Analysis of Construction Loads on Concrete Formwork

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ABSTRACT

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This study presents analytical procedures for determining the loads on the shoring system and supporting slabs during the construction of multistory concrete buildings and for determining the lateral pressures imposed by fresh concrete against the wall forms. The procedures assume two-dimensional frame models, which employ both the analysis computer program (CPF) and the maturity-based model. The interaction of structure and compressible shoring, time-dependent concrete properties of strength, creep and shrinkage of concrete and the change in construction load during construction cycles are considered in the shoring system analysis. For wall formwork analysis, the time-dependent concrete properties of strength and the properties of wall-element parts are considered.

The analytical results obtained by both the shoring system and the wall formwork analyses are checked against the field measurements and several existing methods. Various parameters that affect the construction load distribution among shores and reshores and interconnected slabs, and the fresh concrete pressure distribution on wall formwork are investigated. The present method can be used to design and construct safe and economical concrete structures, especially during concrete placing and formwork removal operations.

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I wish to acknowledge the constant encouragement and moral support given throughout my education by my family, especially, my mother who passed away while I was doing this research. Finally, greatest thanks go to my God, by whose blessings I was able to persevere through out the completion of this thesis.

To my son, Aboukhzam

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A_t	cross-section area of tie
b	width of wall strip
C	coefficient dependent on fresh concrete properties
C_1	coefficient (1 or 1.5) as specified below Eq. (2.9)
C_2	coefficient (0.3, 0.45, or 0.6) as specified below Eq. (2.9)
C_t	construction slab load
D	dead load
d	minimum form dimension or wall thickness
d'	equivalent form thickness
E_c	modulus of elasticity of concrete
E_t	modulus of elasticity of tie
E_f	modulus of elasticity of real form
E'_f	modulus of elasticity of modeled form
E_w	modulus of elasticity of wood
F	shore load
f'_c	28-day cylinder strength of concrete
f'_w	compression strength of wood
H	vertical form height
h	maximum height of fresh concrete in the form
h_i	immersed depth of vibrator
h_{max}	head at which maximum pressure occurs

h'_{max} head at which maximum initial pressure occurs

HP horsepower of vibrator

I_f moment of inertia of real form

I'_f moment of inertia of modeled form

K temperature coefficient as specified below Eq. (2.9)

k_t axial stiffness of tie

L live load

L_t length of tie

M slab load (moment)

m maturity defined as the integral over time of temperature

P_{max} maximum lateral pressure

P' initial lateral pressure at any time

P'_{max} maximum initial lateral pressure

T temperature of concrete

T_0 datum temperature

R rate of concrete placing

V slab load (shear force)

w intensity of distributed load

γ density of concrete

β ratio of early age compressive strength to design strength

CHAPTER I

INTRODUCTION

1.1 General

In construction industry, the freshly cast-in-place concrete elements (e.g. slabs, walls, etc) are usually supported temporarily by a system of formwork until the imposed loads can be carried by the concrete structure itself. Formwork is an expensive component in most concrete structures. Its cost generally ranges from 30% to 60% of the cost of the concrete structure. Savings depend on the ingenuity and experience of all those involved in the design and construction of concrete structures. Good judgement in the selection of materials and equipment, in planning fabrication and erection procedures, and in scheduling reuse of forms, can expedite the job and cut costs.

Nowadays, using technological improvements in construction methods, materials, and equipment plays key roles in productivity. As a result, casting one or two floors per week in multistory buildings has been very common, especially in metropolitan areas (Stivaros and Halvorsen, 1990) and (Gross and Lew, 1986). For walls, the use of concrete pump increases the placement rate of concrete to about 200 cubic yards per hour. While it is desirable to have a rapid construction progress, it is essential that the quality and safety of the structure be maintained.

Construction loads, imposed by the shoring system, may be greater than the service loads for which the slabs were designed, even if they were at full design strength. Removal of shores/reshores may also be conducted before the slab/slabs attained their required strength. Exceeding the capacity of some floors may lead to a partial or total failure of the system, which in turn causes loss of property, injuries and/or loss of lives. Therefore, rigorous analysis, based on the performance and behavior of the early age concrete, must be performed during the design and construction stages of concrete structures.

Overestimating or underestimating the maximum lateral pressures of fresh concrete against the vertical forms may not help to achieve the main objectives in designing formwork, namely: quality, safety, and economy. Vertical formwork failures are a consequence of poor design utilizing undersize members and excessive placement rate. Failures of vertical formwork generally result in property damage requiring cleanup and reconstruction with a low incidence of bodily harm and very few fatalities. The degree of accuracy of determining the magnitude and the nature of loads and pressures, by studying all factors that may affect the forms, is the only way to achieve safe and economical vertical formwork.

1.2 Causes of Formwork Failures

Failure of formwork in general results from various causes: excessive loads, premature removal of forms or shores, inadequacies related to formwork, and human error on the job, whether due to indifference, haste, or lack of knowledge. Hadipriono and Wang (1986) studied the causes of 85 major falsework failures that have been documented over the past 23 years. Three causes of failure were identified: enabling, triggering, and procedural causes (Table 1.1). The enabling causes are defined as events that contribute to the deficiencies in the design and construction of falsework. The triggering causes are

Table 1.1 Causes of Falsework Failure (Hadipriono and Wang, 1986)

Number of occurrence (1)	Causes of failure (3)
	(a) Triggering Cause of Failure
3	Heavy rain causing falsework foundation slippage
1	Strong river current causing falsework foundation slippage
1	Strong wind
4	Fire
5	Failure of equipment for moving formwork
4	Effects of formwork component failure
1	Concentrated load due to improper prestressing operation
2	Concentrated load due to construction material
2	Other imposed loads
27	Impact loads from concrete debris and other effects during concreting
3	Impact load from construction equipment/vehicles
5	Vibration from nearby equipment/vehicles or excavation work
6	Effect of improper/premature falsework removal
20	Other causes or not available
	(b) Enabling Causes of Failure
17	Inadequate falsework cross-bracing/lacing
14	Inadequate falsework component
9	Inadequate falsework connection
7	Inadequate falsework foundation
8	Inadequate falsework design
4	Insufficient number of shoring
1	Inadequate reshoring
4	Failure of movable falsework/formwork components
2	Improper installation/maintenance of construction equipment
1	Failure of permanent structure component
4	Inadequate soil foundation
2	Inadequate design/construction of permanent structure
30	Other causes or not available
	(c) Procedural Causes of Failure
23	Inadequate review of falsework design/construction
22	Lack of inspection of falsework/formwork during concreting
2	Improper concrete test prior to removing falsework/formwork
4	Employment of inexperienced/inadequately trained workmen
1	Inadequate communication between parties involved
5	Change of falsework design concept during construction
38	Other causes or not available

usually external events that could initiate falsework collapse. The procedural causes are frequently hidden events that produce the enabling and, many times, the triggering events. Most failures occurred as result of the interaction between triggering and enabling events that were, in many cases, produced by inadequacies in procedural methods.

According to the type of collapse, almost half of the building falsework failures were due to deficiencies in vertical shores (Fig. 1.1). Figure 1.2 shows that concrete placing, and falsework and formwork removals are two critical stages of operation. One out of two falsework collapses occurred during concrete casting. In many cases, the excessive rate of pouring and the direct vibration to form ties led to vertical formwork failures. As a result of vertical formwork bursting in multistory buildings, the additional loads imposed by the fallen debris triggered a collapse in the level below. In other cases, the accident caused by impact of powered equipment, such as motorized buggies, imposed additional loads on falsework and led to falsework collapses. Falsework or formwork removal is the other critical operation stage, about 12% of falsework failures occurred as a result of improper reshoring, and premature falsework or formwork removal.

1.3 Objectives

This study has two main objectives: 1) to study the behavior of formwork during the construction of concrete structures, with a focus on the interaction of placed concrete and the structural elements of formwork; and 2) to develop practical computer-based models to determine the load distribution among the shores and interconnected slabs during construction of multistory reinforced concrete buildings, and to determine the lateral

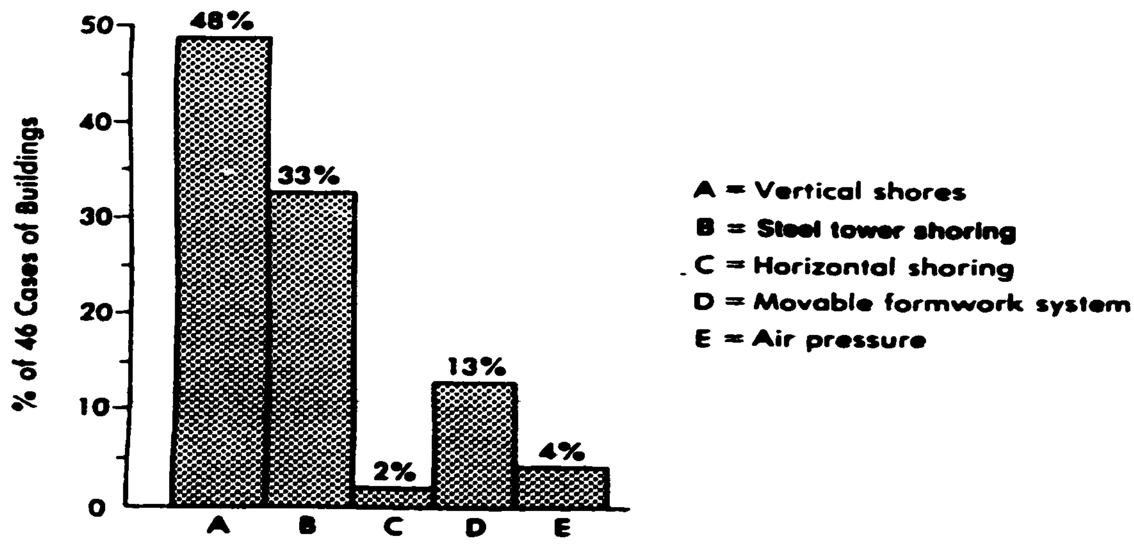


Fig. 1.1 Falsework Collapse by Type of Falsework (Hadipriono and Wang, 1986)

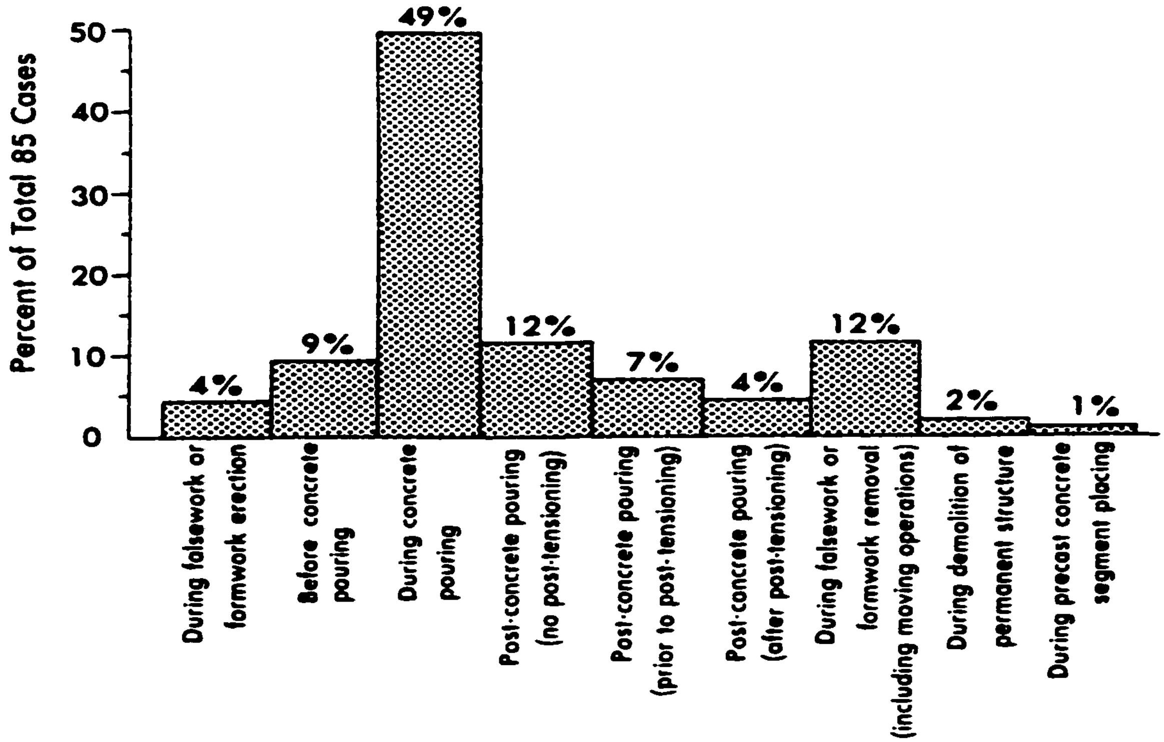


Fig. 1.2 Falsework Collapse by Construction Stage (Hadipriono and Wang, 1986)

pressures exerted by fresh concrete on wall forms. Specifically, it intends to carry out, through review of literature, a relation to design the formwork, with a focus on design loads imposed by the placed concrete and construction operations, and develop computer models for the analysis of the loads imposed by placed concrete and construction operations on those temporary structures, accounting for current construction practice. The overall objectives are as the follows:

- To develop computer analysis models on the basis of construction practice on job sites,
- To determine the distribution of construction loads among the shores and interconnected slabs,
- To determine the maximum lateral pressures of fresh concrete on wall formwork,
- To conduct a parametric study to determine the influence of different factors on load distribution during construction among shoring system, using the computer-model analysis,
- To conduct a parametric study to determine the influence of different factors on the magnitude of lateral concrete pressures on wall forms.

In general, the main purpose of this thesis is to provide construction managers, contractors and engineers an additional tool to design and construct safe and economical concrete structures, especially during concrete placing and formwork removal operations.

1.4 Organization of Thesis

This thesis is organized as follows: Chapter I provides an introduction to this study and defines the problem, and the research objectives. In Chapter II, the various types of formwork and shoring systems are discussed. A review of the previous investigations and the existing standards and codes of design of formwork is presented. Chapter III explains the basic concepts of analyses and describes the idealization of both the shoring system and the wall formwork models. Chapter IV presents validation of the present models, which are checked against the field measurements and the several existing methods. Chapter V addresses and identifies the major factors that affect the construction load distribution among shores, reshores and interconnected slabs, and the distribution of lateral pressure exerted by fresh concrete on wall forms. Chapter VI provides a summary, conclusions and recommendations for future work. Input data files for both the shoring system and wall formwork analyses are provided in Appendix A and B respectively.

CHAPTER II

LITERATURE REVIEW

2.1 Forming Systems

The type of forming system has an essential effect in the analysis of concrete structures during construction. Therefore, selecting the forming system, that is, making structural frames faster, simpler, and less costly to build, must begin in the earliest phase of the design efforts. There are several horizontal and vertical forming systems currently used for different structural elements; these are described below.

2.1.1 Horizontal Forming Systems

There are five horizontal forming systems that can be used to support different slab types. These are conventional, flying truss, column-mounted shoring, tunnel, and joist-slab forming systems.

Conventional wood or metal systems may be either hand-set or panelized. Hand-set system in wood or aluminum generally consist of wood shores supporting wood or aluminum stringers and runners or joists, with the deck surface made of plywood. The same type of deck form can be made up in larger panels tied or ganged together and supported on attached scaffold-type shoring. The shoring may be either steel, aluminum, or wood; see Figs. 2.1 and 2.2 (Jensen, 1986).

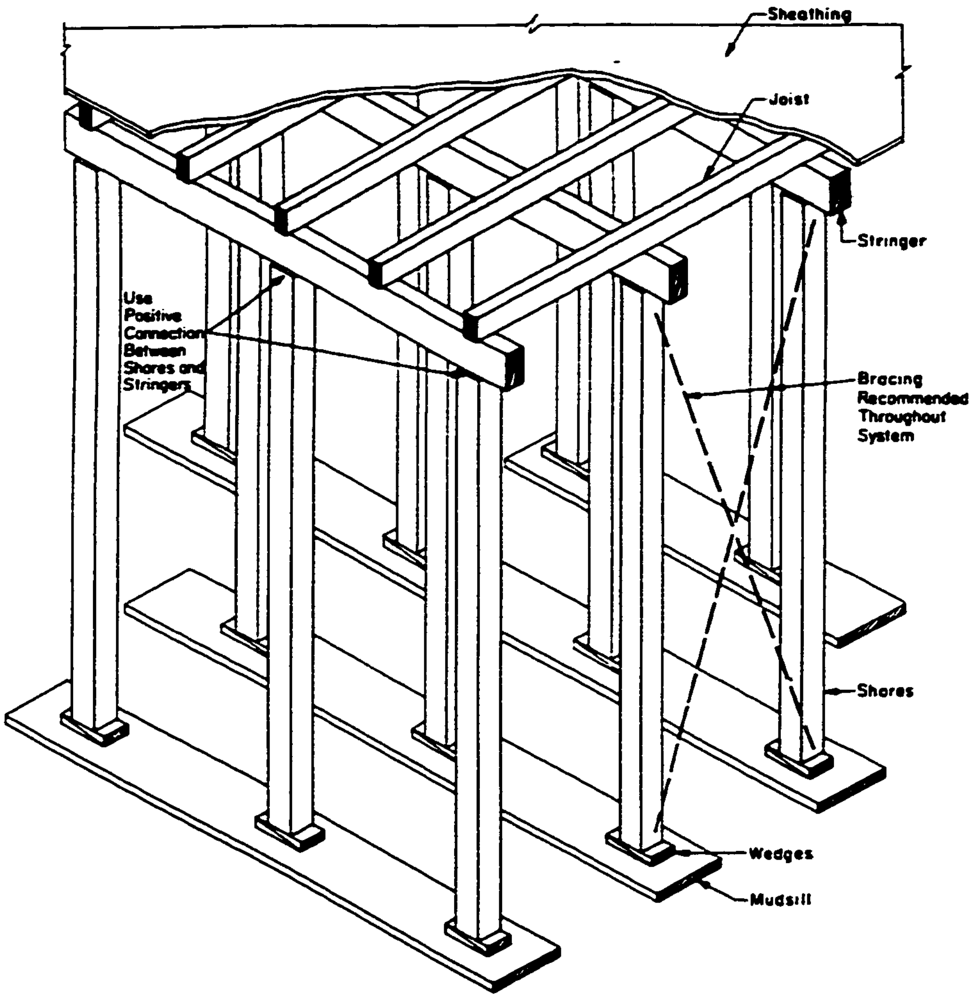


Fig. 2.1 Conventional Wood System
(Hurd, 1995)

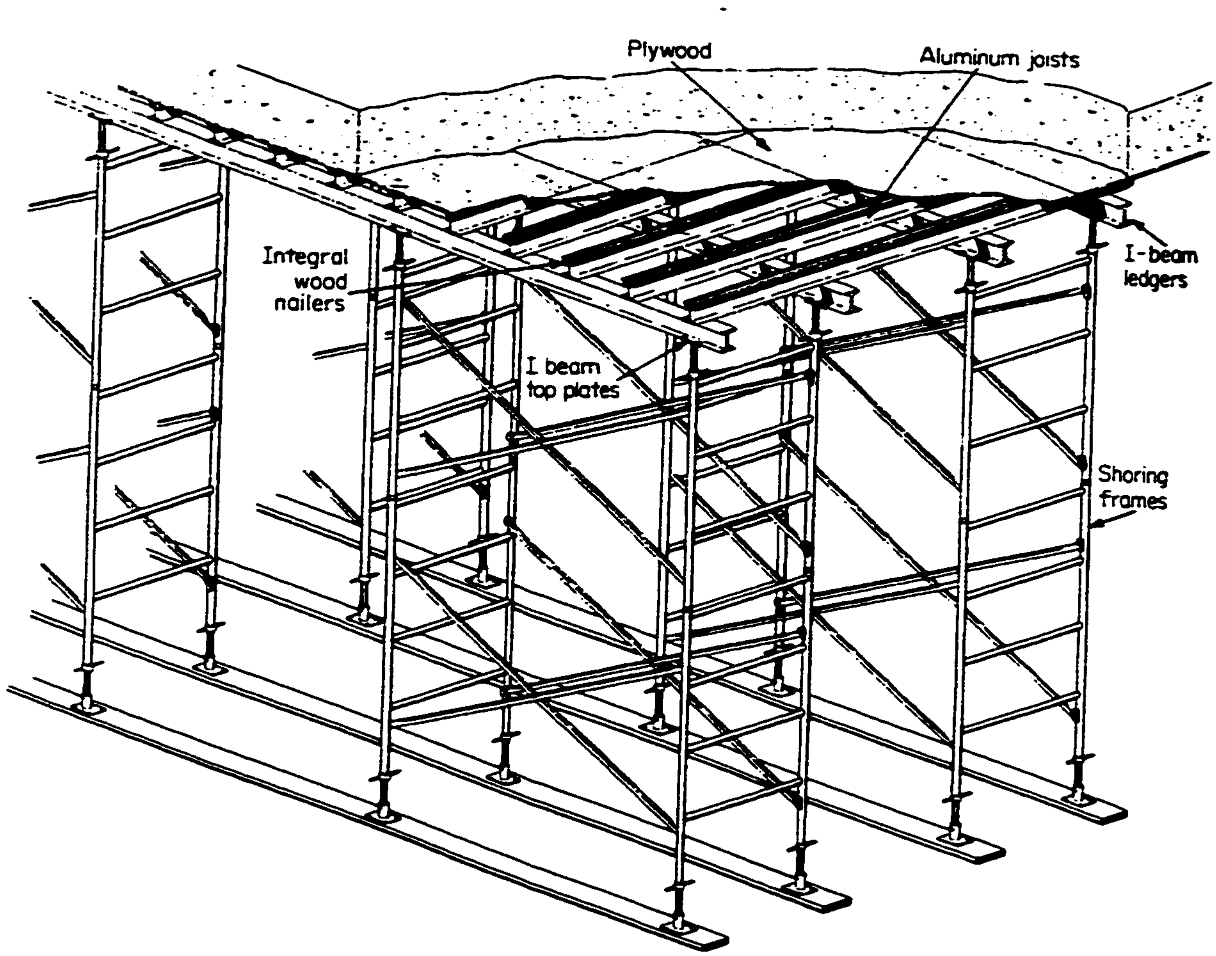


Fig. 2.2 Conventional Metal System
(Ratay, 1996)

Flying truss forms usually consist of large sheathing panels supported by steel or aluminum trusses (beams) (Fig. 2.3). Adjustable shores support the truss on a previously cast slab. The truss-mounted forms are moved by crane from one casting position to the next (Jensen, 1986).

Column-mounted shoring systems are long span form panels supported by up-and-down brackets or jacks anchored to concrete columns. Similar systems available for bearing wall buildings support slab forms on brackets anchored to the walls. The system consists of plywood decking which can be assembled on the ground or in place, once the form has been flown and set on the columns (Fig. 2.4). These systems make it possible to eliminate vertical shoring (Jensen, 1986).

Tunnel forms are factory-made U-shaped or inverted L-shaped steel form systems, which permit casting both slab and supporting walls at the same time (Fig. 2.5). For stripping, after the concrete has gained enough strength, the tunnels are collapsed or telescoped and moved to the next pour. For longer slab spans, the tunnel form may be made in tow inverted L-shapes (Jensen, 1986).

