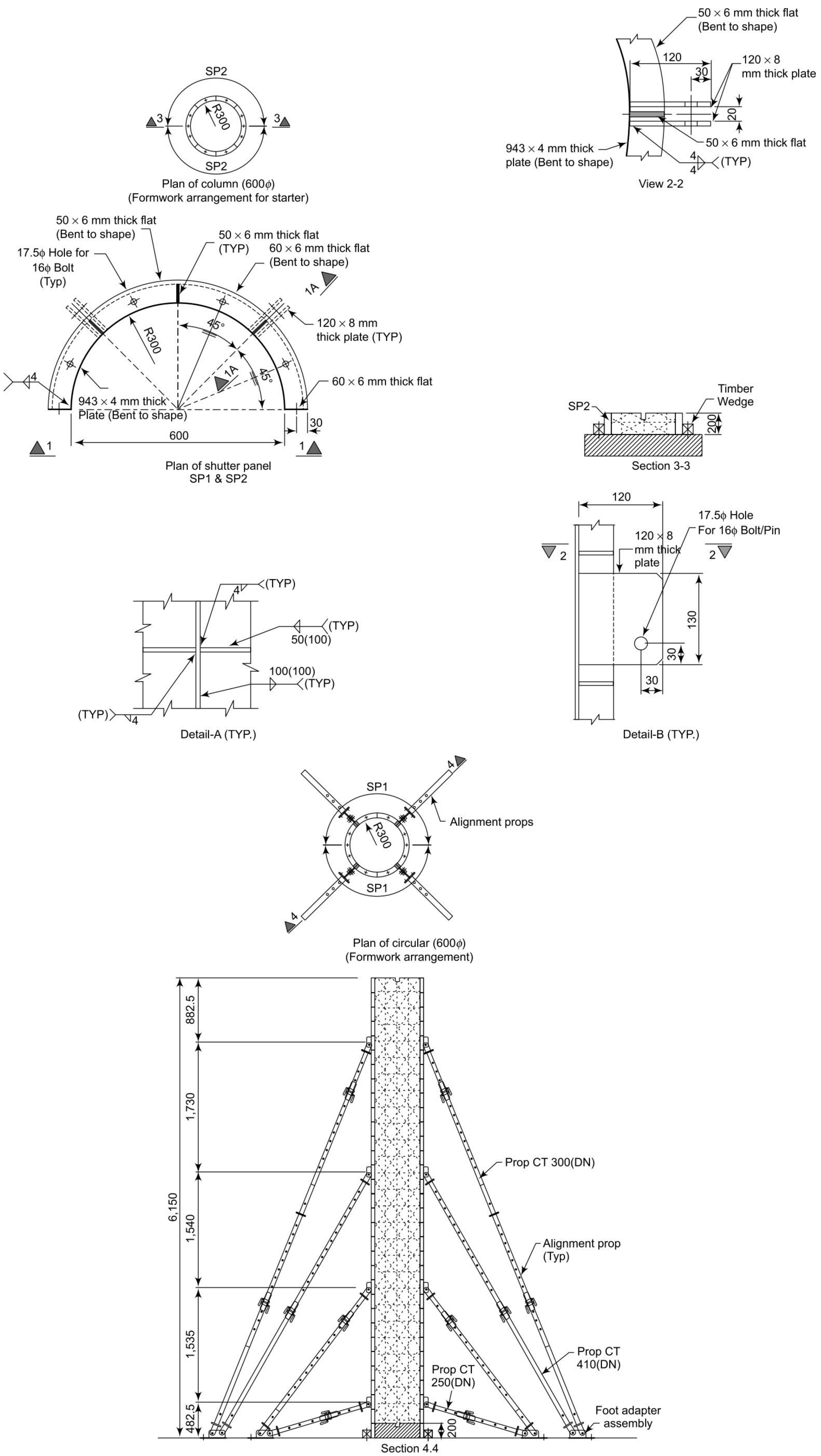

Figure 6.27 Formwork Arrangement for a Circular Column.

Author's Profile

Dr. Kumar Neeraj Jha is with the Department of Civil Engineering, IIT Delhi. He started his career with Larsen and Toubro Ltd as Graduate Engineer Trainee in 1993 and held a number of positions in the same company till 2005. He was instrumental in successful completion of a number of construction projects of national importance. He then joined the Department of Civil Engineering-IIT Kanpur for a brief period. Thereafter he has been with the Indian Institute of Technology Delhi. He has been involved with a number of construction projects as consultant and specialises in project management and formwork. He has published in a number of international and national journals and conference proceedings. His book on Construction Project Management published by Pearson Education is widely accepted as a text book in different universities. He teaches various courses in construction technology and management. He has conducted a number of training programs for industry and has also been involved with a number of consultancy projects. He is a reviewer of all the leading journals in construction management and also on the editorial board of two International Journals.