Concrete joist construction is a monolithic combination of regularly spaced joists arranged and a thin slab cast in place to form an integral unit with the beams, girders, and columns (Fig. 2.6). When the joists are all parallel this is referred to as a ribbed slab or one-way joist construction. If the joists intersect each other at right angles, it is two-way

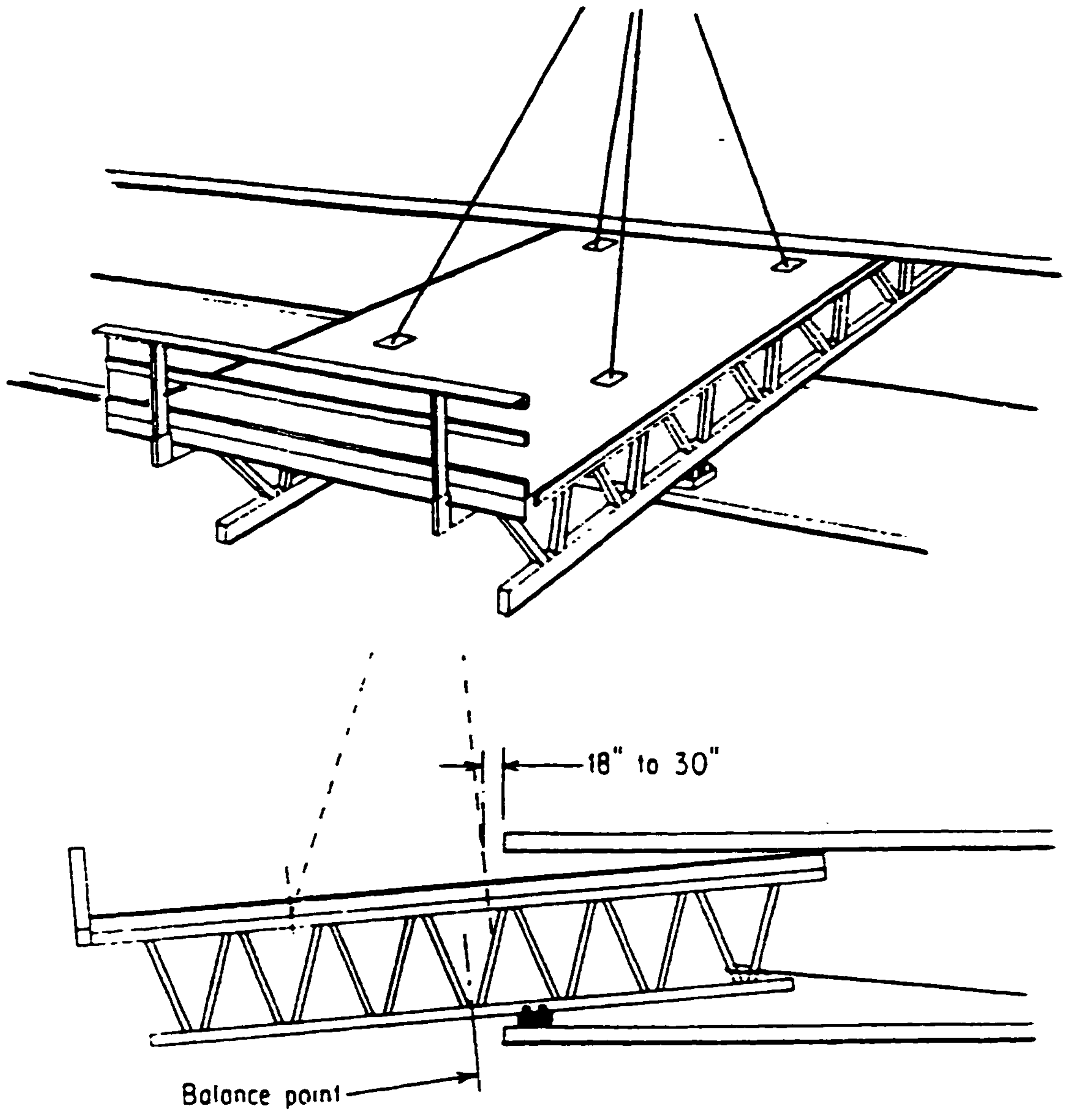


Fig. 2.3 Flying Truss System
(Peurifoy and Oberlender, 1996)

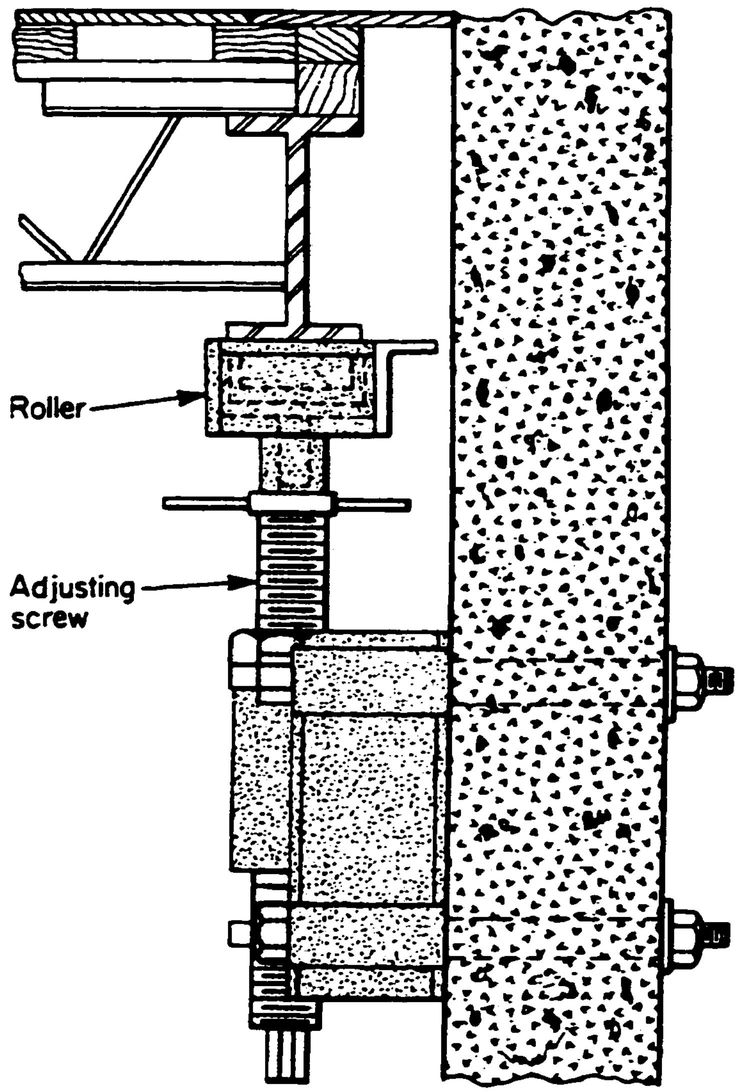


Fig. 2.4 Wall or Column-Mounted Shoring System
(Hurd, 1995)

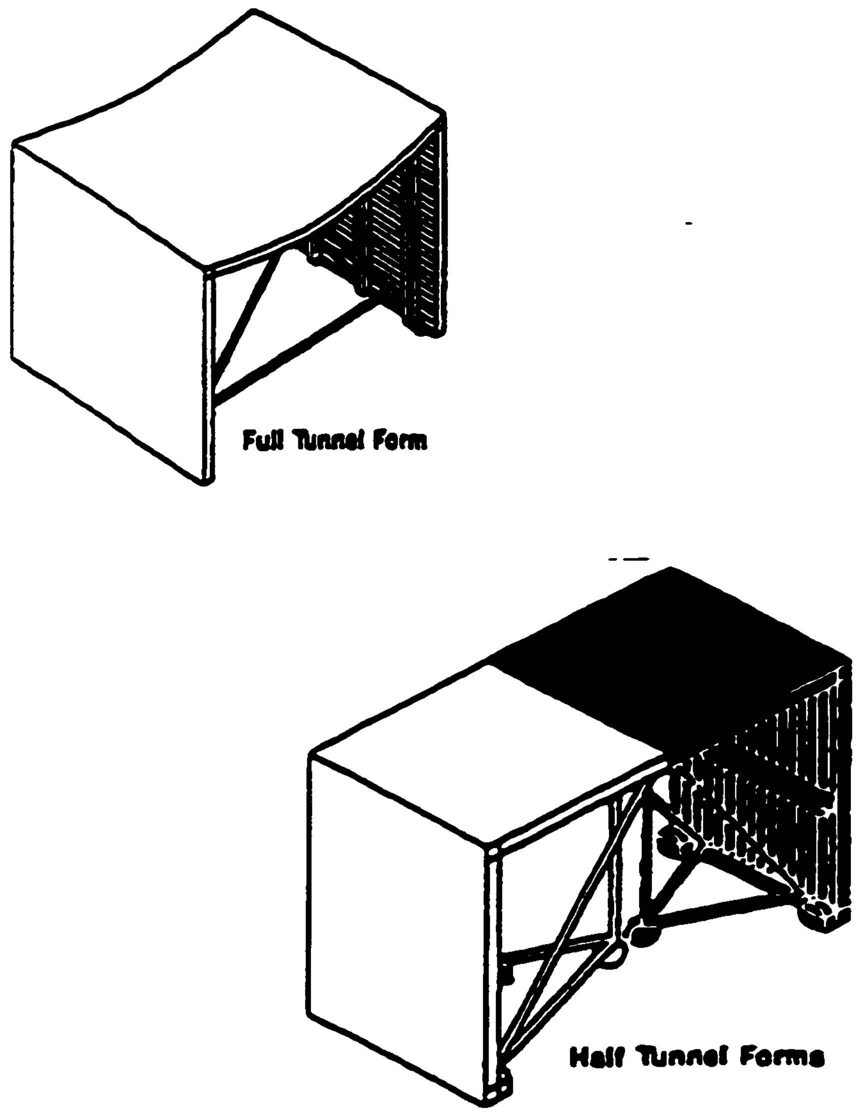
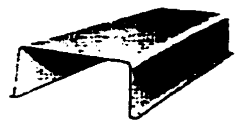
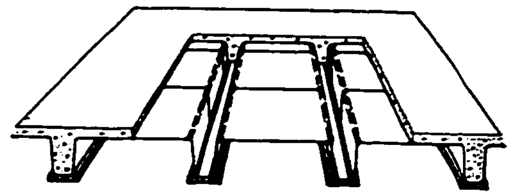
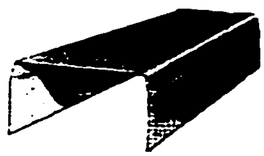


Fig. 2.5 Tunnel Forming System
(Hanna, 1989)

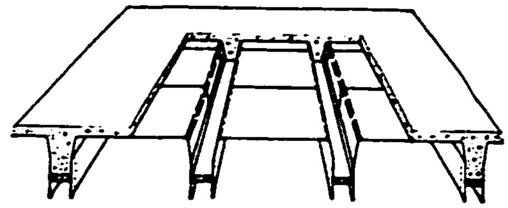


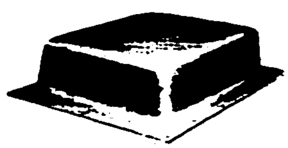
NAIL-DOWN FLANGE TYPE can be placed, aligned, and nailed from the top side. Simplest to use, but does not generally provide architectural concrete surface.



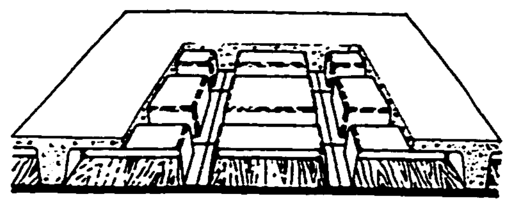


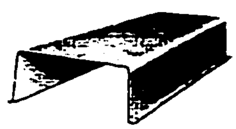
ADJUSTABLE TYPE has no flanges, provides smooth joist bottoms without flange marks. Pans can be removed without disturbing soffits and shore supports.





DOME TYPE used for two-way or waffle slabs. This kind of slab design may offer certain structural economies. Flanges are butted at joints, not lapped.





SLIP-IN TYPE is based on the same form as the nail-down type, but uses soffit board between pans to form a smooth joist bottom. Like adjustables, pans can be removed without disturbing soffit supports.

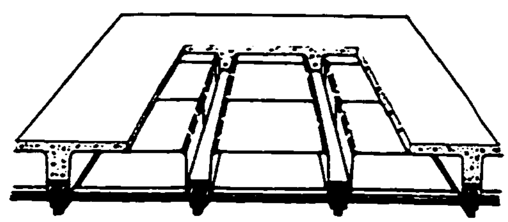


Fig. 2.6 Joist-Slab Forming System (Hurd, 1995)

joist construction or waffle slab. These types of construction have been frequently formed with ready-made steel or other materials pans or domes of standard sizes (Hurd, 1995).

2.1.2 Vertical Forming Systems

Vertical forming systems are those used to form the vertical supporting elements of the structure (i.e. columns, walls). Five types of vertical forming systems are available. These are conventional, gang, slip, jump, and self-raising forms.

Conventional forming system consists of five basic parts (Fig. 2.7): sheathing, made of plywood or lumbers to shape and retain the concrete until it sets; studs, to form framework and support the sheathing; single or double wales, to keep the form aligned and support the studs; braces, to hold the forms erect under lateral pressure; and ties and spreaders or tie-spreader units, to hold the sides of the forms at the correct spacing (Smith and Andres, 1993).

Gang forms are made of large panels joined together with special hardware and braced with strongbacks or special steel or aluminum frames (Fig. 2.8). These large panels are usually built or assembled on the ground and raised into place by crane. In high wall construction, the ganged panels are frequently raised by crane to the next pouring position, and supported by steel brackets fixed to the lift below (EFCO, Forming Systems Catalog, 1988).

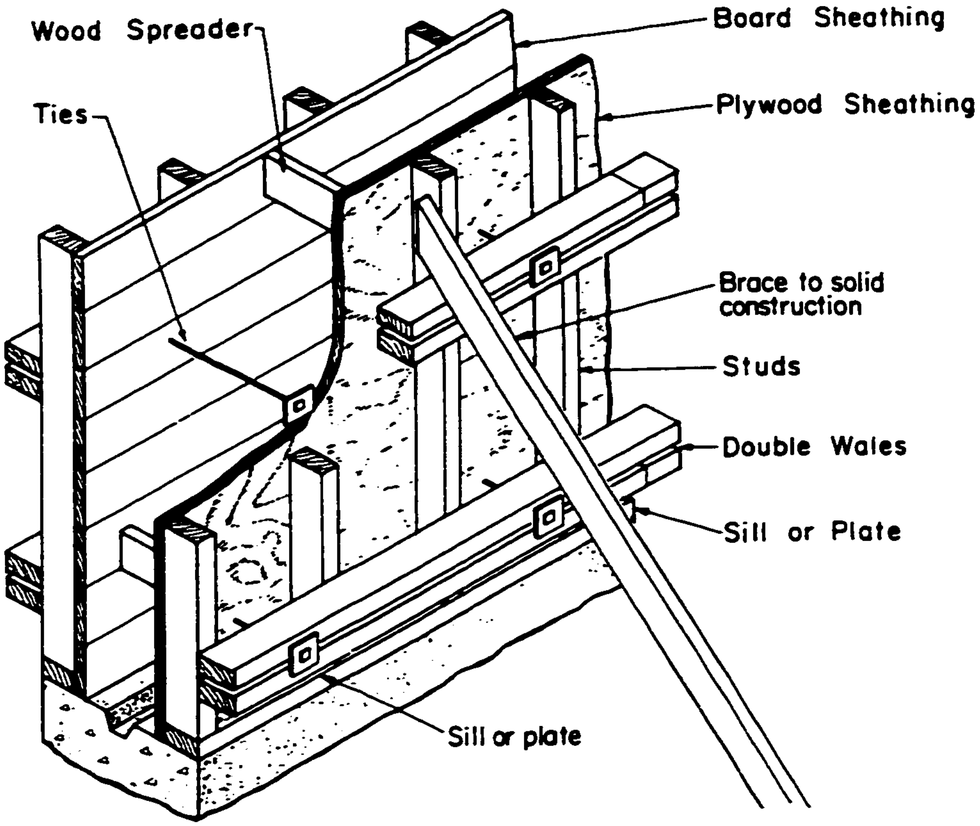


Fig. 2.7 Conventional Wall System
(Hurd, 1995)

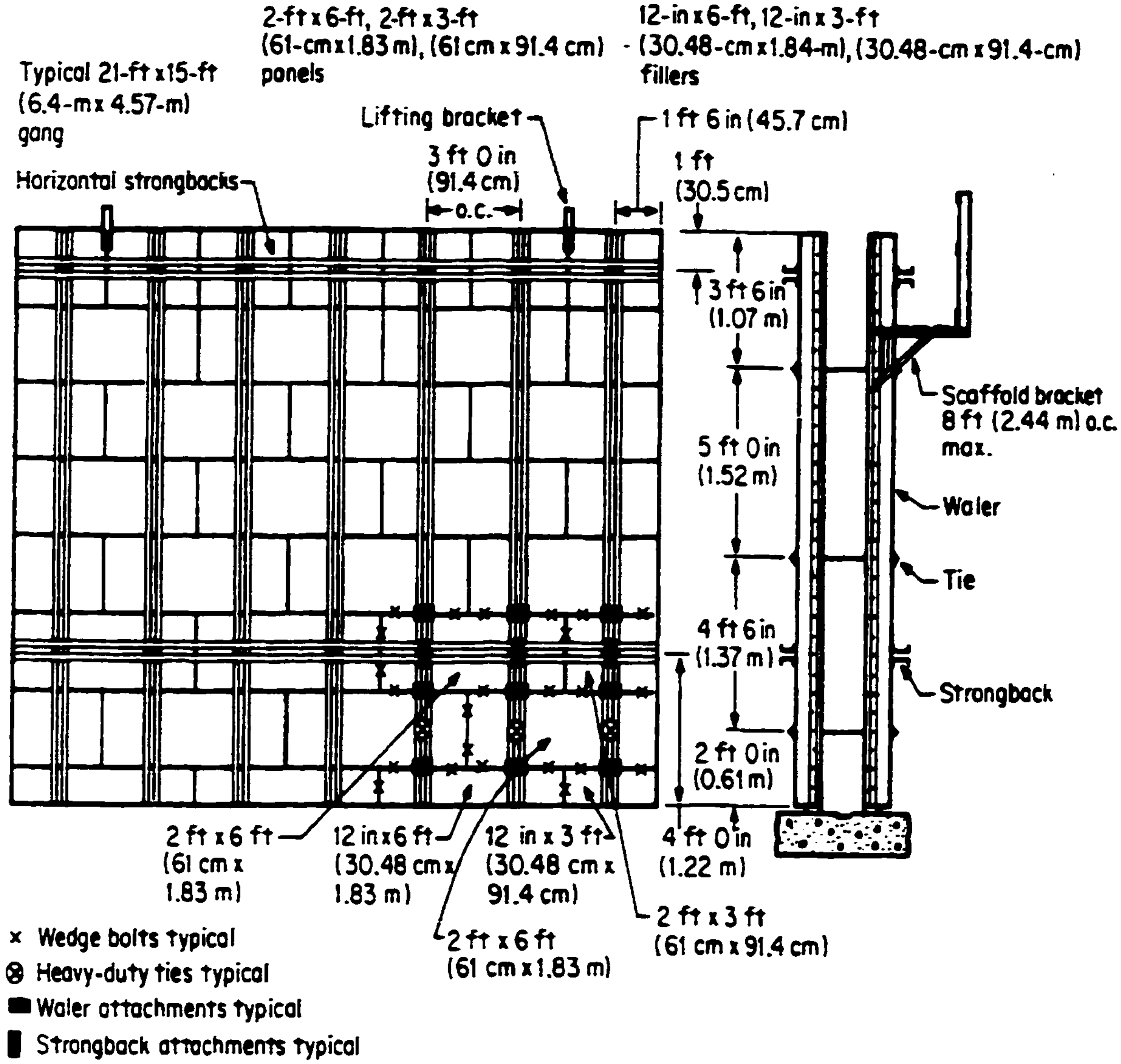
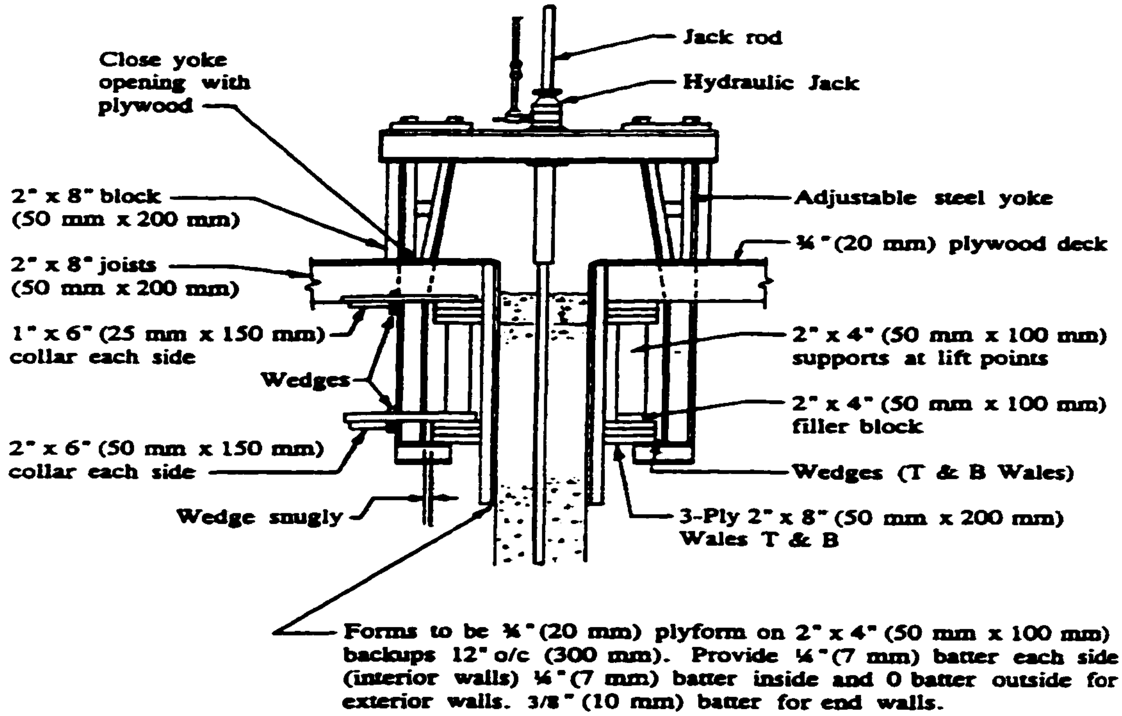


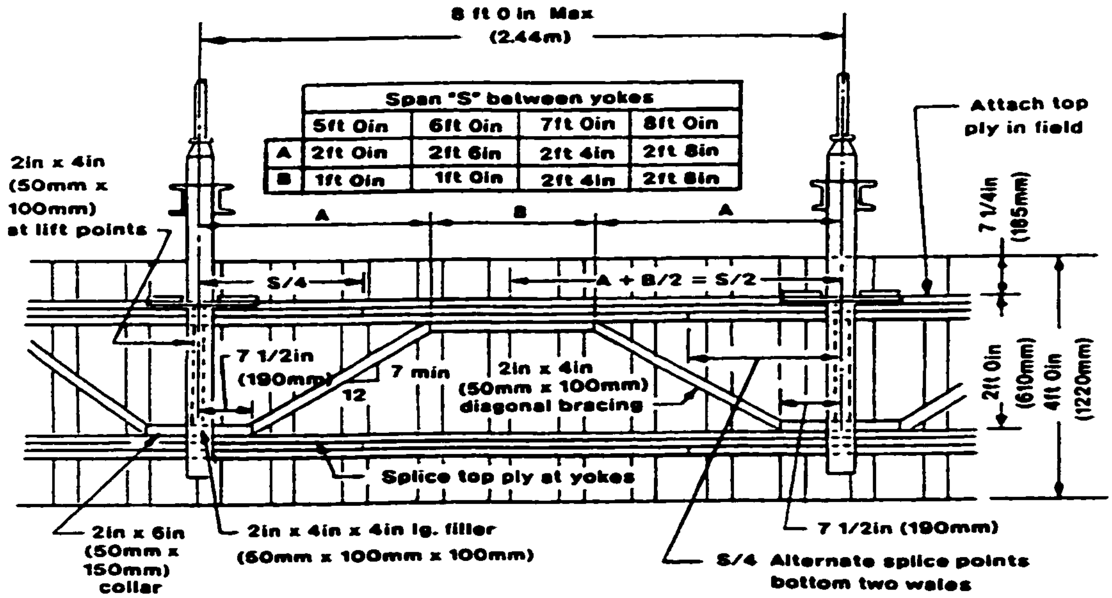
Fig. 2.8 Gang Form Made Up of Hand-Set Modular Panels (Ratay, 1996)

Slip forming is originally designed for curved structures such as bins, silos, and towers, for which the conventional forming systems are not suitable. However, slip forms are now used for a variety of structures, including rectangular buildings, bridge piers, and canal lining. Slip forms consist of inner and outer forms, 3 to 5ft in height. Forms may be fabricated from wood or steel, and supported by strong vertical yokes. These yokes are tied together at the top to give the form sides the rigidity needed. The bottom ends of the form sheathing are slightly tapered to help make the form self-clearing. To provide a platform, from which workers may look after placing the concrete in the forms, a working platform is attached to the inner form and rides upward with it. At the same time a finisher's scaffold is suspended from the outer form so that workers can finish the newly extruded concrete as it emerges. Slip forms move continuously upward, drawn by jacks climbing on vertical steel jack rods. These are anchored at the base of the structure and embedded in the concrete below the forms. The jacks may be hydraulic, electric, or pneumatic and are capable of producing form speeds of up to 20 in/hr. Concrete is placed into the form at the top end as it is drawn upward. The continuous process is carried on, filling and moving the forms upward, often 24 hours a day until the structure is complete. Concrete joint between lifts is to be carried out if the operation stopped. Fig. 2.9 shows a schematic diagram of slip form (Smith and Andres, 1993).

Jump forms (Fig. 2.10) are used where no floor is available on which to support the wall formwork, or the wall and column proceed ahead of the floor. Jump-forms consist of a framed panel attached to two or more strongbacks. They can be one floor high, supported



(a) Section through Typical Slip-Form



(b) Side View of Typical Slip-Form

Fig. 2.9 Typical Slip-Form
(Ratay, 1996)

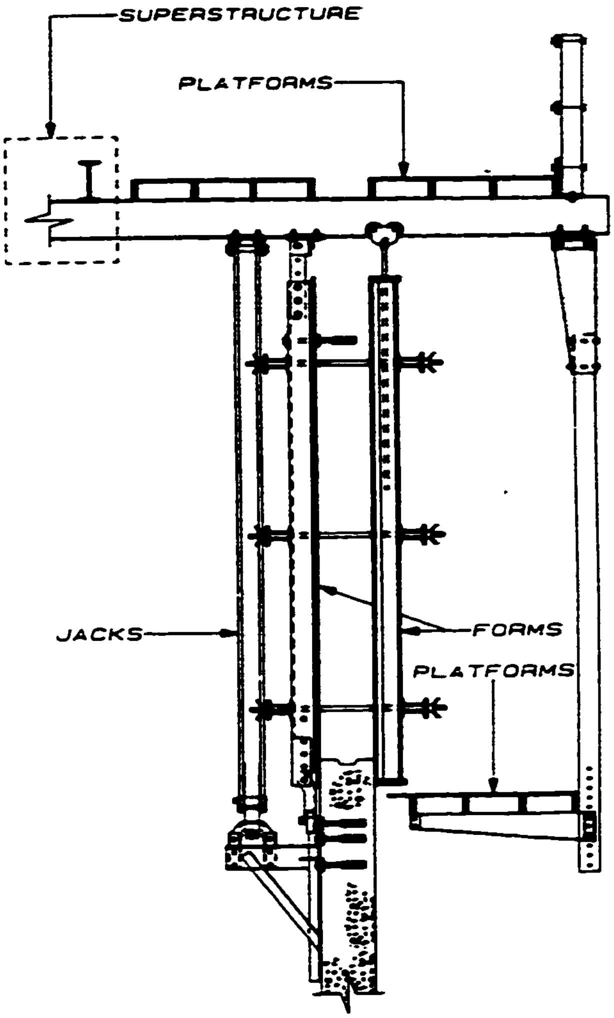


Fig. 2.10 Jump Forming System
(Ratay, 1996)

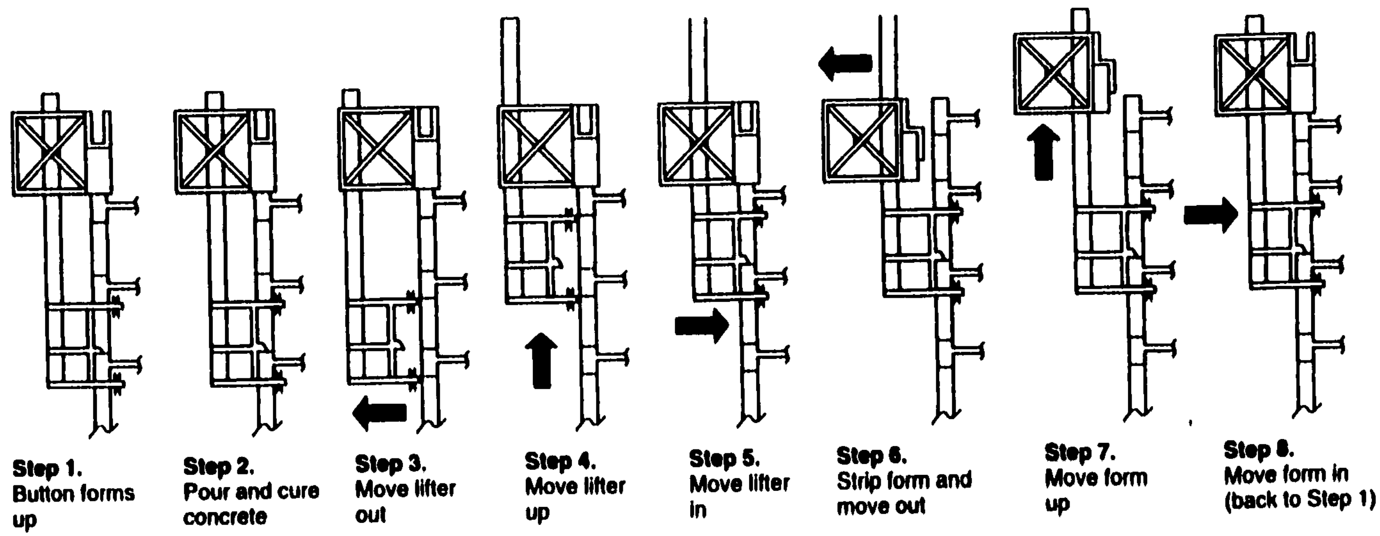
on inserts set in the lift below. Two sets, each one floor high, that alternately jump past each other can also be used (PATENT Scaffolding, JumpForm, 1986).

Self-raising forms retain all the basic characteristics of jump forms that is, unit lifts one lift at a time, with no movement of plastic surface contact areas of concrete relative to the forms. It is necessary to move the forms laterally away from the placed concrete before they can be lifted to the next position and again moving them laterally back to the forming location once they have been lifted. They are not an extrusion-type forming system like slip-form. The self-raising forms are more economical than the jump forms because of the elimination of the necessity for heavy use cranes and hand labor in the handling, stripping, and reinstallation of the forms. The system consists of upper form(s) and lower lifters (self-raisers) (Fig. 2.11). The lifters are attached to the wall already cast below the form (Ratay, 1996).

2.2 Design Loads

Numerous loads and pressures, vertical and horizontal, live and dead, must be taken under consideration during the design of formwork/falsework facilities (Lew, 1985). The design loads for formwork can be divided into four categories: vertical; lateral pressures; horizontal; and special loads.

Vertical loads consist of dead and live loads. The weight of the formwork, shores, reshores, lateral bracing and fresh concrete and reinforcement is considered as dead load. The live loads include the weight of workmen, equipment, material storage, and impact produced by concrete placement and equipment. While the dead load can be estimated



Typical cycle for post-lifter form involves two basic operations. First, forms, which encase cured concrete, are used to raise the lifter. Then the lifter is reattached to the building, and after forms are released it raises them to the next floor.

Fig. 2.11 Self-Raising Form for High-Rise Building
(Ratay, 1996)

with reasonable certainty, the live load can vary significantly depending on the construction process. ACI 347R-94 (1994) recommends a minimum value of 2.4 kPa of horizontal projection for live load ("Until an accurate assessment of actual live loads is made, 2.4 kPa or greater for design live load would be desirable",). When motorized carts are used the minimum live load should be 3.6 kPa. Regardless of slab thickness, however, the minimum design value for combined dead and live loads is 4.8 kPa.

Lateral pressures of fresh concrete impose loads against wall or column forms. As a result of various studies, several recommended and proposed procedures for empirically estimating pressures have been developed. Each method assumes that concrete pressure increases linearly with depth to a maximum value, P_{max}, and remains constant thereafter. However, the procedure for determining P_{max} varies significantly in the proposed methods (Johnston, Kahn and Phillips, 1989). The recommended and proposed methods are described in Section 2.8.

The most serious deficiency in formwork is in the consideration of horizontal (or lateral) loads. Many practical design guides do not include any lateral load provision to warn the designer of the importance of adequate lateral bracing to insure the stability of the falsework. Horizontal loads arise from wind, guy cables, inclined supports, impact of concrete during placement, and the starting and stopping of equipment. Horizontal loads may also arise from the sequence of concrete placement and thermal effects. Most codes and standards, including ACI Standard 347R-94 (1994) and ANSI Standard A10.9 (1983), require the formwork to be designed for horizontal loads in any direction of not less than1.5 kN/m of floor edge or 2% of the total dead load of the floor whichever is greater. Wind loads may be more important for falsework than in permanent structures, since falsework is generally relatively light and provides little inertial resistance to wind such that provisions for resistance must be additionally provided in the form of wind braces or guy cables. Some codes and standards require a minimum design wind load of 480 or 720 Pa. A more rational approach to determine the design wind loads should be taken from the ANSI Standard A58.1 (1982).

Special loads include all temporary and accidental construction loads occurring during concrete construction, such as unsymmetrical placement of concrete, concentrated loads of reinforcement and storage of construction materials. Both the designer and the contractor should be alert to provide resistance against these special loading conditions.

2.3 Factors to be Considered in the Analysis of Shoring and Reshoring of Multistory Buildings

In determining the number of floors to be shored, reshored or backshored and to determine the loads transmitted to the floors, shores, and reshores or backshores as a result of the construction sequence, ACI Committee 347R-94 (1994) recommends consideration of the following factors:

1. Structural design loads of the slab or member including live loads, partition loads, and other loads for which the engineer designed the slab. Where the engineer included a reduced live load for the design of certain members and allowances for construction loads, such values should be shown on the

structural drawings and be taken into consideration when performing this analysis.

2. Dead load weight of the concrete and formwork.
3. Construction live loads, such as placing crews and equipment or stored materials.
4. Design strength of concrete specified.
5. Cycle time between placement of successive floors.
6. Strength of concrete at the time it is required to support shoring loads from above.
7. The distribution of loads between floors, shores, and reshores at the time of placing concrete, stripping formwork, and removal of reshoring.
8. Span of slab or structural member between permanent supports.
9. Type of formwork systems, i.e., span of horizontal formwork components, individual shore loads, etc.
10. Minimum age where appropriate.

2.4 Formwork Capacity

Overall strength of the falsework depends on the strength and stiffness of the formwork, the shoring together with lateral bracing, and the support, which provides reaction to the loads, transmitted through shores. Factors of safety used in each of these components vary from 1.5 to 4.0. Some of these factors are based on allowable stresses and others are based on ultimate load tests (Lew, 1976).

Both wooden and metal forms are subjected to bending, shearing, and compression stresses, each of which must be kept within acceptable allowable limits. In addition to strength requirements, deflection limitation must be taken into consideration. The adequacy of shores and reshores can be determined simply by checking that the uniformly distributed loads calculated from the slab/shore analysis multiplied by an appropriate tributary area are less than the safe working loads of the shores.

Formworks are used for many times for different projects. Therefore, the quality of formwork elements should be checked before reuse. Sufficient lateral braces should be provided to maintain integrity of supporting system in case of uplifting to prevent shore buckling and resist horizontal load (Duan and Chen, 1996).

2.5 Concrete Strength

Formwork should be removed as soon as possible to provide the greatest number of uses, but not until the concrete has attained sufficient strength to ensure structural stability and to carry both the dead load and any construction loads that may be imposed on it. The prediction of concrete strength is very important because it is the main decisive factor for the earliest possible removal of formwork. In general, the decision regarding the safe formwork removal depends on the rate of strength gain in the concrete, the accuracy of strength determination of in-place concrete and the level of temporary stress and deformation that the structure can withstand (Lew, 1976).

Determination of early-age strength is usually based on the results of tests performed either on concrete specimens or on structural members. Both ANSI A10.9 (1983) and ACI 347R-94 (1994) recommend the use of test results of field-cured standard 6×12-in cylinders. Carino, Lew and Voz (1983) pointed out that the curing history of the cylinders and in-place concrete will not be identical, and strength development of the cylinders and the structure will not be the same simply because of the difference in shape and size of the actual structural members. Therefore, it is desirable to have reliable nondestructive test methods to estimate the concrete strength for formwork removal operations. Several nondestructive methods are available for estimating the in-place strength of concrete. Such methods include; rebound hammer (ASTM C805), penetration (Windsor) probe (ASTM C803), pulse velocity measurements (ASTM C597), pullout test (ASTM C900), and maturity methods (ASTM C918). Most of these methods measure some properties of concrete, like amount of rebound, depth of penetration, velocity of ultrasonic pulse propagation, concrete daily temperature etc., from which an estimate can be made for the concrete strength.

Maturity-based model is an alternative approach to the use of field-cured cylinder strength for predicting the in-place strength of maturity, which does not involve any field-testing. The strength-maturity relationship of concrete mixes was developed on the basis that the strength of concrete depends on the curing time-temperature history of concrete. Maturity represents quantitatively the cumulative effects of temperature and time up to any given age. Based on the work of Saul (1951) and others, Gross and Lew (1986) proposed the maturity-based model of concrete strength prediction, which presented in the following equations:

$$m = \int (T - T_0)dt \tag{2.1}$$

where m is the maturity defined as the integral over time of the temperature, T is the temperature of concrete (^{o}C), and T_0 is the datum temperature, that is, the temperature below which there is no further strength gain with time.

$$\beta = [0.01222m/(1+0.01222m)]+31.40/28.00 \tag{2.2}$$

where β is a ratio of early age compressive strength to design strength or is a modification coefficient which depends on the concrete age, temperature, type of cement, and the critical mode of structural member behavior.

Equation (2.2) is based on results of compression tests conducted at 7 and 28 days respectively under standard curing condition (T=22.8 ^{o}C). The specified compressive strength for design was 28.00 MPa.

2.6 Slab Capacity

To insure a safe construction scheme, the capacity of the slabs taking into account the reduced strength of the immature slabs, must be compared with the slab loads determined by a shoring analysis at various stages of construction. If a slab load exceeds its capacity, then the proposed construction procedure cannot be used and one or more measures must be taken to either increase the strength of concrete slab or reduce the load on the slab at the critical age. Several possibilities for doing this include increasing concrete strength at the time of loading by using heat, higher strength concrete or high early strength cement,

Table 2.1 Development of Concrete Strength (Gross and Lew, 1986)

Age(days)	Type I Cement		
	22.8°C	12.8°C	4.4°C
1	0.25	0.16	0.06
2	0.41	0.28	0.11
3	0.53	0.37	0.16
4	0.61	0.44	0.21
5	0.67	0.51	0.25
6	0.72	0.56	0.29
7	0.76	0.60	0.32
8	0.79	0.64	0.35
9	0.82	0.67	0.38
10	0.84	0.70	0.41
11	0.86	0.73	0.43
12	0.88	0.75	0.46
13	0.90	0.77	0.48
14	0.91	0.79	0.50
21	0.98	0.88	0.61
28	1.00	0.93	0.69

Note: For a given treatment, β can be obtained by linear interpolation.

changing the construction cycle, or modifying the number of shores or reshores stories to produce a more favorable distribution of construction loads. The strength capacity of a reinforced concrete slab is a function of the design ultimate load, age of the slab, construction temperature, type of cement, and the critical modes of the slab behavior (Mosallam and Chen, 1990).

Mosallam and Chen (1990) presented a method to check the slab adequacy, which is similar to that given by Gross and Lew (1986) except that it uses the simplified method and it accounts for construction live loads. The following equation was derived to check the slab adequacy at any construction stage, t.

$$C_t/D \leq \beta(1.0 + 1.2\, L/D) \tag{2.3}$$

where C_t is a construction slab load at age t, β is a ratio of early age compressive strength to design strength, and D and L are dead and live loads respectively.

C_t/D is the maximum permissible slab load ratio determined from analysis such as simplified method, and it can be expressed in terms of the strength ratio, β, and the live-to-dead load ratio, L/D. β can be determining using either Table 2.1 or maturity-based model described in Section 2.5.

2.7 Previous Researchs on Construction Load Distribution in Shoring System

Neilsen (1952) presented a method of analysis for determining the load distribution among the slabs and the supporting shores in multistory building structures. An extensive theoretical analysis was used to calculate those load ratios. The analysis was based on the following assumptions: the slabs behave elastically, shrinkage and creep of concrete were neglected, shores were represented by uniform elastic supports, the base support (foundation) is assumed to be rigid, and the torsional moments and the shearing forces in the formwork were neglected. However, this method of analysis seems lengthy and inconvenient for practical use.

Grundy and Kabaila (1963) developed a simplified method for determining the construction loads carried by the slabs and the shores during the construction of multistory concrete buildings. This method assumed that the axial stiffness of shores and reshores are infinite relative to the stiffness of slabs, shrinkage and creep of the concrete are neglected, and the reactions of the shores are uniformly distributed. Blakey and Beresford (1965), and Feld (1974) extended the method of Grunday and Kabaila by considering different slab-shore systems and construction rates.

Some analytical studies were done to verify the Grunday-Kabaila's simplified method by comparing the calculated values with field measurements. Agarwal and Gardner (1974) compared the calculated maximum load ratios of slabs, shores and reshores with the field measurements of the construction loads from two high-rise-flat-slab buildings. They showed that the field measurements of construction loads agreed with acceptable accuracy. Lasisi and Ng (1979) also compared the calculated load ratios by Grunday-Kabaila's simplified method with field measurements, and found that the measured reshores loads were lower than the predicted loads. They modified the Grunday-Kabaila's method by considering the construction live loads. Sbarounis (1984) extended Grunday-

Kabaila's simplified method to include the effect of cracking on the stiffness of supporting slab. He concluded that the overloading of the early age concrete affects the long-term deflections as well.

Nowadays, the use of computer-aided programs in construction industry has becomes very common. Many attempts have been made either to translate hand-calculation techniques directly into computer code or to develop an analytical computer model for determining the construction loads distribution between slabs and shores system in multistory structures during construction. Liu, Lee and Chen (1988) developed a computer program by using the mathematical formulation of the simplified method for several possible construction operations. This program can analyze construction loads on shores and slabs during different steps. It also can check slab safety at any construction stage. Liu, Chen and Bowman (1986) checked the major assumptions of Grunday-Kabaila's simplified method by developing a refined 2-D computer model. They found the simplified method adequate for predicting the construction step and location of maximum slab moments and shore loads, and proposed to correct the numerical values of slab moments and shore loads by using a modification coefficient. Mosallam and Chen (1991) improved the 2-D computer model developed by Liu, Chen, and Bowman by considering more construction parameters and procedures, and then compared with simplified method. This study showed that the simplified method is adequate for predicting the construction location of the maximum slab and shore loads, however, it generally overestimates the actual load ratios. They also suggested that the construction live load must be considered in the analysis of construction loads. Duan and Chen

(1995a) conducted a study to improve the simplified method for slab and shore load analysis during construction. This study considered the effect of elasticity of shores on the load distribution in a simple manner and treated the simplified method as a special case. Comparisons were made with the simplified method and refined finite element methods as well as with some field measurements. They found that the load distributions in supporting systems during concrete placing and during shore removal are different with inclusion of the elasticity of shores.

Liu and Chen (1985) developed 3-D model (Purdue university model) to simulate a variety of construction processes. They examined the influences of boundary condition of slabs, slab and shore stiffness, foundation rigidity, column deformation, slab aspect ratio, and the effect of different shore stiffness distributions. They concluded that all of these factors have insignificant influence on the load distribution during construction. Liu and Chen (1987) proceeded with a further refinement of computer model by including in the analysis the material creep properties and the nonlinear behavior of shores in analysis. They found that those factors have a little influence on load distribution.

Stivaros and Halvorsren (1990) developed a 2-D structural computer model according to the equivalent frame method requirements as described in the ACI Code. They examined the influence of slab continuity, slab type and stiffness, type of shoring system and its stiffness, extra support capacity provided by columns, method and rate of construction load distribution in multistory building during construction. They found that the correction coefficient 1.05 to 1.10 of the simplified method suggested by Liu, Chen and

Bowman is not adequate for all cases of shoring schemes. They also concluded that the slab stiffness acquired from the aging of concrete during the construction does not significantly affect the construction-load distribution among the slabs, and the most important factor affecting the construction-load distribution is the stiffness of the shoring system.

Duan and Chen (1995b) established finite element structural models to verifying the effects of non-uniform loading and impact load on shore loads for a one-level shoring system supported on a slab during concrete pouring and shore removal. This study is based on the field measurement project of concrete building at Beckley site, West Virginia, which is conducted by University of Vermont. They extended the study to the cases of one-level of shoring on a rigid ground and multi-level shoring during construction. They concluded that the construction loads during concrete placing transmit to shores and supporting slabs in different ways, the maximum shore loads in the uppermost level of shoring system are influenced by concrete placing sequences. It was also found that the non-uniform load effect has little influence on slabs and shores in lower parts of a multi-level supporting system and that the uplift or swift effects for shores may occur during concrete placing.

Mosallam and Chen (1992) developed a 3-D computer model to simulate the concrete construction process by considering the interaction of gravity and lateral loads, and to examine the effect of the construction process in the analysis of multistory concrete buildings. This model is based on the following assumptions. The slabs and the columns behave elastically and their stiffness is time-dependent. The shores and reshores act as continuous uniform elastic supports with equal axial stiffness, the braces also have equal stiffness, and the joints between the shores (or braces) and the slabs are pin-ended. The foundation is rigid and unyielding, and the columns are cast one week ahead of the floor slab. They concluded that the effect of lateral loads on the construction load distribution is very small and can be neglected for practical purposes, and the temporary lateral bracing of shores is very important for safety and stability of the entire formwork assembly.

Taylor (1967) suggested a method for formwork stripping to reduce load on the interconnected slabs. This method is based on technique of slackening and lightening of the shores. The performance of this technique requires a strict process in order to meet the desired sequence of construction. Marosszeky (1972) went a step further and suggested a method of complete release of shores before reshoring of the floor slab. This technique allows the slab to deflect and to carry its own weight, and then the reshores are installed allowing the construction loads to be shared by the interconnected slabs. He found that this technique allowed less load to be carried by each slab in comparison with the undisturbed shoring technique.

Gardner (1985) presented an analytical method for evaluating slab capacity, and the distribution of different kinds of construction loads that should be considered in the design of formwork and their treatment in existing codes and standards. He also suggested several procedures, which may lead to safe construction of multistory structures.

Gardner and Chan (1986) studied the effects of the pre-shoring technique during construction of multistory buildings. Preshoring or scheduled reshoring is an attempt to reduce the adverse effects of clear bay reshoring by reducing the area stripped before reshoring. This technique can reduce the slab deflections by reducing the unsupported span lengths. The loads in the reshores used in a preshore schedule are higher than those in a clear strip reshore schedule. With respect to the slab load ratio, preshoring has a little advantage compared with traditional reshoring and requires a close control of construction process.

El-Sheikh and Chen (1988) studied the effect of fast construction rate on short and long-term deflection of reinforced concrete buildings. They found that the fast construction rate would not affect the short-term or long-term deflection. Duan and Chen (1995c) investigated the effects of creep and shrinkage on slab-shore loads and deflection during construction by using the method of creep rate. In this study, the change in shore loads was assumed to be linear during curing. They verified this method against the field measurements of one-level supporting system at Beckley site, West Virginia, and then used it to study the multi-level shoring cases and to compare with other researches. They also extended this study to investigate the influence of the variation of slab-shore stiffness ratios on the changing rules of slab-shore loads during curing, and the influence of the construction load on the serviceability by considering the long-term deflection and the degree of cracking in slab. They suggested modifying the simplified or the refined simplified methods by considering the creep and shrinkage effects on supporting systems for a cycle of about 7 days.

Peng, Yen, Lin, Wu and Chan (1997) examined the behavior of a large full-scale scaffold frame shoring subjected to pattern loads with various load paths. This test was performed by using sandbags to simulate the placed fresh concrete during construction. They found that the effect of influence surfaces in scaffold frame shoring is not significant during load paths, the maximum axial tube forces with different load paths do not differ much from that of the uniform load used in a typical design calculation. It was also found that the regions of some larger axial forces in the scaffold frame vary with the variation in load paths.

Kothekar, Rosowsky and Huston (1998) investigated the adequacy of vertical design loads for shoring during construction. This study was accomplished by comparing actual load data from six multistory buildings with design loads prescribed by existing codes and standards: ANSI A10.9, ACI 347, the OSHA standard and the proposed ASCE construction load standard. This study confirmed the need for continuous measurements during the pouring process to characterize accurate shore loads. They found that the tributary area load provides a good estimate of the maximum average shore load, the live loads on the early age slab may increase again as preparations are made for shoring the next level, and ACI and ANSI A10.9 provide reasonable estimates for the maximum shore load.

Karshenas and Rivera (1996) investigated the performance of used Ellis shores and the factors most influencing their strength. The shores made of two pieces of dimension lumber, which are joined by a patented clamping device to permit length adjustments, are called Ellis shores. One hundred and four Ellis shore samples were randomly collected from eight concrete building projects. The collected samples were tested in compression with lengths ranging from 2.4 to 4.0 m and out of plumb angles from zero to 3 degrees. They studied the observed physical conditions and the ultimate loads of the collected samples. During the construction site visits for sample collection, out of plumb angles of Ellis shores installed in slab formwork were measured, also statistics of the observed out of plumb angles were provided. They found that the factors such as shore installation workmanship, quality of lumber used, clamp condition, shore end condition, and number of shore reuses have significant influence on the load bearing capacity of Ellis shores. Karshenas (1997) investigated wooden slab formwork installation inaccuracies and strength variabilities. This study was based on laboratory experiments on shore, stringer, and joist samples obtained from eight multistory concrete building projects. In addition to formwork member sample collection, installed shore out-of-plumb angles and stringer, joist, and sheathing spans were also measured. In this study, Karshenas investigated the formwork member physical conditions, installation inaccuracies, shore compressive strengths, and bending and shear strengths of the stringers and joists. He also summarized the observed structural properties with appropriate probability distributions.

Webster (1980) outlined a reliability methodology that gives the designer, contractor and owner an analytical tool with which to weigh alternative design and construction methods against progressive collapse. The study examined the influence of design decisions and construction control procedures on the probable collapse performance. He concluded that the designer and contractor share the responsibility in determining the reliability against progressive failure during construction.

Mosallam and Chen (1990) examined some factors relating to the analysis of multistory reinforced concrete buildings during construction, including causes of falsework failures, and a review of existing codes and national standards as they affect safety in construction. They found that a few studies were conducted on the development of analysis models to simulate the construction process; moreover, non-of those models was chosen or confirmed by code or standard writing bodies. All of previous analytical models did not include the effect of lateral bracing on the maximum slab or shore load distribution and it was impossible to recommend a single general procedure for shoring and reshoring in multistory structures.

2.8 Previous Researches on Concrete Pressure on Wall Formwork

Rodin (1952) collected and critically reviewed the published experimental data on the lateral pressures of concrete against forms. He presented a rational explanation of the physical phenomena causing the types of pressure distribution found in practice, and explained why this pressure is not hydrostatic, except in special circumstances. He also discussed the factors affecting the lateral pressure such as the rate of concrete placement, the method of placing the concrete, the consistency and proportions of the mix, the temperature of the concrete, the rate of setting of the concrete, and the size and shape of the form. Rodin concluded that where external vibrators are used, the full depth of concrete would be fluidized and the formwork should be designed for full hydrostatic pressure. For internal vibration, he proposed some formulas and curves to determine the lateral concrete pressures. These formulas were based on a concrete mix having proportions of 1:2:4 (by weight) with a slump of 150 mm at a temperature of 21°C. Rodin's formulas are:

$$h_{max} = 1.63R^{1/3} \tag{2.4}$$

$$P_{max} = 23.5h_{max} \tag{2.5}$$

where concrete density is assumed to be 2400 kg/m3; h_{max} = head at which maximum pressure occurs, m; P_{max} = maximum lateral pressure, kN/m^2; and R = rate of placing, m/hr.

ACI Committee 347R-94 (1994), which was formerly ACI Committee 622 (1958) collected and analyzed the existing literature and test reports, and then developed design recommendations and formulas for determining the magnitude and distribution of lateral pressure on concrete formwork. It was proposed that for design purposes, the lateral pressure envelope should be hydrostatic up to some limiting value and then constant at the limiting value. The objective of the ACI Committee 347 was to keep the determination of pressure straightforward with a minimum of variables and assumptions. The Committee concluded that placement rate, concrete mix temperature and the effect of vibration are the most important variables to be considered for wall form pressures. For walls and a concrete unit weight of 23.5 kPa, the maximum pressure for design is currently given as follows (ACI Committee 347R-94, 1994; Hurd, 1995):

a- For walls, rate of placement not exceeding 2 m/hr

$$P_{max} = 7.2 + \frac{785R}{T + 17.8} \tag{2.6}$$

(with a maximum of 95.8 kPa , a minimum of 28.74 kPa, but in no case greater than 23.5*h*)

b- For walls, rate of placement from 2 to 3 m/hr

$$P_{max} = 7.2 + \frac{1156}{T + 17.8} + \frac{244R}{T + 17.8} \tag{2.7}$$

(with a maximum of 95.8 kPa , a minimum of 28.74 kPa, but in no case greater than 23.5*h*)

c- For walls, rate of placement greater than 3 m/hr

$$P_{max} = 23.5h \tag{2.8}$$

where P_{max} = maximum lateral pressure, kPa; R = rate of placement, m/hr; T = temperature of concrete in the forms, °C; and h = maximum height of fresh concrete in the forms, m.

These formulas apply to internally vibrated concrete, made with Type I cement, containing no pozzolans or admixtures, and having a slump of not more than 100 mm. Depth of vibration is limited to 1.22 m below the top of concrete.

The Construction Industry Research Information Association, CIRIA Report 108 (Clear and Harrison, 1985) extended and improved the method used in CIRIA Report 1 (Kinnear and et al., 1965) to cover concrete using admixtures and blended cements. Based on studies conducted on concrete lateral pressure on formwork, CIRIA recommended equation for the maximum concrete pressure on formwork. This equation considered some influencing variables such as vertical form height, rate of rise, concrete temperature at placing, and the use of admixture and /or blends or blended cements. The shape of concrete pressure envelope is the same as that one described by ACI 347. The recommended formulas for maximum concrete pressure on formwork are the following:

$$P_{max} = \gamma\left[C_1\sqrt{R} + C_2K\sqrt{H - C_2\sqrt{R}}\right] \tag{2.9}$$

$$P_{max} = \gamma h \tag{2.10}$$

(whichever is the lesser)

where C_1 = coefficient (1.0 or 1.5) dependent on the size and shape of formwork (√mhr); C_2 = coefficient (0.3, 0.45, or 0.6) dependent on the constituent materials of concrete (√m); γ = weight density of concrete (kN/m^3); H = vertical form height (m); h = vertical pour height (m); K = temperature coefficient taken as $[36/(T+16)]^2$; R = rate of concrete rise (m/hr); and T = temperature of concrete at placing (°C). When $C_1R > H$, the fluid pressure (Dh) should be taken as the design pressure.

Gardner (1980) developed a design lateral pressure equation. This equation was derived from the experimental results obtained at the University of Ottawa on a large instrumented form. He concluded that the envelope of the lateral pressure exerted by fresh concrete against a vertical form face can be considered as hydrostatic from the free surface to a maximum value and thereafter constant at the maximum value. Gardner also found that the maximum lateral pressure increased with: depth of immersion of vibrator, power of vibrator, minimum dimension of form, rate of placing, slump of concrete, and decrease in concrete temperature. He compared the experimental results with the results predicted by his proposed equation and the accepted ACI and CIRIA equations. The comparison showed that the ACI equation is conservative except at low rates of placing and the minimum limiting lateral pressure is to be 47.9 kN/m^2, and CIRIA arching criterion was inappropriate for low friction forms. Gardner's equation for predicting the maximum lateral concrete pressure is:

$$P_{max} = 24h_i + \frac{3000HP}{d} + \frac{d}{40} + \frac{400R^{1/2}}{18+T} + \frac{slump - 75}{10} \tag{2.11}$$

with a limiting value of

$$P_{max} = 24h \tag{2.12}$$

where d = minimum form dimension (mm); h = total height of the form (m); h_i = immersed depth of vibrator (m); HP = horsepower of vibrator; R = rate of placing (m/hr); $slump$ (mm); and T = temperature (^{o}C).

Johnston, Kahn and Phillips (1989) conducted a field test on two tall and thick walls to determine the loads and pressures exerted by the fresh concrete on the forms and to investigate the effects of various parameters. These include such as mix design, admixtures, concrete temperatures, rates of pours, and stiffening times for concrete on determining the lateral pressures. They also compared the field results with recommended and proposed procedures such as Rodin, ACI, CIRIA and Gardner recommendations. They found that the three theories, ACI, CIRIA and Gardner agree that lateral pressure exerted by fresh concrete increases with the decrease in temperature. They found that the term in Gardner's equation for influence of wall thickness may need an upper limit, and the ACI limitation could have a P_{min} equal to 28.74kPa (600 psf).

Dunston, Johnston and McCain (1994) conducted a field test on tall test-wall constructed with concrete containing extended-set admixtures and high content of slag and fly ash to investigate the lateral concrete pressure and to evaluate the applicability of recommended procedures, ACI and CIRIA recommendations. They concluded that the filed test results supported the conservative assumption by pressure theories for design pressure envelope that follows the linear hydrostatic pressure from the top of placement to a maximum pressure and remains at that pressure at greater depths. It was found that the tributary width approach to distribute the pressure appeared reasonable for design purposes, the ACI recommendations for estimating lateral pressures were inadequate for predicting the pressures encountered, and the CIRIA recommendation were reasonable and conservative for predicting the measured values of lateral concrete pressures.

2.9 Codes and Standards

For concrete construction, many sets of requirements and standards exist varying widely from a minimal coverage in proposed procedures and local codes to a detailed coverage in national standards.

ACI Committee 347R-94 (1994), "Guide to Formwork for Concrete," provides extensive and detailed recommended practices for formwork. This document covers preparation of contract plans and specifications, design criteria for vertical and horizontal forces and lateral pressures, design considerations including capacities of formwork accessories, preparation of formwork design drawings, construction and use of forms including safety consideration, materials for formwork, formwork for special structures, and formwork for special methods of construction.

Chapter 6 of ACI 318 (1989), "Building Code Requirements for Reinforced Concrete," deals with construction practices and provides general guidelines for design of formwork, removal of forms and shores, and placing and curing of concrete. However, no quantitative values are specified for any of these guidelines.

ANSI Committee A10.9 (1983), the American National Standards Institute "American National Standard for Construction and Demolition Operations-Concrete and Masonry Work-Safety Requirements," provides both quantitative and qualitative provisions for the design of formwork. Many of the detailed provisions of this document are based on ACI 347.

OSHA Subpart Q (Hurd, 1988) of the Federal Construction Safety and Health Regulations is the most significant concrete construction safety standard and applies to most concrete construction. The OSHA Standard includes all the provisions of ANSI Committee A10.9 (1983). This document is mandatory and all provisions must be followed in formwork operation.

The Canadian Standards Association (CSA): Falsework for Construction Purposes CAN3-S269.1-1975 (1998) provides rules and requirements for the design, fabrication, erection, inspection, testing, and maintenance for falsework. Formwork CAN3-269.3-

M92 (1998) provides rules and requirements for the design, fabrication, erection, and use of concrete formwork. Concrete Materials and Methods of Concrete Construction CAN-A23.1-M90 (1990) covers materials to be used and methods to be followed for proportioning, manufacturing, transporting, placing, finishing, curing, protecting, and accepting of concrete, together with the requirements for the forming and setting of reinforcements, when the placing and subsequent operations are performed in the field, including field pre-cast operations.

The British Code of Practice for Falsework BS5975 (1982) content is organized under the general heading procedures, materials and components, loads, foundations and ground conditions, design, and construction. However, it also contains a considerable amount of in-depth commentary and several detailed appendixes of properties of some components.

The Standard Association of Australia: Formwork for Concrete AS3610 (1990). This standard with its commentary present design and construction requirements for falsework and formwork of all structure types.

Falsework for construction projects in New Zealand is regulated by the Code of Practice for Falsework (1988). Volume 1 contains the code and appendixes and Volume 2 serves as commentary. The content of this code is organized under the general headings of procedures, materials, foundation and soil conditions, loads, design, and construction. The New Zealand code also contains several detailed appendixes, which include scaffold tube falsework, proprietary components, foundation investigation, and lateral concrete pressure on forms. The latter appendix reviews both the ACI and Construction Industry Research and Information Association (CIRIA) equations for lateral concrete pressure.

In addition to these standards, there are other documents, which provide design guidelines, operation and specification for formwork. These documents are guidelines prepared by contractors and formwork manufactures' associations, and manuals prepared by owners and certain construction agencies. Moreover, there are some recommended and proposed recommendations in certain subject areas of formwork.

There is a considerable variation in the requirements for design loads and in the safety factors required for construction among the existing standards, guidelines and manuals for design of formwork. Presently, no codes and standards provide a uniform approach to the design of the partially completed slab/shore system as consistent system with the design standard and procedures used for permanent structures (Mosallam and Chen, 1990).

CHAPTER III

MODEL IDEALIZATION

3.1 General

A model is a representation of real life system or process by some mathematical or numerical expression, which can be used as a substitute for the real thing, and allows to predict what would happen in a real system by changing the input data parameters of the model (Hanna, 1989). It is important to select a model that matches as closely as possible the behavior of the real system. No matter how precise the calculations involved in the analysis, they can never correct any possible errors, which result from an inaccurate idealization of the actual structure or process (Stivaros, 1988).

This chapter explains the basic concepts of the analyses and describes the idealization of both the shoring system and wall formwork models.

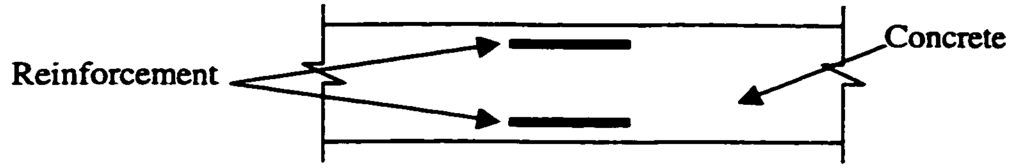
3.2 Adopted Software System

The computer program, CPF (Cracked Plane Frames in Prestressed Concrete) developed by Elbadry and Ghali (1990) to perform the analysis necessary in the design for serviceability of reinforced concrete plane frames with or without prestressing is employed for the purpose of the present research. The analysis by CPF gives the instantaneous and time-dependent changes in the displacements, in the support reactions and in the statically indeterminate internal forces, along with the corresponding changes in stress and strain in individual sections. It takes into consideration the effects of creep and shrinkage of concrete, relaxation of prestressed steel, and cracking of concrete on the deflection, reactions and internal forces in statically indeterminate structures. Herein, a brief description of the method of analysis by CPF is given. The theory behind the computer program CPF, and more details on its use and application can be found in Elbadry and Ghali (1989).

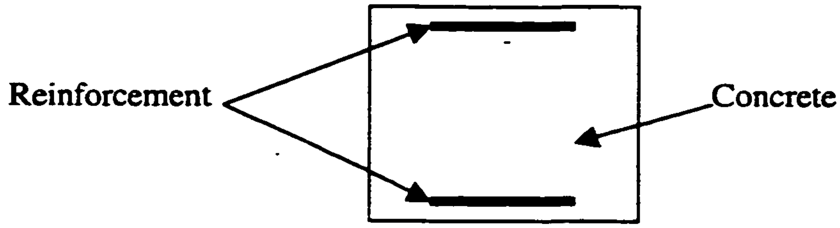
A member cross-section can consist of one or more parts (Fig. 3.1). The steel layer (reinforcement) is defined by its area and its depth from the top fibers. The length of steel layer can cover a part or full length of member. Thus, the number of steel layers can be differing from one section to another.

External loads can be in the form of forces or couples applied at the nodes, concentrated loads acting at any point on the member axis, and a distributed load of any variation covering a part or the full length of the member.

The time period for which the analysis is required is divided into n intervals (Fig. 3.2). The instants t_1, t_2,..., t_{n+1} coincide with an event: casting of new concrete members or parts of these members, application of loads, and introduction or removal of shores or reshores. An instant t_i can simply be a time at which the stresses or the deformation are required. At any instant t_i, the instantaneous changes in stresses and deformations are calculated and added to the existing values. Also for any time interval, i, the gradual change in stresses and deformations taking place between t_i and t_{i+1} are determined and added to update the existing values.

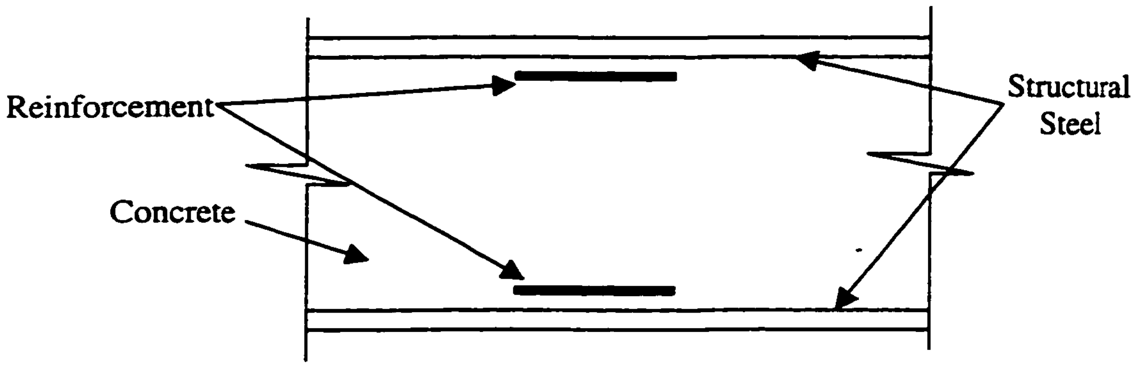


(a) Cross-Section of Slab



(b) Cross-Section of Column

(c) Cross-Section of Shore



(d) Cross-Section of Wall

Fig. 3.1 Typical Cross Sections Treated in the Present Analysis

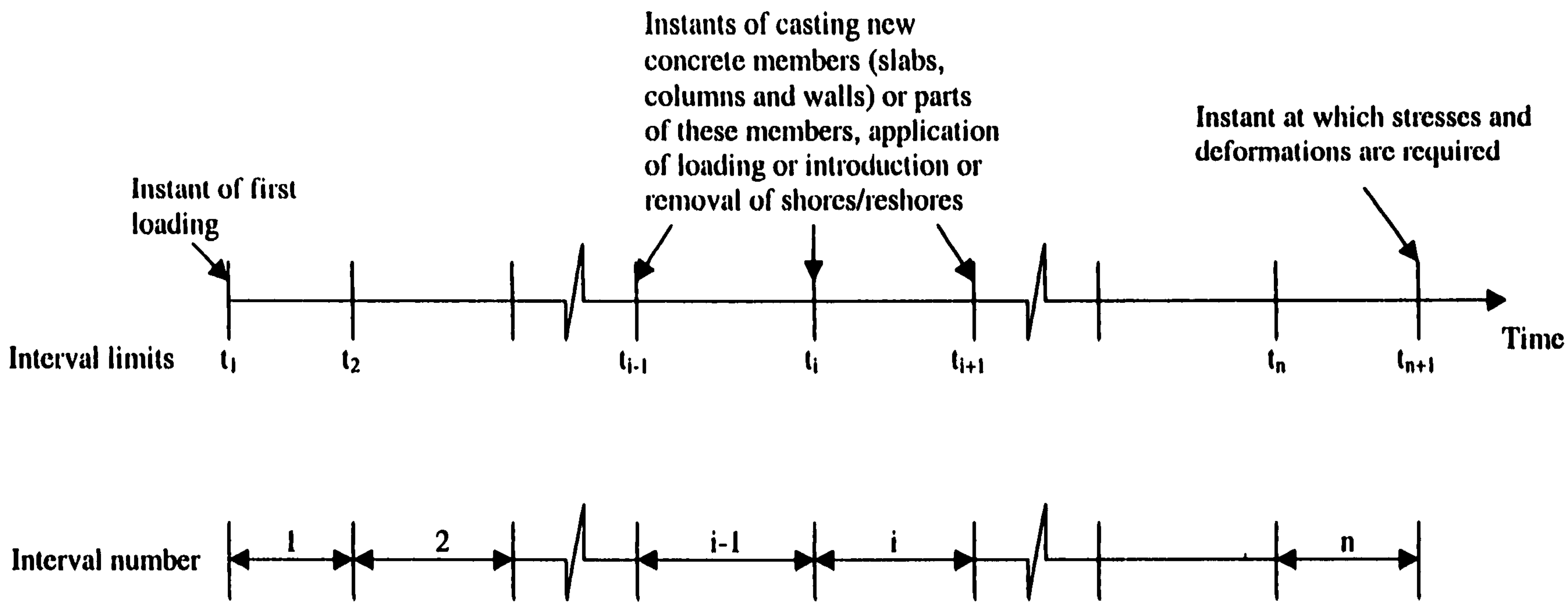


Fig. 3.2 Division of Time into Intervals

The material properties that affect the behavior of concrete structures under service conditions are: the compressive strength, the modulus of elasticity, creep, shrinkage and the tensile strength of concrete and the modulus of elasticity of steel. The program CPF uses the ACI Committee 209 (1982) and the CEB-FIP Model Code (1978) for predicting the variation over time of concrete properties from data related to variables such as mix composition, the member size and the environmental conditions. In addition to the above methods, the program CPF allows the user to provide the values of concrete properties as obtained from experimental data or according to the method of the user's choice. Here, for the purpose of this study, the maturity-based model proposed by Gross and Lew (1986) for determining the development of concrete strength is adopted.

The time-dependent analysis of stresses and strains as performed by CPF utilizes the age-adjusted effective modulus method and, therefore, the aging coefficient introduced by Bazant (1972) is required for the analysis. This coefficient can either be calculated by the program or provided by the user as input data. An approximate value of this coefficient can be taken as 0.8.

3.3 Structure Idealization

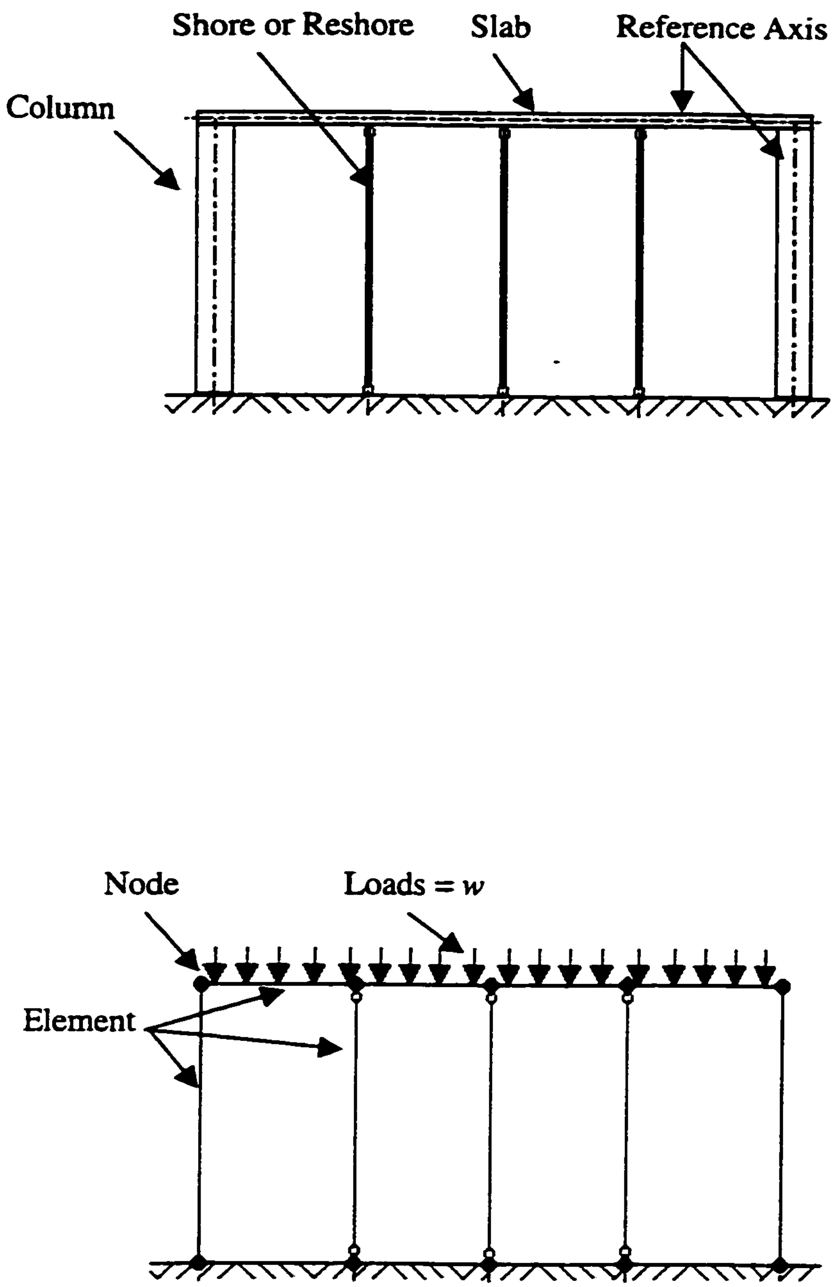
Idealization of the structure has been carried out in full compliance with the requirement of the CPF program. This analysis is based on the displacement method in which a plane frame is idealized as an assemblage of straight beam elements connected at the joints (nodes) as shown in Figs. 3.3(b) and 3.4(c). Loads can be applied either at nodes or along the length of the structure elements.

An individual element may have a constant or a variable cross section (Fig. 3.5) over its length, but the cross section must have one of its principal axes in the plane of the frame. Each member in the frame is represented by a reference straight axis, chosen at an arbitrary constant depth from the top of the member (Fig. 3.3a and Fig. 3.4b). The nodes of the frame are located at intersections of the reference axes of individual members (Fig. 3.3b and Fig. 3.4c).

Each node has three degrees of freedom: two translations and one rotation, which are defined with respect to an arbitrarily chosen global (structure) system of axes, x, y and z (Fig. 3.4a). The local system of axes, x^*, y^* and z^*, for each member is defined as shown in Fig. 3.4b. The x^* axis coincides with the reference axis of the member and is directed from node O_1 to node O_2. The y^* and z^*-axes are mutually perpendicular to the x^*-axis.

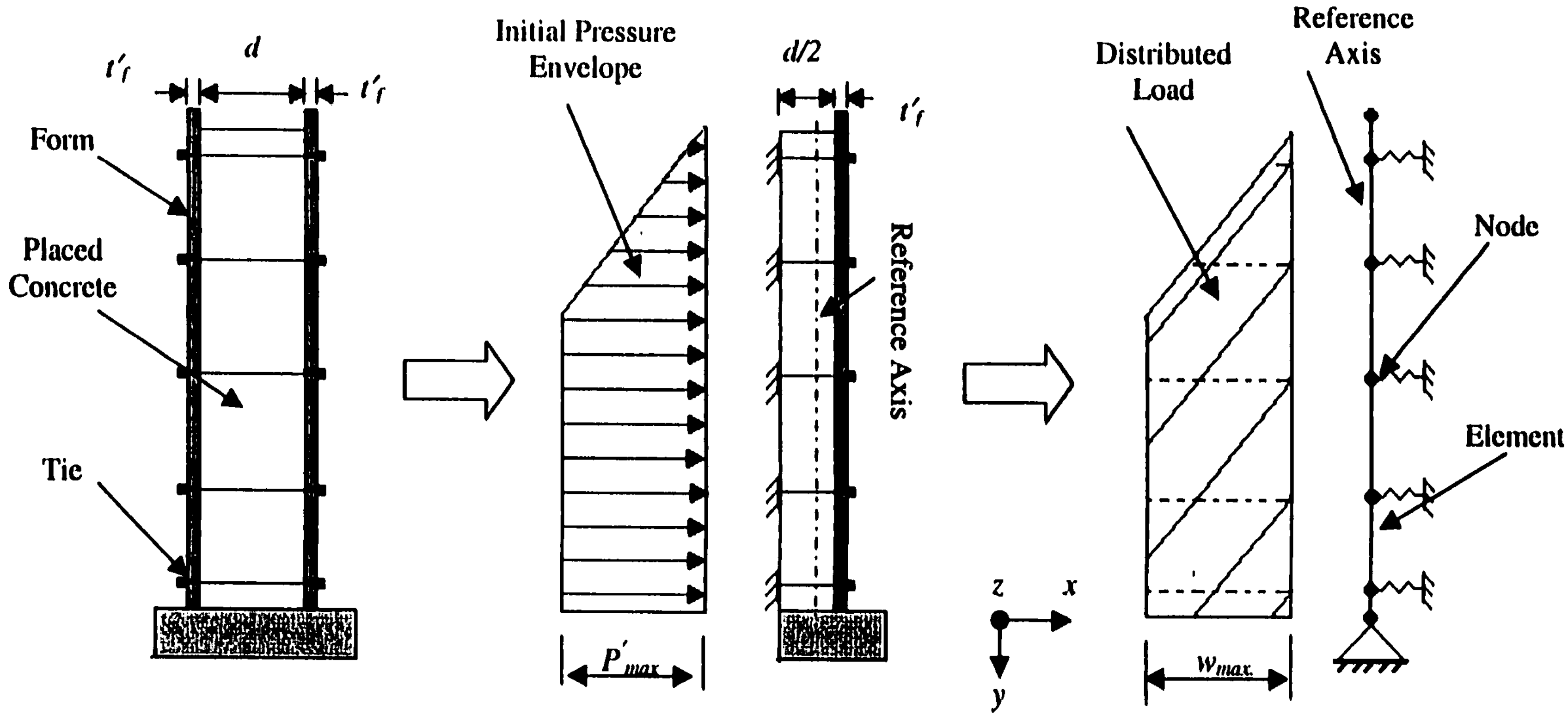
3.4 Shoring System

In multistory cast-in-place concrete construction, the system of shores and reshores supports freshly placed concrete floor. This system transfers the construction loads to the floors below. Therefore, shores or reshores must be provided for enough floors to allow the placed concrete to develop its needed capacity. The number of used shores or reshores levels depend on the job plans for reuse of materials as well as the rate of strength gain in the structure.



(b) Analytical model

Fig. 3.3 Concrete Building during Construction



(a) Typical Structure (b) Structure Representation (c) Analytical Model

Fig. 3.4 Concrete Wall during Concrete Placing

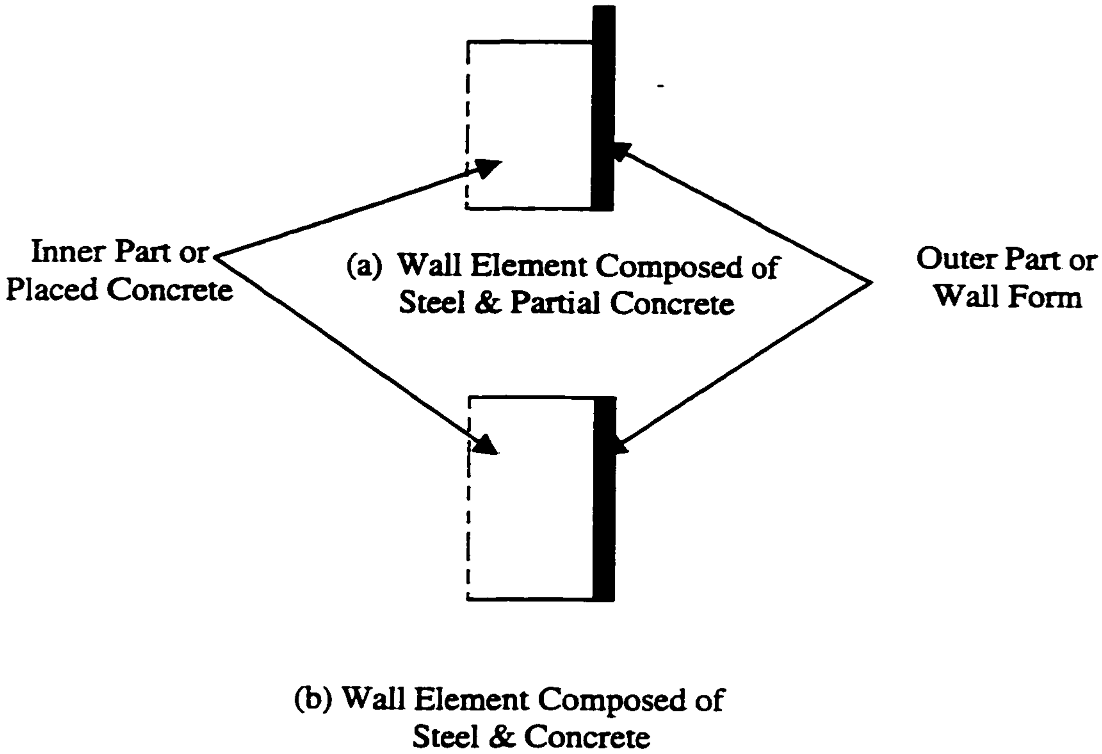
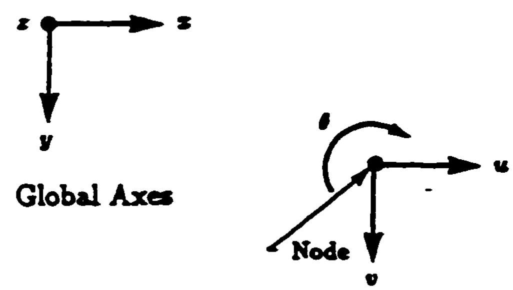
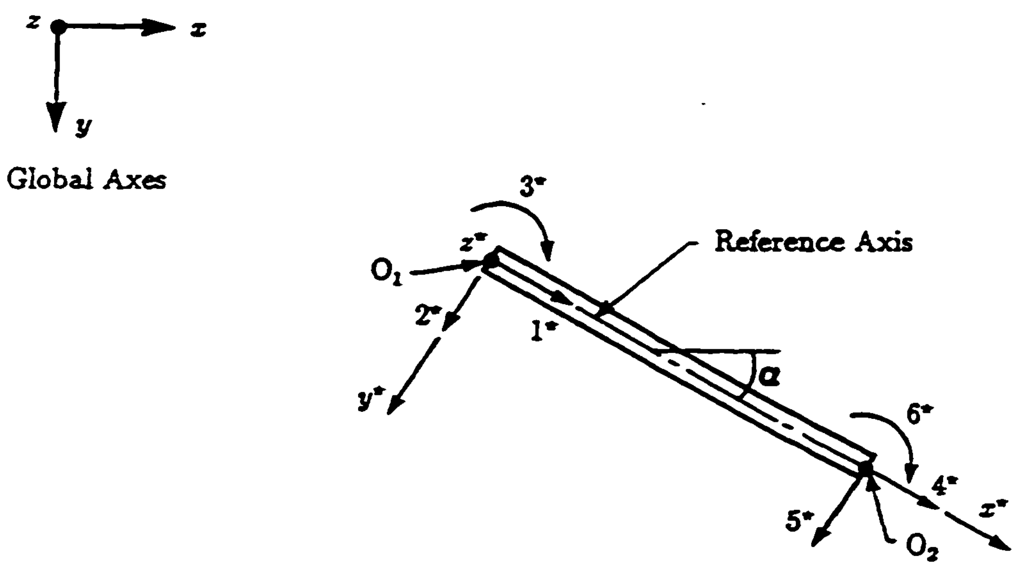


Fig. 3.5 Element Parts of Composite Wall Structure



(a) Displacement Components at a Typical Node.



(b) Local Axes and Positive Directions of Member End Forces and Displacements.

Fig. 3.6 Coordinate Systems for Plane Frame Analysis (Elbadry and Ghali, 1990)

At any given time (stage), the distribution of construction loads carried by shores and interconnected slabs are computed. The amount of load carried by slab is equal to the calculated shear force of the slab-beam member transferred to the supported columns. The load carried by each individual shore or reshore member supporting that slab is equal to the calculated axial compressive force in that shore or reshore. The maximum calculated axial force of shores and reshores are only taken into account for safe design and construction purpose. This procedure is repeated for all steps until all shores/reshores and interconnected slabs loads are determined.

For comparison purposes in some cases, the maximum calculated slab moments are taken into consideration to determine the slab loads. In these cases the slab moments are normalized with respect to the same slab subjected to a uniformly distributed dead load at the corresponding locations (Mosallam and Chen, 1992).

3.4.1 Structural Model Assumptions and Limitations

1. The slabs and columns behave elastically and their stiffnesses are time-dependent.
2. The shores and reshores act as axially loaded members with specified axial stiffness.
3. All joints between the shores or reshores and the slabs are pin-ended.
4. Shrinkage and creep of concrete are considered
5. The ground support is rigid.
6. The rate of construction is taken as one floor per week.

3.4.2 Model Description

Figure 3.3 depicts the 2-D structural model used to analysis the concrete slab building during construction. The column base is fixed and any joint between the column and slab strip can translate in x-direction and y-direction, and rotate about z-axis according to its stiffness. The connections between the shores or reshores and the slabs or the base are pin-ended. Therefore, the shores and reshores are treated as vertical truss members.

3.4.3 Construction Process

A typical construction cycle for a shoring scheme using both shores and reshores as described by Gross and Lew (1984) is performed in four repeated basic operations:

1. Install a story of shores and forms and place the fresh concrete.
2. Remove the reshores from the lowest reshored level.
3. Remove the forms and shores from the lowest shored level.
4. Place the reshores under the slab, which forms and shores were just removed.

According to the ACI Standard 347-94R (1994), the reshores should be installed snugly under the slab just stripped. Therefore, the reshores carry no load when they are first placed. The above four operations are used for shoring system using shores and reshores. However, in the case of using shores only, the second and fourth operations are eliminated.

There are numerous construction procedures for multistory concrete buildings; two of them are most commonly used (Mosallam and Chen, 1991). Both essentially repeat the above four basic operations, but in different sequence. In the first procedure, the freshly placed concrete is always supported by two levels of shores and one level of reshores. The second is similar to the first except the use of shores rather than reshores in the lowest level. Mosallam and Chen (1991) summarized these procedures in ten steps, as follows:

A. Two levels of shores and one level of reshores

Step1: Place the first level of concrete slab.

Step 2: Place the second level of concrete slab.

Step 3: Remove the shores under first level of concrete slab.

Step 4: Place the reshores under first level of concrete slab.

Step 5: Place the third level of concrete slab.

Step 6: Remove the reshores under first level of concrete slab.

Step 7: Remove the shores under second level of concrete slab.

Step 8: Place the reshores under second level of concrete slab.

Step 9: Place the fourth level of concrete slab.

Step 10: Remove the reshores under second level of concrete slab.

B. Three Levels of Shores

Step1: Place the first level of concrete slab.

Step 2: Place the second level of concrete slab.

Step 3: Place the third level of concrete slab.

Step 4: Remove the shores under first level of concrete slab.

Step 5: Place the forth level of concrete slab.

Step 6: Remove the shores under second level of concrete slab.

Step 7: Place the fifth level of concrete slab.

Step 8: Remove the shores under third level of concrete slab.

Step 9: Place the sixth level of concrete slab.

Step 10: Remove the shores under fourth level of concrete slab.

Figures 3.9 and 3.10 illustrate the procedure described above in the two cases. Steps 5 to 8 of construction procedure consisting of two levels of shores and one level of reshores has been chosen to illustrate the four basic construction operations. The configuration of Step 5 is shown in Figure 3.9a, in which level 3 is being placed. The construction load is distributed among the interconnected slabs through the shoring system. Since the concrete has no strength initially, a small value for the modulus of elasticity of the newly placed concrete slab ($E_c = 1.0 \times 10^5$ N/m^2) is assumed in the analysis.

The removal of reshores under level 1, Step 6, is shown in Fig. 3.9b. This removal is equivalent to applying a load to the first level slab, in a downward direction, which is equal to the compressive force in the reshores prior to their removal. Figure 3.9c shows the removal of shores under level 2, Step 7. The equivalent compressive forces in those shores prior to their removal are applied to the interconnected slabs above and below the vacated story. Installation of the reshores under level 2, Step 8, is shown in Fig. 3.9d. In this operation the reshores are snugly placed with no structural disturbance to the system.

3.5 Wall Formwork

Concrete is generally placed into wall and column forms in lifts, which are vibrated to

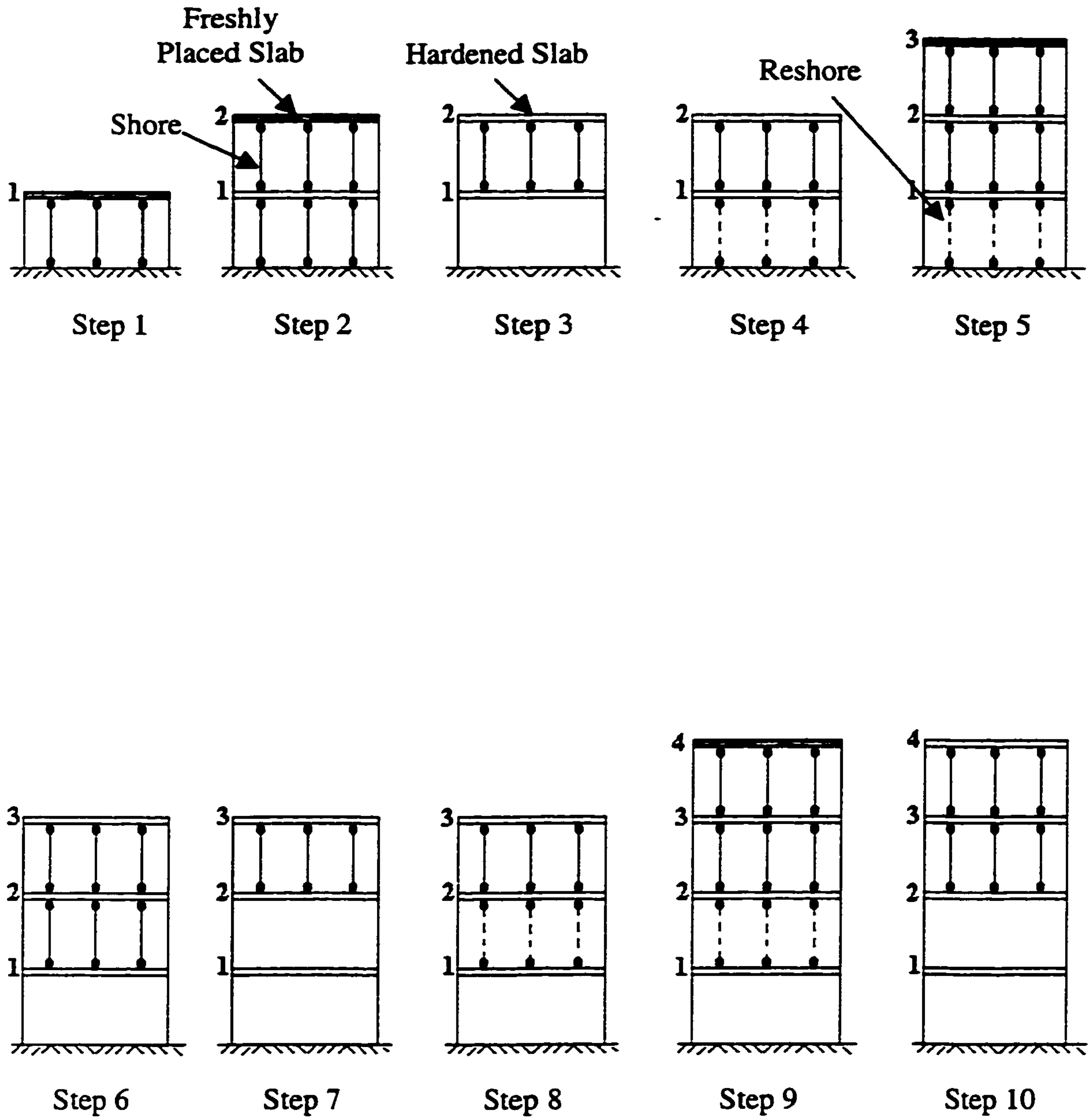


Fig. 3.7 Construction Procedure for Two Levels of Shores

and One Level of Reshores

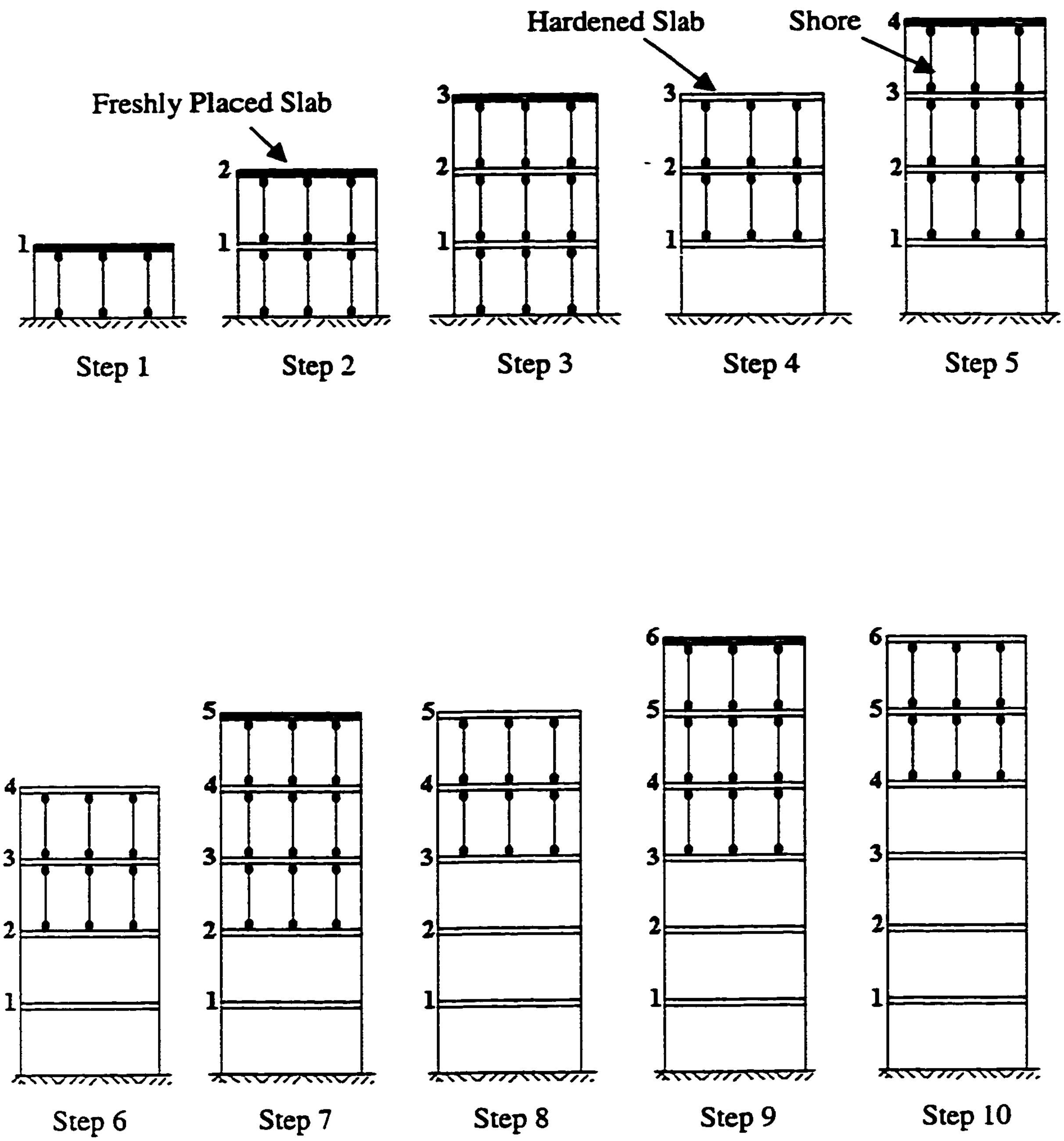
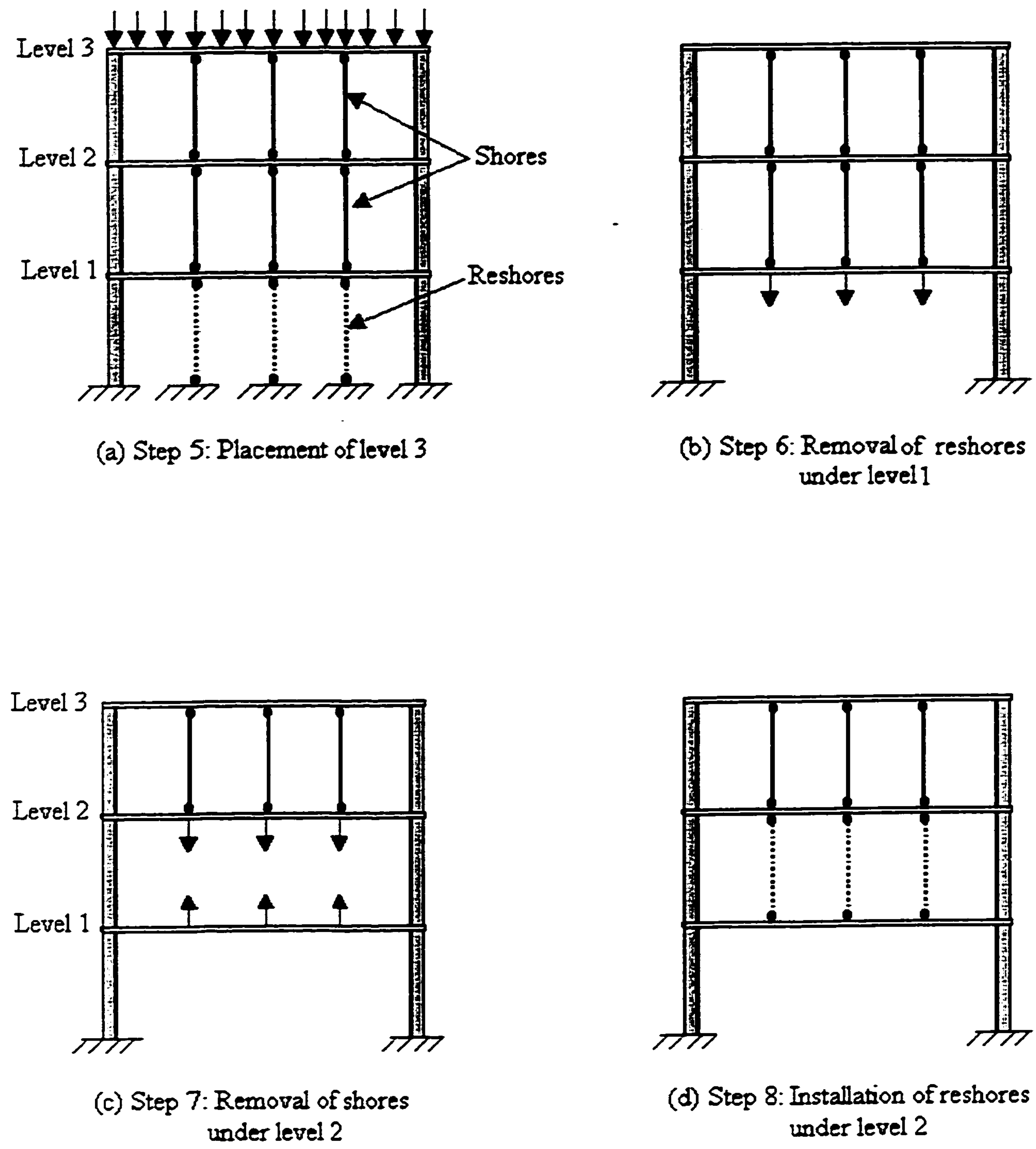


Fig. 3.8 Construction Procedure for Three Levels of Shores



(a) Step 5: Placement of level 3

(b) Step 6: Removal of reshores under level 1

(c) Step 7: Removal of shores under level 2

(d) Step 8: Installation of reshores under level 2

Fig. 3.9 Construction Processes of Concrete Building

consolidate the concrete. The purpose of vibration is to fluidize the concrete, destroying its shear strength capability and any friction between the concrete and the form (Gardner, 1980). The ability to change from a semi-liquid or plastic to a solid state appears to be the result of two actions within the fresh concrete. The first action is the result of the setting of the cement paste, which can take a few minutes to a few hours according to concrete properties and conditions. The second action is the development of internal friction between the particles of aggregate in the concrete, which restrain them from moving freely past each other. The magnitude of that friction increases with the loss of water from the concrete (Peurifoy and Oberlender, 1996).

This change in structure of concrete is important. While the aggregates and the cement are suspended in water, the concrete exerts a fluid pressure on the formwork, but once a stable particle structure has been formed, further increments in the height of placed concrete would have an insignificant effect on lateral pressure. Therefore the maximum lateral pressure is generally below the fluid head, and it is controlled by this change of structure (Clear and Harrison, 1985). The degree of accuracy for estimating the lateral concrete pressures is as much as the analytical model has a close matching to the real fresh concrete behavior, the formwork characteristics and the construction process.

The effective pressure envelope is a result of two main processes that act against each other. The initial pressure envelope represents the first process, which is a function of the behavior, characteristics and placing rate of fresh concrete. The second process is the restrain of the lateral forces that is a function of the rate of concrete shear strength development and the characteristics of form elements. Loads carried by the form ties are determined by using the computer program described in Section 3.2, and then, the effective pressure is calculated by dividing the forces in the ties by their tributary areas.

3.5.1 Structural Model Assumptions and Limitations

1- In the analysis, the wall form and the placed concrete are treated as composite steel and concrete structure, consisting of three parts: the two outer parts represent the equivalent wall form and the inner part represents the placed concrete (Fig. 3.5).

2- The two outer parts (wall form) and elastic supports (ties) are considered to be linearly elastic.

3- The inner part of the wall assemblies (placed concrete) is assumed to behave elastically and its stiffness to be time-dependent.

4- As a result of internal vibration, the inner part of each wall element (cast concrete) has the same concrete properties.

5- The initial pressure envelope is to be taken as hydrostatic from the top of placement up to the depth where the maximum initial pressure (P'_{max}) occurs and then remains constant at that value.

$$P'_{max} = \gamma h'_{max} \tag{3.1}$$

where, P'_{max} is the maximum initial pressure (N/m^2), γ is the fresh concrete density (N/m^3) and h'_{max} is the head at which maximum initial pressure occurs (m).

$$h'_{max} = 2.13C(R)^{1/3} \tag{3.2}$$

where, R is the rate of pour (m/hr) and C is a coefficient dependent on fresh concrete properties and is assumed to be 1 for purpose of this study.

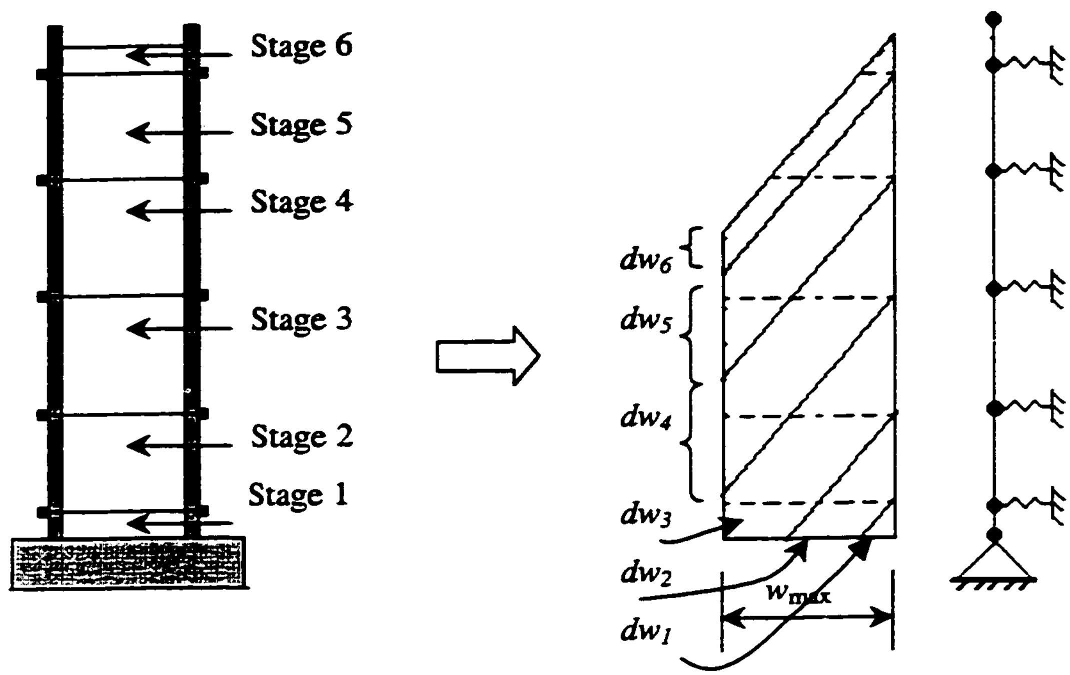
3.5.2 Model Description

Figure 3.4 shows the basic model, which idealizes the fresh constructed concrete wall. Because of symmetry of the wall and to simplify the analysis, half section of the wall is considered in the analytical model. The model base sits on a hinge to meet the structure conditions such as the discontinuity of the wall form and the plasticity of the fresh cast concrete. The base is free to rotate about the z-axis. The form ties are represented by equivalent elastic supports. These supports can freely translate in the y-direction and freely rotate about the z-axis. They can translate in the x-direction according to their stiffness. The free end of the wall is free to translate in both the x and y-directions and to rotate about z-axis.

3.5.3 Construction Process

In practice, the wall forms are firstly installed and then the concrete is placed into wall forms in lifts, which are vibrated to consolidate the concrete. The inner part (cast concrete) of each wall element is assumed to be placed at the same time and considered to be homogenous over the entire length (Fig. 3.10a).

Fresh concrete imposes loads on the fresh composite wall structure in form of initial pressure envelope. The lateral load acting on wall model equals to the initial pressure at that time times the wall strip width, b. Fig. 3.10b shows the distributed load at last stage.



(a) Placing Concrete of The Last Wall-Element (Last Stage)

(b) Distributed Load at Last Stage

where dw_1, dw_2, dw_3, dw_4, dw_5 and dw_6 are load increments at stages 1, 2, 3, 4, 5 and 6 respectively, $w_{max} = P'_{max}\, b$, and b is the width of wall strip considered in the analysis.

Fig. 3.10 Construction Processes of Concrete Wall

$$w = P' b \tag{3.3}$$

where, w is the distributed load at any point due to initial pressure (N/m), P' is the initial pressure at that point (N/m^2) and b is the wall strip width (m).

3.5.4 Properties of Wall-Element Parts

The CPF program has the ability to generate time-dependent properties of concrete, as well as to allow the user to provide the concrete properties as input data. Here, in this model, the modulus of elasticity of the wall parts is only provided as input data. The modulus of elasticity is constant for the wall form (outer parts), and time-dependent for the cast concrete (inner part). In multistage construction, the modulus of elasticity of concrete differs according to the age, and can be estimated by using the maturity-based-model, described earlier in Chapter 2 (Section 2.5). This allows for the time-dependent modulus of elasticity of the placed concrete to be expressed as $E_c = 5 \times 10^9 (f_c')^{1/2}$ (ACI 318-89M) where E_c is modulus of elasticity of fresh concrete (N/m^2) and f_c' is the compressive strength at early age (MPa).

Since the outer part of the wall has the equivalent bending stiffness and the same width of the real wall form, the equivalent form thickness, d', can be calculated as follows:

$$E_f' I_f' = E_f I_f \tag{3.3}$$

where E_f', I_f', E_f and I_f are the modulus of elasticity and moment of inertia of the modeled and real form respectively.

$$d' = (12 E_f I_f / b E_f')^{1/3} \tag{3.4}$$

Form ties are represented by elastic supports that have equivalent axial stiffness of the form ties.

$$k_t = E_t A_t / L_t \tag{3.5}$$

where k_t is the axial stiffness (N/m), E_t is the modulus of elasticity (N/m^2), A_t is the cross-section area (m^2) and L_t is the length of tie (m).

3.6 Input Data

The format used to read the input data is free-field format. The examples for input data files that used for both shoring system and wall formwork analyses are shown in Appendixes A and B, respectively. More details about the input data procedures can be found in Elbadry and Ghali (1990). The data is divided into sets as follow:

- Control Parameters

  These are a list of keywords that represent the names of integer or real variables.

- Material Properties

  In this set, the constant properties of concrete and steel are only considered as input data.

- Coordinates of Nodes

  In this set, every node is represented by its global x and y-coordinates.

- Topology of Members and Section Dimensions

  The data in this set defines the topology of each member and gives the dimensions or the area properties of the concrete parts in various sections and the layout and properties of nonprestressed steel layers in the member.

- Time-Dependent Parameters

  This set gives data for the introduction or removal of shores/reshores, for creep and shrinkage properties and for external loads applied at any instant.

3.7 Output Data

The output of the program CPF can be stated as the following:

- All input data
- Nodal forces
- Increments of reaction and total reactions
- Increments of displacements and total displacements
- Increments of member end forces ant total member end forces
- Updated deflections
- Total internal forces, strains and stresses

For the purposes of this study, some of the above output data are required. The total member end forces are utilized by the shoring system model, while the total reactions are utilized by the wall formwork model.

CHAPTER IV

MODEL VERIFICATION

4.1 General

In this chapter, the present models are checked against field measurements and other methods, in current use, to verify the present method and to investigate the range of applicability of the others.

4.2 Shoring System Model

Shoring system model is checked against the field measurements (Agarwal and Gardner, 1974), the simplified method (Grundy and Kabaila, 1963), the refined method (Liu, et al., 1986), the improved simplified method (Duan and Chen, 1995a) and the 2-D and 3-D methods advanced by Mosallam and Chen (1991 and 1992). The following three examples are considered.

Example 1

Shore load measurements at 27-Story Flat Slab Office Building, Quebec, Canada were reported by Agarwal and Gardner (1974). The slab thickness for this building was 254mm (10in.) up to the 14^{th} floor and 203mm (8in.) above the 14^{th} floor. The construction rate was approximately one floor per week with three levels of steel shores. A typical floor area of 7.62 x 7.62m (25 x 25ft.) was selected for instrumentation (Fig. 4.1). The field measurements of nine shores were made from level 19 to level 22 for a complete cycle of construction.

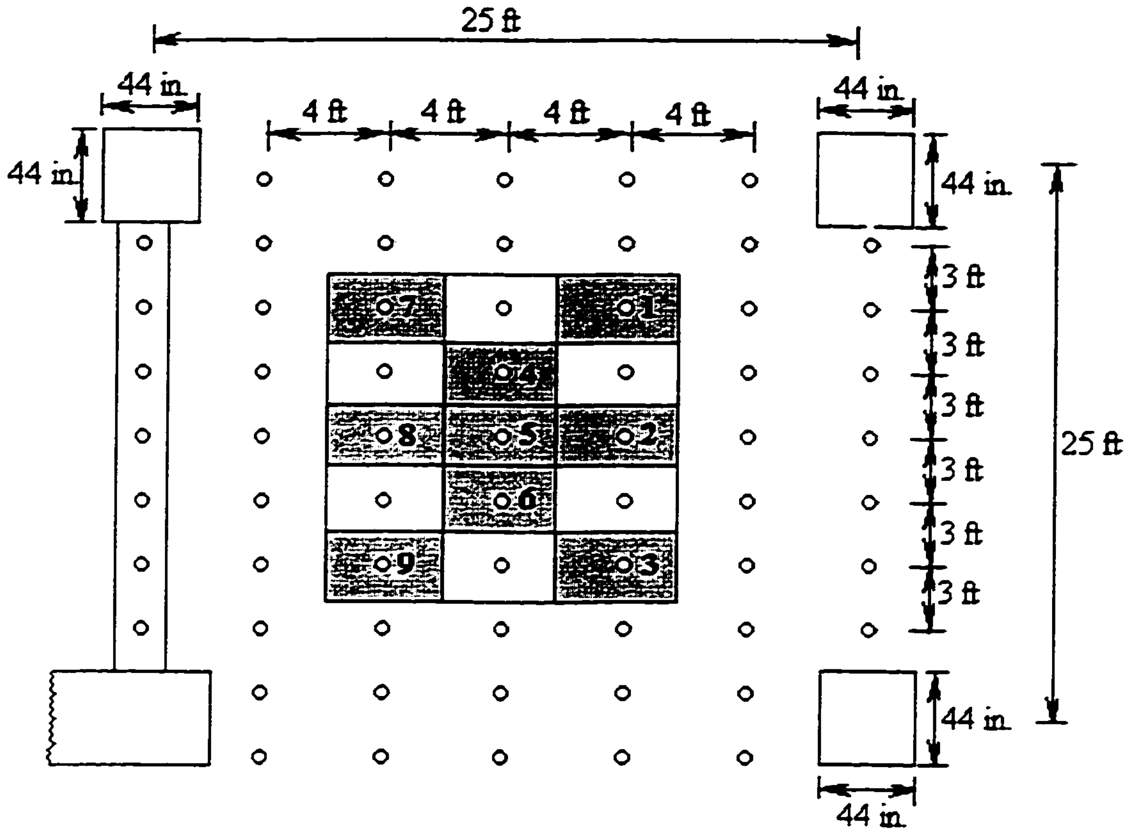


Fig. 4.1 Typical Plan of Shoring Arrangement - Place Du Portage Project (Agarwal and Gardner, 1974)

Table 4.1 shows the results of the present method at the beginning and end of each construction operation. The comparison between field measurements and simplified method, improved simplified method and present method is given in Table 4.2. The loading history for 19^{th} level slab using the above methods is shown in Fig. 4.2. The results indicate that the present method is in agreement with the field measurements and the improved simplified method, particularly on the location and time of the maximum shore load. The field measurements, the improved simplified method and the present method show that the absolute maximum shore load occurs during the concrete placing of the 20^{th} level at the19^{th} level, while the simplified method shows that the absolute maximum shore load occurs during placing the 21^{st} level at the 20^{th} level. The present method overestimates the absolute maximum shore load by 20% while the simplified and improved simplified methods have slight difference. For slab loads, the present method agrees with the field measurements and the simplified method on the magnitude and the location and time of occurrence of the absolute maximum slab load.

Example 2

Figure 4.3 shows the 2-D structural model used by Liu, et.al (1986). The nominal slab-strip cross section is 900 x 180mm (35.4 x 7.1in.), the nominal cross section of columns is 500 x 500mm (19.7 x 19.7in.) and the nominal cross section of the wooden shores is 50 x 100mm (2 x 4in.). The construction load is expressed in term of D, the weight of a freshly placed concrete slab for the tributary area of a given shore. The sustained live load is taken as a total of 0.15D, 0.1D accounting for the weight of forms and shores and 0.05D accounting for the weight of reshores.

Table 4.1 Construction Load Distribution for Example 1 of Shoring System (Present Method)

Operations	Level	F_{max} (*D*)		*V* (*D*)	
		at beginning of operation	at end of operation	at beginning of operation	at end of operation
Placing 19th level	19	1.00	1.00	0.07	0.03
Removal 17th level	19	0.85	1.18	0.16	0.09
Placing 20th level	20 19	1.00 1.55	1.00 1.78	0.07 0.47	0.03 0.42
Removal 18th level	20 19	0.74 1.22	0.93 1.35	0.30 0.82	0.25 0.85
Placing 21st level	21 20 19	1.00 1.35 1.50	1.00 1.34 1.53	0.07 0.61 1.13	0.07 0.60 1.13
Removal 19th level	21 20 19	0.75 1.01 -	0.96 1.16 -	0.29 0.93 1.78	0.23 0.94 1.82
Placing 22nd level	22 21 20 19	1.00 1.37 1.32 -	1.00 1.37 1.32 -	0.07 0.57 1.25 2.11	0.07 0.57 1.25 2.11

D = slab weight + formwork weight for the tributary area of a given shore
F = shore load
V = slab load (shear force)

Table 4.2 Comparison of Simplified, Improved Simplified and Present Methods with Field Measurements

Operations	Level	Measured		Simplified Method		*Improved Simplified Method		Present Method		ComFarison					
		1**	2	3	4	5**	6	7	8	9	10	11	12	13	14
		F_{ave} (D)	V_{ave} (D)	F_{max} (D)	V_{max} (D)	F_{ave} (D)	V_{ave} (D)	F_{max} (D)	V_{max} (D)	3/1	4/2	5/1	6/2	7/1	8/2
Placing 19th level	19	0.99	0.00	1.00	0.00	1.00	0.00	1.00	0.07	1.01	-	1.01	-	1.01	-
Removal 17th level	19	0.83	0.17	0.73	0.27	0.79	0.21	1.18	0.16	0.88	1.59	0.95	1.24	1.42	0.94
Placing 20th level	20	1.01	0.00	1.00	0.00	1.00	0.00	1.00	0.07	0.99	-	0.99	-	0.99	-
	19	1.48	0.53	1.41	0.59	1.45	0.54	1.78	0.47	0.95	1.11	0.98	1.02	1.20	0.89
Removal 18th level	20	0.79	0.21	0.73	0.27	0.79	0.21	0.93	0.30	0.92	1.29	1.00	1.00	1.18	1.43
	19	0.88	0.91	0.75	0.98	0.87	0.90	1.35	0.83	0.85	1.08	0.99	0.99	1.53	0.91
Placing 19th level	21	1.02	0.00	1.00	0.00	1.00	0.00	1.00	0.07	0.98	-	0.98	-	0.98	-
	20	1.44	0.58	1.44	0.56	1.45	0.54	1.35	0.61	1.00	0.97	1.01	0.93	0.94	1.05
	19	1.12	1.32	1.11	1.33	1.19	1.23	1.53	1.13	0.99	1.01	1.06	0.93	1.37	0.86
Removal 19th level	21	0.77	0.23	0.70	0.30	0.78	0.22	0.95	0.29	0.91	1.30	1.01	0.96	1.23	1.26
	20	0.85	0.92	0.75	0.95	0.83	0.93	1.16	0.94	0.88	1.03	0.98	1.01	1.36	1.02
	19	-	1.85	-	1.75	-	1.81	-	1.82	-	0.95	-	0.98	-	0.98
Placing 22nd level	22	1.02	0.00	1.00	0.00	1.00	0.00	1.00	0.07	0.98	-	0.98	-	0.98	-
	21	1.45	0.57	1.41	0.59	1.44	0.55	1.37	0.57	0.97	1.04	0.99	0.96	0.94	1.00
	20	1.11	1.34	1.11	1.30	1.16	1.26	1.32	1.25	1.00	0.97	1.05	0.94	1.19	0.93
	19	-	2.11	-	2.11	-	-	-	2.11	-	1.00	-	-	-	1.00

D = slab weight + formwork for the tributary area of a given shore, F = shore load, V = slab load (shear force), * = Duan and Chen (1995b) and
** = average of 9 shores

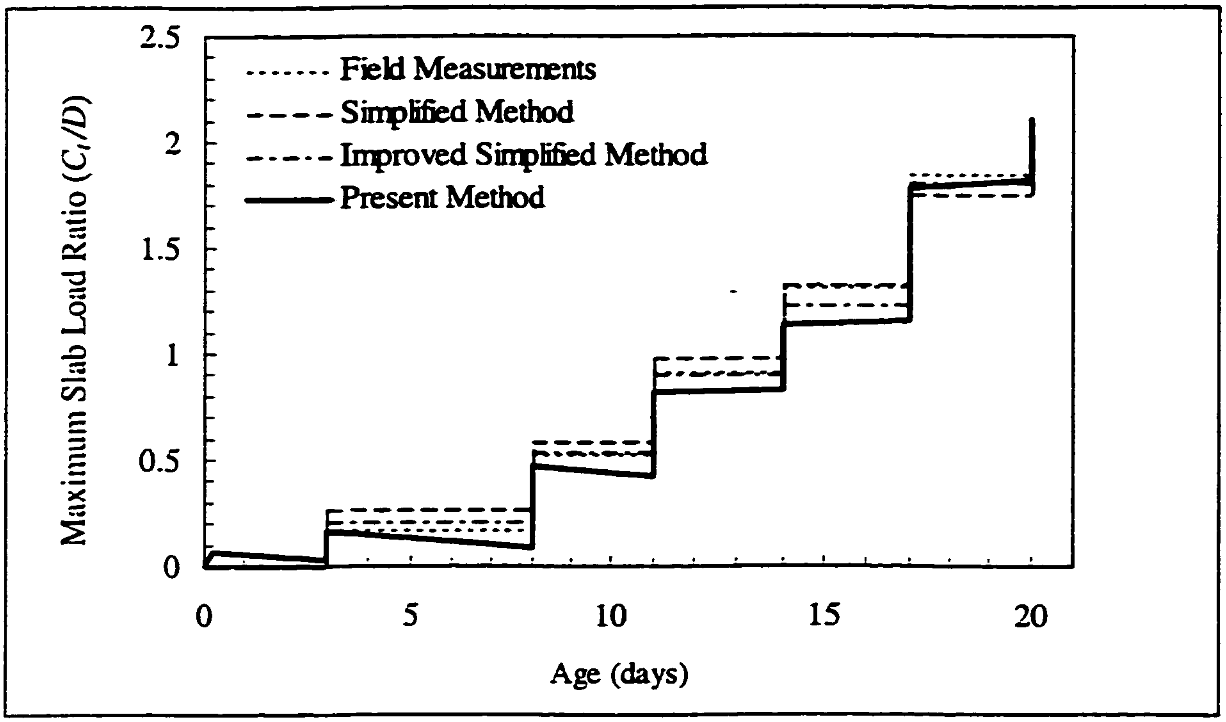


Fig. 4.2 Comparison of Loading Histories for the 19th Floor Slab During Construction (Example 1 of Shoring System)

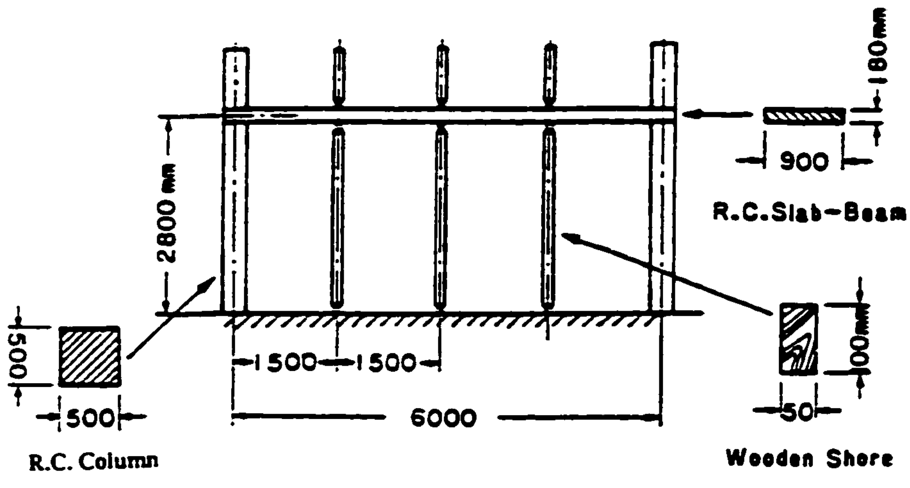


Fig. 4.3 2-D Structural Model (Liu, et al., 1986)

The extraordinary live load is taken as 0.5D. The construction rate is one floor per week with two levels of shores and one level of reshores construction procedure.

The modulus of elasticity, E_c, and the 28-day cylinder strength of concrete, $f_c^{'}$, for both the floor slab and the columns, are 3.5 x 10^4MPa (5.1 x 10^6psi) and 41MPa (6,000psi), respectively. The modulus of elasticity for wooden shores, E_w, and the compression strength of wood, f_w, are 7.75 x 10^3MPa (1.1 x 10^6psi) and 5.6MPa (810psi), respectively.

The results of present method at the beginning and end of each construction step are shown in Table 4.3. Table 4.4 shows a comparison between the present method and both the simplified and the refined methods. The maximum slab loads calculated by the above methods, using shear force, for ten construction steps are shown in Fig. 4.4. All of the three methods agree on the location and time of the absolute maximum shore and slab loads. The simplified method, however, underestimates the absolute maximum shore load by 7% and has a slight difference in the absolute maximum slab load with respect to the present method. The refined method also underestimates the absolute maximum shore load by 15% and overestimates the absolute maximum slab load by 7% with respect to the present method.

Table 4.5 shows a comparison between the present method and both the refined method and that of Mosallem & Chen. Figure 4.5 shows the maximum slab loads, using slab moments, for ten construction steps.

Table 4.3 Construction Load Distribution for Example 2 of Shoring System (Present Method)

		Shore Load		Slab Load			
		at beginning of operation	at end of operation	at beginning of operation		at end of operation	
Step	Level	F_{max} (D)	F_{max} (D)	V (D)	M_{max} (D)	V (D)	M_{max} (D)
1	1	1.00	1.00	0.04	0.13	0.01	0.13
2	1	2.07	2.16	0.34	0.34	0.13	0.17
	2	1.00	1.00	0.05	0.13	0.01	0.20
3,4	1	-	-	1.33	1.26	1.33	1.28
	2	0.56	0.44	0.67	0.96	0.67	0.79
5	1	0.65	1.04	1.49	1.46	1.35	1.29
	2	0.94	1.06	1.09	1.19	0.91	0.94
	3	1.00	1.00	0.05	0.15	0.07	0.16
6	1	-	-	1.69	1.60	1.69	1.60
	2	0.87	0.87	1.11	1.31	1.11	1.31
	3	0.87	0.87	0.20	0.28	0.20	0.28
7,8	1	-	-	1.00	1.00	1.00	1.00
	2	-	-	1.57	1.74	1.57	1.74
	3	0.60	0.60	0.43	0.47	0.43	0.47
9	1	-	-	1.21	1.35	1.29	1.43
	2	0.40	0.45	1.83	2.31	1.81	2.18
	3	1.54	1.44	0.90	0.84	0.85	0.83
	4	1.00	1.00	0.07	0.15	0.05	0.18
10	1	-	-	1.00	1.00	1.00	1.00
	2	-	-	1.92	1.90	1.92	1.90
	3	1.16	1.16	0.92	1.18	0.92	1.18
	4	0.90	0.90	0.16	0.23	0.16	0.23

D = Slab weight + formwork weight for the tributary area of a given shore
F = shore load
V = slab load (shear force)
M = slab load (moment)

Table 4.4 Comparison of Simplified and Refined Methods with Present Method for Example 2 of Shoring System

		Simplified Method		Refined Method (Liu, et al.)		Present Method		Comparison			
		1	2	3	4	5 -	6	7	8	9	10
Step	Level	F_{max} *(D)*	V *(D)*	F_{max} *(D)*	V *(D)*	F_{max} *(D)*	V_{max} *(D)*	1/5	2/6	3/5	4/6
1	1	1.00	0.00	1.00	0.00	1.00	0.04	1.00	0.00	1.00	0.00
2	1	(2.00)	0.00	(1.83)	0.40	(2.16)	0.34	0.93	0.00	0.85	1.18
	2	1.00	0.00	1.00	0.00	1.00	0.05	1.00	0.00	1.00	0.00
3,4	1	-	1.00	-	1.43	-	1.33		0.75		1.08
	2	0.00	1.00	0.54	0.57	0.56	0.67	0.00	1.49	0.96	0.85
5	1	1.00	1.00	0.44	1.67	1.04	1.49	0.96	0.67	0.42	1.12
	2	1.00	1.00	0.94	1.07	1.06	1.09	0.94	0.92	0.89	0.98
	3	1.00	0.00	1.00	0.00	1.00	0.07	1.00	0.00	1.00	0.00
6	1	-	1.33	-	1.79	-	1.69		0.79		1.06
	2	0.33	1.34	0.84	1.11	0.87	1.11	0.38	1.21	0.97	1.00
	3	0.67	0.33	0.96	0.10	0.87	0.20	0.77	1.65	1.10	0.50
7,8	1	-	1.00	-	1.00	-	1.00		1.00		1.00
	2	-	1.50	-	1.68	-	1.57		0.96		1.07
	3	0.50	0.50	0.73	0.32	0.60	0.43	0.83	1.16	1.22	0.74
9	1	-	1.34	-	1.18	-	1.29		1.04		0.91
	2	0.34	1.83	0.25	1.95	0.45	1.83	0.76	1.00	0.56	1.07
	3	1.17	0.83	1.18	0.87	1.54	0.90	0.76	0.92	0.77	0.97
	4	1.00	0.00	1.00	0.00	1.00	0.07	1.00	0.00	1.00	0.00
10	1	-	1.00	-	1.00	-	1.00		1.00		1.00
	2	-	(1.95)	-	(2.05)	-	(1.92)		1.02		1.07
	3	0.95	0.94	1.07	0.95	1.16	0.92	0.82	1.02	0.92	1.03
	4	0.89	0.11	1.00	0.00	0.90	0.16	0.99	0.69	1.11	0.00

D = Slab weight + formwork weight for the tributary area of a given shore
F = shore load
V = slab load (shear force)
( ) = maximum load

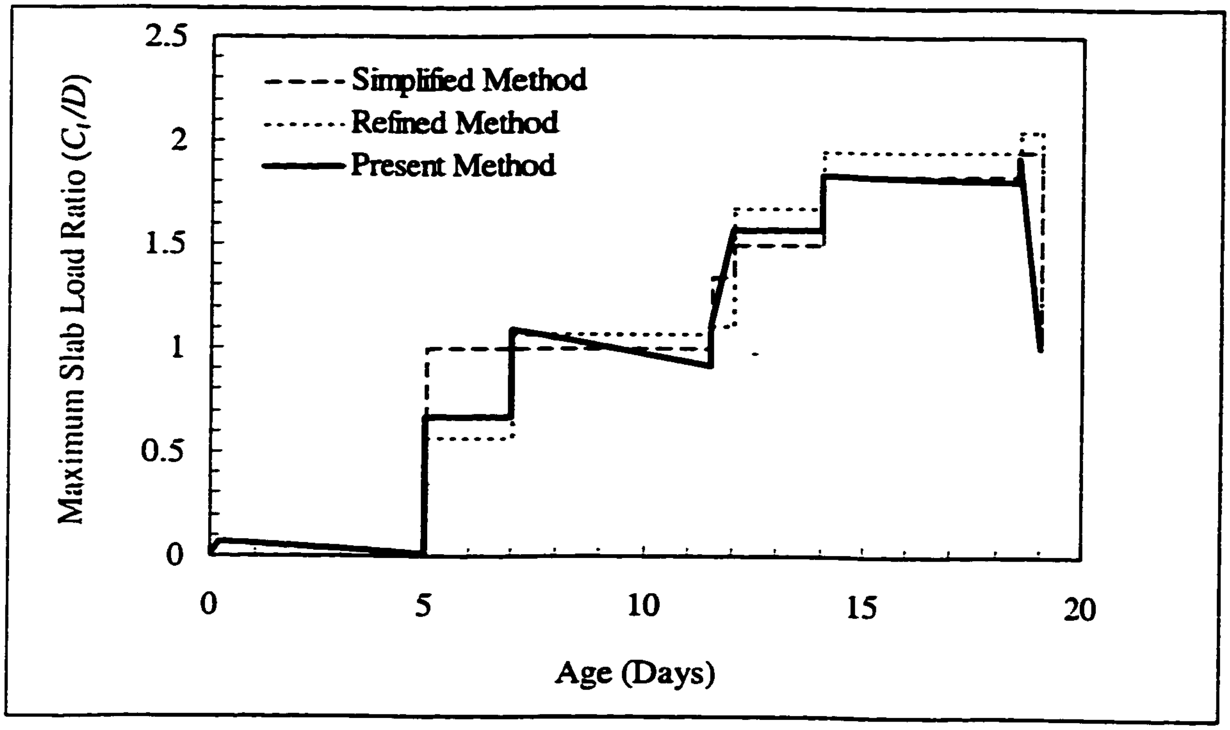


Fig. 4.4 Comparison of the Maximum Slab Load Ratio During Construction Using Shear Force (Example 2 of Shores System)

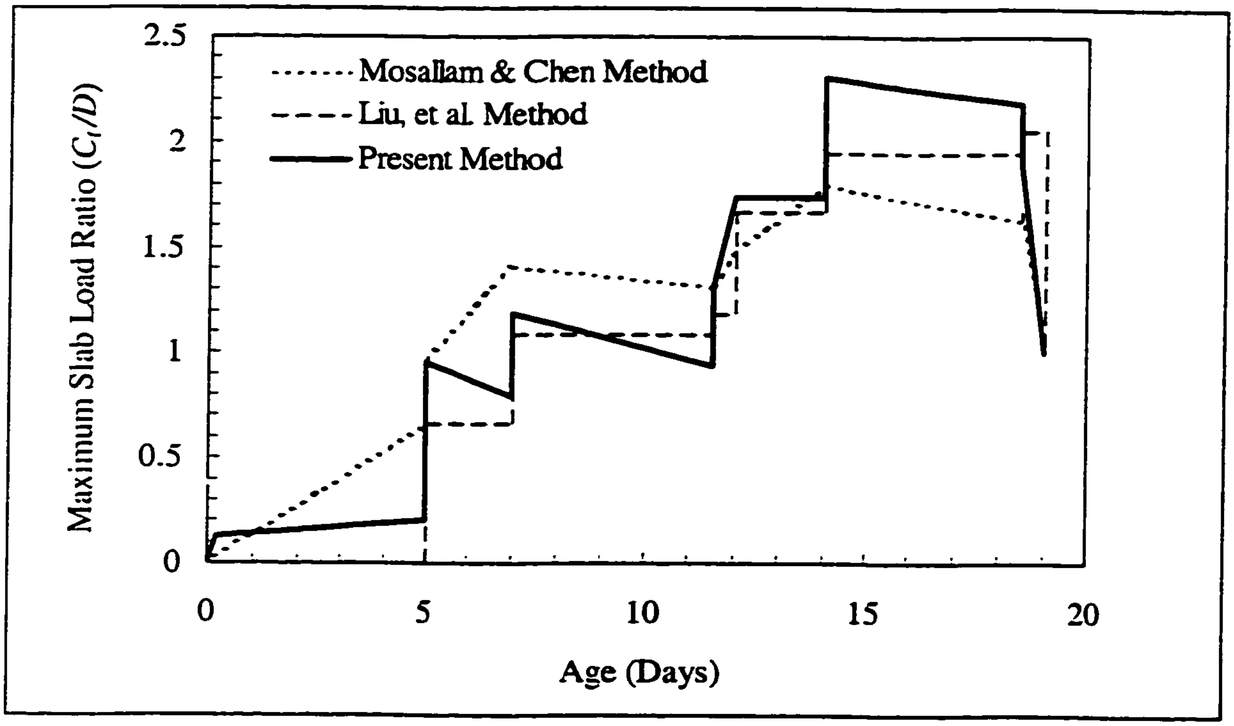


Fig. 4.5 Comparison of the Maximum Slab Load Ratio During Construction Using Slab Moment (Example 2 of Shoring System)

Table 4.5 Comparison of Mosallam & Chen and Refined Methods with Present Method for Example 2 of Shoring System

		Mosallam & Chen Method		Refined Method (Liu, et al.)		Present Method		Comparison			
		1	2	3	4	5	6	7	8	9	10
Step	Level	F_{max} (D)	M_{max} (D)	F_{max} (D)	M_{max} (D)	F_{max} (D)	M_{max} (D)	1/5	2/6	3/5	4/6
1	1	1.00	0.00	1.00	0.00	1.00	0.13	1.00	0.00	1.00	0.00
2	1	(1.51)	0.86	(1.83)	0.37	(2.16)	0.34	0.70	2.53	0.85	1.09
	2	1.00	0.00	1.00	0.00	1.00	0.20	1.00	0.00	1.00	0.00
3,4	1	-	1.07	-	1.41	-	1.28		0.84		1.10
	2	0.06	0.97	0.54	0.67	0.56	0.96	0.11	1.01	0.96	0.70
5	1	0.39	1.32	0.44	1.60	1.04	1.46	0.38	0.90	0.42	1.10
	2	0.72	1.41	0.94	1.09	1.06	1.19	0.68	1.18	0.89	0.92
	3	1.00	0.00	1.00	0.00	1.00	0.16	1.00	0.00	1.00	0.00
6	1	-	1.27	-	1.78	-	1.60		0.79		1.11
	2	0.29	1.31	0.84	1.19	0.87	1.31	0.33	1.00	0.97	0.91
	3	0.62	0.51	0.96	0.06	0.87	0.28	0.71	1.82	1.10	0.21
7,8	1	-	1.00	-	1.00	-	1.00		1.00		1.00
	2	-	1.47	-	1.67	-	1.74		0.84		0.96
	3	0.50	0.59	0.73	0.37	0.60	0.47	0.83	1.26	1.22	0.79
9	1	-	1.22	-	1.21	-	1.43		0.85		0.85
	2	0.25	(1.80)	0.25	1.95	0.45	(2.31)	0.63	0.78	0.63	0.84
	3	1.12	1.10	1.18	0.87	1.54	0.84	0.73	1.31	0.77	1.04
	4	1.00	0.00	1.00	0.00	1.00	0.18	1.00	0.00	1.00	0.00
10	1	-	1.00	-	1.00	-	1.00		1.00		1.00
	2	-	1.63	-	(2.06)	-	1.90		0.86		1.08
	3	0.75	0.91	1.07	0.92	1.16	1.18	0.65	0.77	0.92	0.78
	4	0.62	0.51	1.00	0.03	0.90	0.23	0.69	2.22	1.11	0.13

D = slab weight + formwork weight for the tributary area of a given shore
F = shore load
M = slab load (moment)
( ) = maximum load

The present method is in agreement with the method of Mosallem & Chen as to where and when the absolute maximum shore and slab loads occur. The refined method, although is in agreement with the others on the location and time of occurrence of the absolute maximum shore load, it is not in agreement with respect to the time when the absolute maximum slab load occurs (Table 4.5). Mosallam & Chen's method underestimates the absolute maximum shore and slab loads by 30% and 22% with respect to the present method, respectively. The refined method underestimates the absolute maximum shore and slab load by 15% and 11% with respect to the present method, respectively.

Example 3

The basic structural configuration of ten-story concrete flat slab building is shown in Fig. 4.6. The building has a uniform story height of 2.90m (9.5ft.) and a typical slab thickness of 216mm (8.50in.). The concrete slab was designed to carry its self-weight, a 2.39KPa (50psf) live load, and a 0.96KPa (20psf) for mechanical load and partitions. The columns have a typical 406 x 406mm (16 x 16in.) nominal square cross section. The wooden shores have a 100 x 100mm (4 x 4in.) nominal square cross section. The building is shored at 1.52m (5.0ft.) intervals in both directions. The construction rate is one floor per week with three levels of wooden shores.

The modulus of elasticity, E_c, and the 28-day cylinder strength of concrete, f_c', for both the floors and the columns, are 2.6 x 10^4MPa (3.8 x 10^6psi) and 28MPa (4,000psi), respectively.

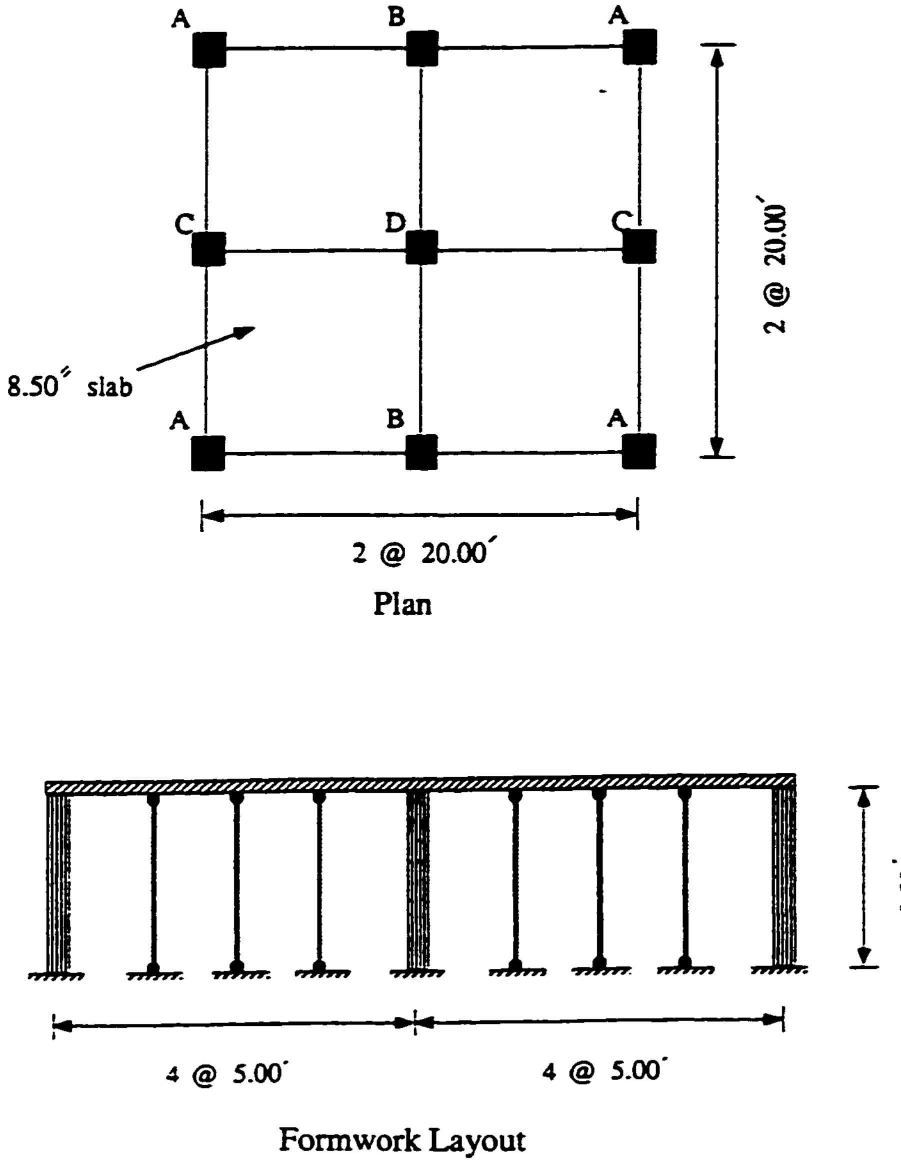


Fig. 4.6 Plan and Formwork Layout of Ten-Story Building (Mosallam and Chen, 1991)

The modulus of elasticity for wooden shores, E_w, and the compression strength of wood, f_w, are 1.16 x 10^4MPa (1.7 x 10^6psi) and 5.6MPa (810psi), respectively.

The results of the present method at the beginning and end of each construction step are shown in Table 4.6. Table 4.7 shows a comparison between Mosallam & Chen's method and the present method. The maximum slab loads calculated by the above methods, using slab moments, for ten construction steps are shown in Fig. 4.7. Both methods are in agreement on the location and time of occurrence of the absolute maximum shore and slab loads. Mosallam & Chen's method, however, overestimates the absolute maximum shore and slab loads by 32% and underestimates the absolute maximum slab load by 32% with respect to the present method.

In the above three examples, the differences in the slab and shore loads determined by the various methods are largely related to variations in there assumptions. The present method considers the changes in loads during construction steps as a result of concrete hardening, shrinkage and creep effects on concrete columns and slabs of whole structural system. It is seen from the results of the above methods that the absolute heavily loaded shores are those at ground level before the first shores are removed and the absolute heavily loaded slab is the last slab placed before shores of the ground level are removed. In example 1, the field measurements and improved simplified method results are the average of nine shores located at the centre of slab panel (see Fig. 4.1). According to those nine shores the slab loads are calculated numerically, which may lead to inaccurate slab loads estimates.

Table 4.6 Construction Load Distribution for Example 3 of Shoring System (Present Method)

		Shore Load		Slab Load	
		at beginning of operation	at end of operation	at beginning of operation	at end of operation
Step	Level	F_{max} (D)	F_{max} (D)	M_{max} (D)	M_{max} (D)
1	1	1.60	1.10	0.18	0.12
2	1	2.42	2.30	0.72	0.08
	2	1.60	1.10	0.23	0.17
3	1	2.92	3.14	0.37	0.07
	2	2.18	1.98	0.99	0.27
	3	1.60	1.10	0.22	0.15
4	1	-	-	2.07	1.78
	2	1.24	0.78	1.05	1.17
	3	0.84	0.64	0.44	0.48
5	1	-	-	1.92	1.87
	2	0.92	0.98	1.78	1.75
	3	1.65	1.38	1.74	1.10
	4	1.60	1.10	0.22	0.16
6	1	-	-	1.00	1.00
	2	-	-	2.28	2.20
	3	0.96	1.01	1.33	1.35
	4	1.04	0.97	0.22	0.23
7	2	-	-	2.48	2.51
	3	1.41	1.42	1.87	1.77
	4	1.93	1.60	1.23	0.58
	5	1.60	1.10	0.22	0.16
8	2	-	-	1.00	1.00
	3	-	-	2.43	2.43
	4	1.08	1.08	0.98	0.98
	5	0.91	0.91	0.25	0.25
9	3	-	-	2.67	2.67
	4	1.39	1.41	1.52	1.46
	5	1.88	1.60	1.23	0.62
	6	1.60	1.10	0.22	0.16
10	3	-	-	1.00	1.00
	4	-	-	2.39	2.39
	5	1.12	1.12	1.04	1.04
	6	0.97	0.97	0.22	0.22

D = slab weight for the tributary area of a given shore
F = shore load
M = slab load (moment)

Table 4.7 Comparison of Mosallam & Chen Method with Present Method for Example 3 of Shoring System

		Mosallam & Chen Method		Present Method		Comparison	
		1	2	3	4	5	6
Step	Level	F_{max} (D)	M_{max} (D)	F_{max} (D)	M_{max} (D)	1/3	2/4
1	1	1.53	0.05	1.60	0.18	0.97	0.28
2	1	2.91	0.31	2.42	0.72	1.20	0.43
	2	1.53	0.05	1.60	0.23	0.96	0.22
3	1	(3.44)	0.49	(3.14)	0.37	1.10	1.32
	2	2.51	0.64	2.18	0.99	1.15	0.65
		1.53	0.05	1.60	0.22	0.96	0.23
4	1		1.25	-	2.07		0.60
	2	0.39	1.11	1.24	1.17	0.31	0.95
	3	0.30	0.89	0.84	0.48	0.36	1.85
5	1	-	1.77	-	1.92		0.92
	2	0.87	1.68	0.98	1.78	0.89	0.94
	3	1.29	1.47	1.65	1.74	0.78	0.84
		1.53	0.05	1.60	0.22	0.96	0.23
6	1	-	1.00	-	1.00		1.00
	2	-	1.24	-	2.28		0.54
	3	0.38	1.14	1.01	1.35	0.38	0.84
	4	0.31	0.87	1.04	0.23	0.30	3.78
7	2	-	1.78	-	2.51		0.71
	3	0.86	1.71	1.42	1.87	0.61	0.91
	4	1.30	1.41	1.93	1.23	0.67	1.15
	5	1.53	0.05	1.60	0.22	0.96	0.23
8	2	-	1.00	-	1.00		1.00
	3	-	1.27	-	2.43		0.52
	4	0.37	1.10	1.08	0.98	0.34	1.12
	5	0.32	0.88	0.91	0.25	0.35	3.52
9	3	-	(1.82)	-	(2.67)		0.68
	4	0.84	1.67	1.41	1.52	0.60	1.10
	5	1.30	1.42	1.88	1.23	0.69	1.15
	6	1.53	0.05	1.60	0.22	0.96	0.23
10	3	-	1.00	-	1.00		1.00
	4	-	1.24	-	2.39		0.52
	5	0.36	1.11	1.12	1.04	0.32	1.07
	6	0.32	0.88	0.97	0.22	0.33	4.00

D = slab weight for the tributary area of a given shore
F = shore load
M = slab load (moment)
( ) = maximum load

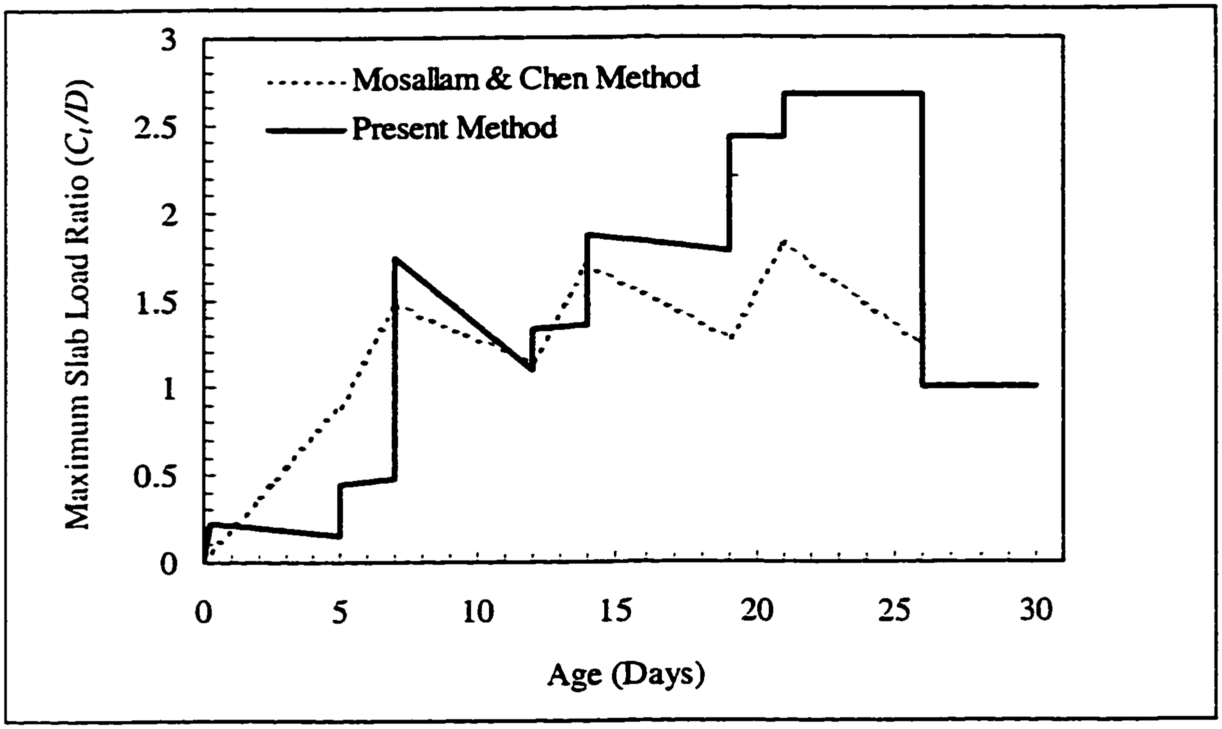


Fig. 4.7 Comparison of the Maximum Slab Load Ratio During Construction (Example 3 of Shoring System)

However, it is seen from Table 4.2 and Fig. 4.2 that the present method shows consistency with both the simplified and the improved simplified methods. The simplified method shows a good estimate of construction loads when compared with field measurements, because of large spacing panel with a small slab-shore stiffness ratio, 0.14. The slab-shore stiffness ratio is defined as the ratio of the deflection of shores in a panel under a unit force distributed uniformly to the deflection at the center of a slab under a unit force distributed uniformly on the slab. It also represents a comprehensive measurement of geometry and material properties and boundary conditions of the supporting system (Duan and Chen, 1995). In example 2, the present method shows also consistency with both the simplified and the refined methods. While the simplified method has slight differences for estimating the absolute maximum shore and slab loads, it underestimates the slab and shore loads throughout the entire structural system, especially in shore loads because of neglecting the effect of shores stiffness. In this case the slab-shore stiffness is large and equals 0.60, which is not suitable for the simplified method. Example 3 shows that the present and Mosallam & Chen's methods agree on the location and time where and when the absolute maximum shore and slab loads occur. However, there are differences in estimating the maximum shore and slab loads throughout the entire structural system. Those differences can be attributed to the use of a 3-D model for double-bay frame with neglecting the effects of shrinkage and creep on concrete elements by Mosallam & Chen's method. On the other hand, the present method uses 2-D model for single-bay frame taking into account the effects of shrinkage and creep of concrete elements.

5.2 Wall Formwork Model

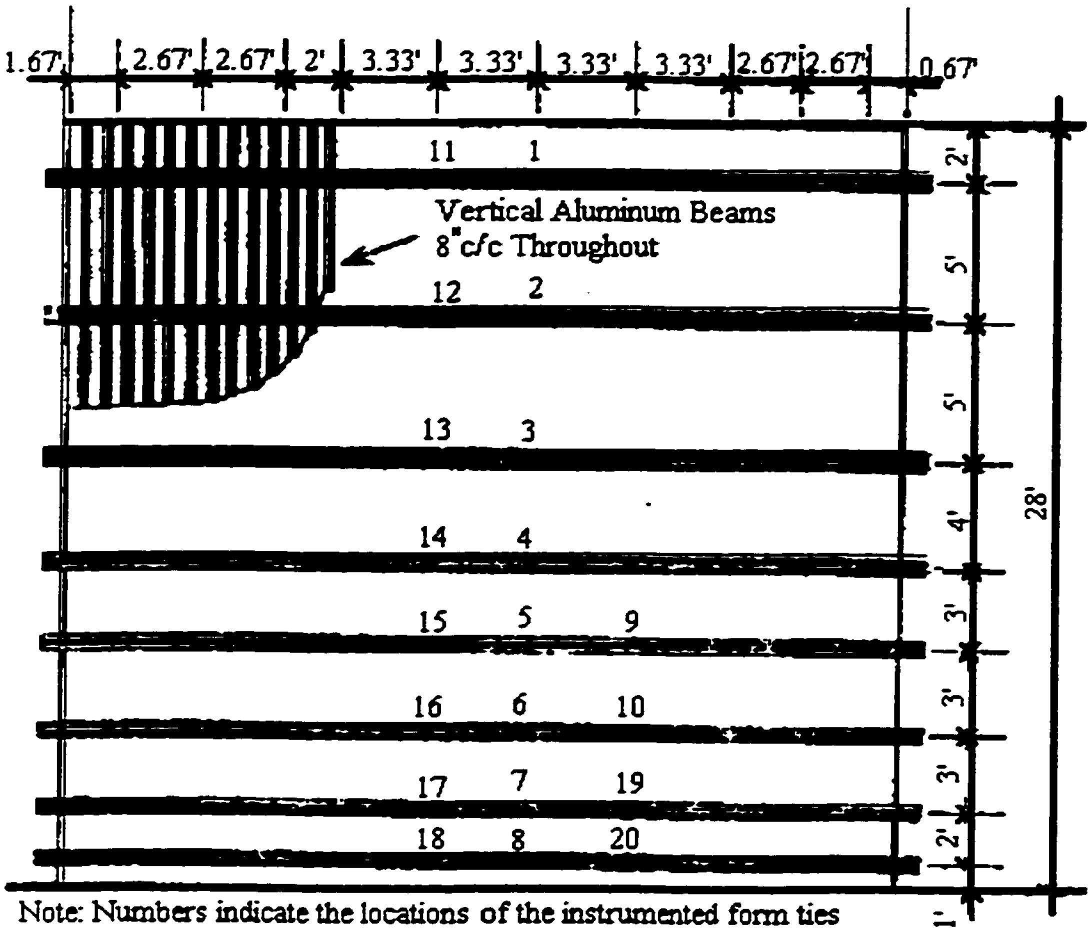
Wall formwork model is checked against the field measurements (Dunston, Johnston and McCain, 1994) and (Johnston, Khan and Philips, 1989), Rodin method (Rodin, 1953), ACI method (ACI Committee 347R-94, 1994), CIRIA method (Clear and Harrison, 1985) and Gardner method (Gardner, 1980) through the following two examples. These experimental examples were analyzed to determine the lateral pressure exerted by fresh concrete on the forms of tall walls.

Example 1

A field test was conducted on three adjacent segments of a single L-shaped test wall (Dunston, Johnston and McCain, 1994). This study took place at the U.S. Department of Energy Savannah River Site near Aiken, South Carolina, and on two separate dates. Wall Segment 1 was placed first and then walls Segments 2A and 2B were placed simultaneously with two sections separated by a subdividing expanded metal lath bulkhead. The wall was 0.6m (2ft.) thick and 8.53m (28ft.) nominal height. Fig. 4.8 and 4.9 show the elevation of wall segments and the location of instrumented form ties. Form faces were 19mm (3/4in.) plywood panels supported by vertical aluminum beams at 200mm (8in.) on center. The form ties supported the wales made from double 200mm (8in.) steel channels, which in turn supported the vertical elements. The form ties consisted of 38mm (1.5in.) diameter she-bolts with a 25mm (1in.) diameter coil threaded inner rod. The aluminum beams were in two segments vertically and the forms were braced externally for lateral stability. All segments of test wall were reinforced on each face with No.6 reinforcing bars at a cover of 61mm (2.4in.) and spaced 300mm (12in.) on center both horizontally and vertically. No.5 reinforcing bar ties were spaced at 1.5m (5ft) on center both vertically and horizontally. Table 4.8 shows the concrete mixes that used in the wall segments. The result and the major variables of the field test are given in Table 4.9.

The results of the present method, showing the maximum lateral pressures and lateral pressures at placing the last lift for the ties in the entire form are given in Tables 4.10, 4.11 and 4.12. For Wall Segment 1, the maximum estimated pressure (75.08kN/m^2) occurred at height 0.90m from the bottom, when the fresh concrete level was at height 4.88m from the bottom. The measured maximum average pressure (66.58kN/m^2) and maximum individual pressure (88kN/m^2) occurred at height 2.74m from the bottom of Wall Segment 1. The maximum estimated pressure (61.39kN/m^2) and (62.05kN/m^2) for Wall Segments 2A and 2B respectively occurred at height 0.90m from the bottom, when the fresh concrete level was at height 4.88m from the bottom. The measured maximum average pressure (54.84kN/m^2) and maximum individual pressure (62.74kN/m^2) occurred at height 2.74m from the bottom of Wall Segment 2A. For Wall Segment 2B, the measured maximum average pressure (60.80kN/m^2) and maximum individual pressure (78.10kN/m^2) occurred at height 4.87m and 2.74m, respectively, from the bottom.

Table 4.13 and Figs. 4.10, 4.11 and 4.12 show the comparison between field measurements and Rodin method, ACI method, CIRIA method, Gardner method and the present method. Rodin method underestimates the maximum lateral pressures by 17%, 19% and 26% for Wall Segments 1, 2A and 2B, respectively. ACI method overestimates the maximum lateral pressures by 190% for Wall Segment 1 and underestimates the



Note: Numbers indicate the locations of the instrumented form ties

Fig. 4.8 Elevation of Wall Segment 1
(Dunston, Johanston and McCain, 1994)

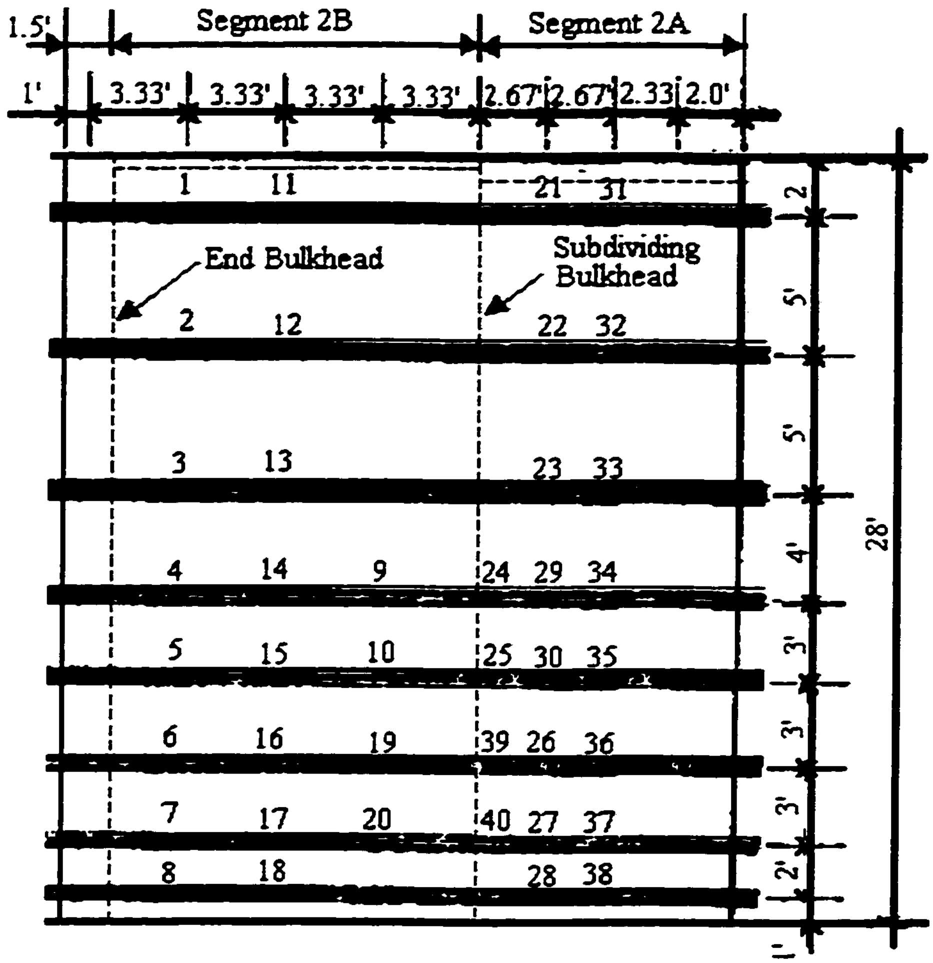


Fig. 4.9 Elevation of Wall Segment 2A and 2B (Dunston, Johanston and McCain, 1994)

Table 4.8 Concrete Mix Proportions (yd^3) of Example 1 of Wall Formwork (Dunston, Johnston and McCain, 1994)

Mix	5-B	5-C	J-2	J-2R	J-2 50G
Cement (Type II), lb.	120	115	598	598	598
Slag (GGBFS), lb.	274	260			
Fly ash, lb.	135	125			
Coarse aggregate, lb.					
N0. 4 stone		600			
No. 67 stone	1658	1110	1845	1845	1845
Sand, lb	1357	1380	1210	1210	1210
Water, lb.	240	220	267	229	267
Admixtures					
Water reducer, oz/yd^3					
-WRDA- 79	21.0	25.0	30.5		30.5
-Pozz 300N				30.0	
Air entertainment, oz/yd^3					
-DARAVAIR-	8-10	7.0	4.0	5.5	4.4
HRWR, oz/yd^3					
-Melment 50G (Plant)	42.3-47.6	45.0			72.0
-Melment 50G (site)	21.2-31.4	25.0			24.0
- Melment 33 -			64.0		
-Rheobild 1000				120	
Segment use	1	2A bottom 7ft	2B bottom 10ft	2A middle 9ft 2B middle 6ft	2A top 11ft. 2B top 12ft

Table 4.9 Test Data for Wall Segments 1, 2A and 2B

	Wall Segment 1	Wall Segment 2A	Wall Segment 2B
Wall height (m)	8. 53	8. 23	8. 47
Wall thickness (m)	0. 61	0. 61	0. 61
Average concrete temperature (C°)	13. 50	17. 22	17. 22
Average rate of pour (m/hr)	3. 35	1. 68	1. 74
Average unit weight (kN/m^2)	22. 60	22. 90	22. 90
Average slump (cm)	8.0	9.0	9.0
Vertical reinforcement	No.6 @ 0.3 m c/c	No.6 @ 0.3 m c/c	No.6 @ 0.3 m c/c
Form tie diameter (m)	0.025	0.025	0.025
Maximum average lateral pressure (kN/m^2)	66.58	54.84	60.80
Maximum individual lateral pressure (kN/m^2)	88.60	62.74	78.10
Concrete height at which average maximum pressure occurred (m)	N/A	N/A	N/A
Form height at which maximum average pressure occurred (m)	2.74	2.74	4.87
Form height at which maximum individual pressure occurred (m)	2.74	2.74	2.74

Table 4.10 Lateral Pressures on Wall Segment 1 (Present Method)

Form Height (m)	Maximum Lateral Pressures (kN/m²)	Lateral Pressures at placing the last lift (kN/m²)
0.30	53.14	52.40
0.91	(75.08)	74.46
1.83	74.87	73.48
2.74	71.85	71.29
3.66	72.69	72.44
4.88*	72.63	72.63
6.40	50.75	50.75
7.92	17.33	17.33
8.53	0.00	0.00

( ) = Maximum pressure
* = Concrete height when the maximum pressure occurred

Table 4.11 Lateral Pressures for Wall Segment 2A (Present Method)

Form Height (m)	Maximum Lateral Pressures (kN/m²)	Lateral Pressures at placing the last lift (kN/m²)
0.30	42.77	41.93
0.91	(61.39)	59.92
1.83	59.84	58.77
2.74	58.55	57.18
3.66	58.67	58.36
4.88*	59.30	59.30
6.40	43.03	43.03
7.92	10.29	10.29
8.23	0.00	0.00

( ) = Maximum pressure
* = Concrete height when the maximum pressure occurred

Table 4.12 Lateral Pressures on Wall Segment 2B (Present Method)

Form Height (m)	Maximum Lateral Pressures (kN/m^2)	Lateral Pressures at placing the last lift (kN/m^2)
0.30	43.25	42.40
0.91	(62.05)	60.61
1.83	60.56	59.45
2.74	59.21	57.74
3.66	59.36	58.84
4.88*	60.66	60.66
6.40	47.61	47.61
7.92	16.44	16.44
8.47	0.00	0.00

( ) = Maximum pressure
* = Concrete height when the maximum pressure occurred

Table 4.13 Comparison of Theoretical and Measured Pressures of Example 1 of Wall Formwork

	Measured	Rodin's Method	ACI Method	CIRIA Method	Gardner's Method	Present Method	Comparison				
	1	2	3	4	5	6	7	8	9	10	11
	P_{max} (kN/m^2)	P_{max} (kN/m^2)	P_{max} (kN/m^2)	P_{max} (kN/m^2)	P_{max} (kN/m^2)	P_{max} (kN/m^2)	2/1	3/1	4/1	5/1	6/1
Wall Segment 1	66.58	55.22	193.03	80.95	55.82	75.08	0.83	2.90	1.22	0.84	1.13
Wall Segment 2A	54.84	44.44	43.59	62.27	48.46	61.39	0.81	0.79	1.14	0.88	1.12
Wall Segment 2B	60.80	44.97	45.03	62.75	48.74	62.05	0.74	0.74	1.03	0.80	1.02

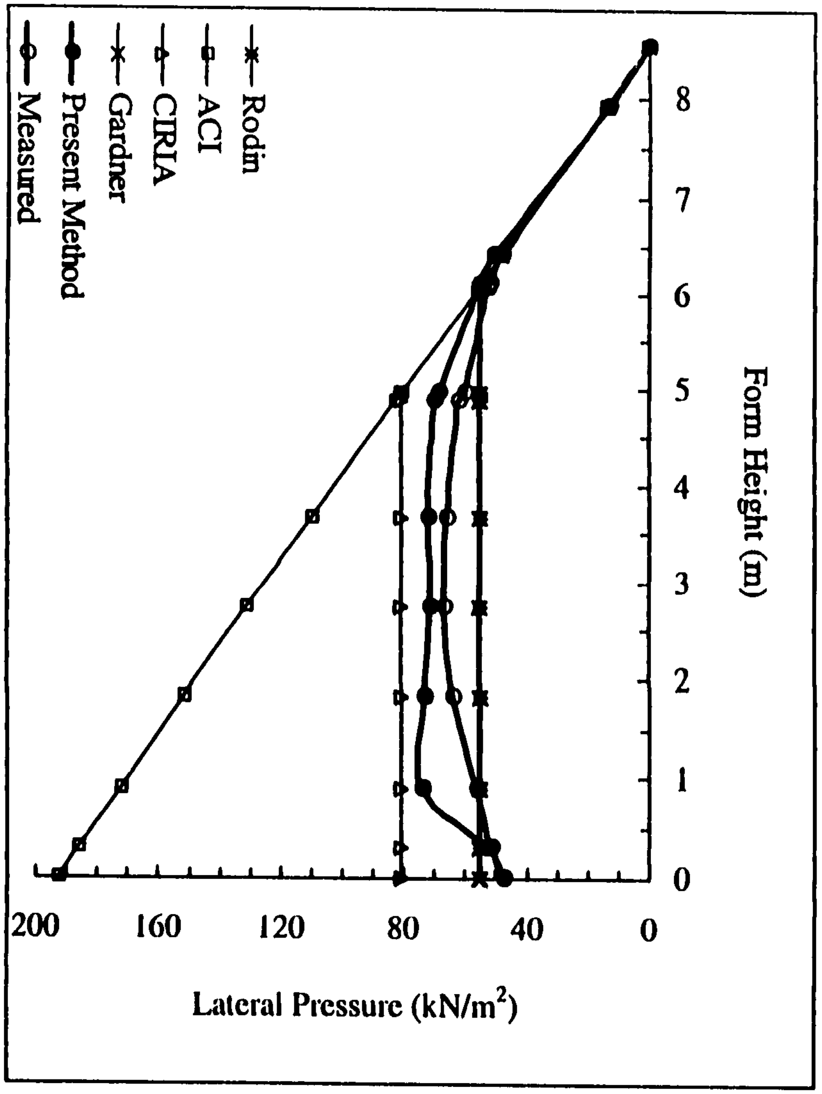


Fig. 4.10 Concrete Pressure Envelopes of Wall Segment 1

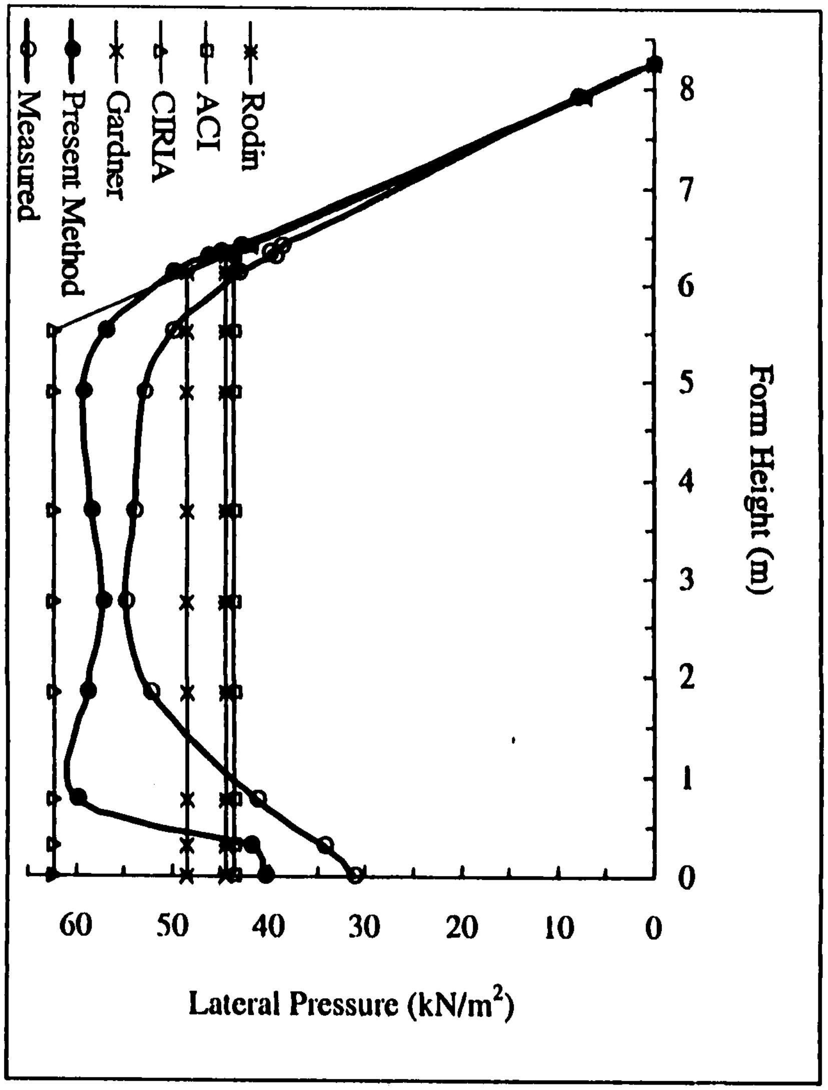


Fig. 4.11 Concrete Pressure Envelopes of Wall Segment 2A

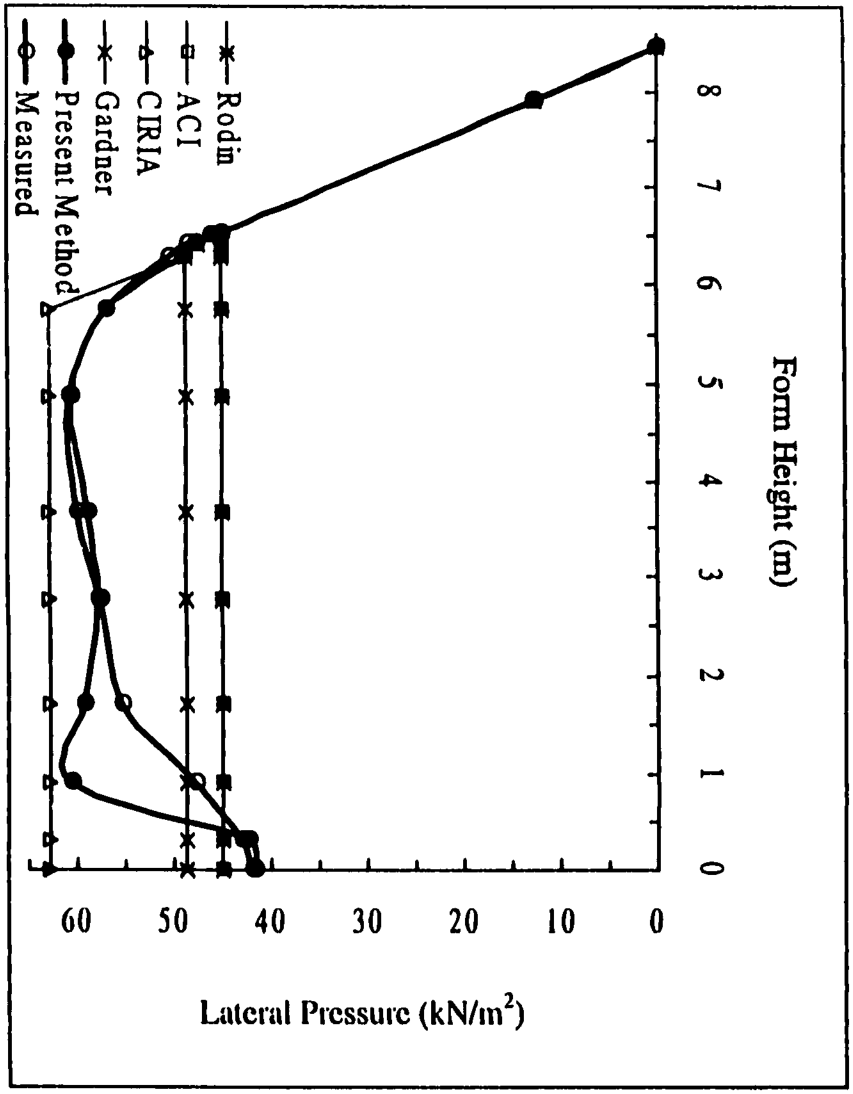


Fig. 4.12 Concrete Pressure Envelopes of Wall Segment 2B

maximum lateral pressures by 21% and 26% for Wall Segments 2A and 2B respectively. CIRIA method overestimates the maximum lateral pressures by 22% and 14% for Wall Segments 1 and 2A, respectively and has a slight difference, 3% higher, in estimating the maximum lateral pressure for Wall Segment 2B. Gardner method underestimates the maximum lateral pressures by 16%, 12% and 20% for Wall Segments 1, 2A and 2B, respectively. The present method overestimates the maximum lateral pressures by 13% and 12% for Wall Segments 1 and 2A respectively and has a slight difference, 2% higher, in estimating the maximum lateral pressure for Wall Segment 2B.

Example 2

A field study, at the Shearon Harris Generating Station located near Raleigh, N.C., was conducted on two walls; 6.25m (20.5ft) high and approximately 13.40m (44.0 ft) long (Johnston, Khan and Philips, 1989). Wall I was 1.52m (5.0ft.) thick and Wall II 1.22m (4.0ft) thick. Each face of both walls was reinforced with vertical No.11 bars spaced 0.15m (6.0in.) on center, and horizontal No.11 bars spaced 0.30m (12in.) on center. Figures 4.13 and 4.14 show the layout of the reinforcement, formwork, and instrumented form ties for Walls I and II respectively. Form faces were 19mm (3/4in.) plywood panels. The panels were stiffened with C3 $\times$ 4.1 horizontal ribs spaced at 0.30m (12in.) on center. The horizontal rips were supported by vertical strongbacks made from double C5 $\times$ 6.7 channels, which in turn were supported at intervals of the vertical span by the ties. The form ties consisted of 28.13mm (1-1/8in.) diameter she-bolts with a 19.00mm (3/4in.) diameter coil threaded inner rod. Lateral stability was provided by a series of guy wires. In addition, horizontal channels were connected to the outside of C5 $\times$ 6.7 channels for panel alignment. Table 4.14 shows the concrete mixes that were used in both walls. The result and the major variables and the results of the field test are given in Table 4.15.

The results of the present method, showing the maximum lateral pressures and lateral pressures at placing the last lift for the ties of the entire form are given in Tables 4.16 and 4.17. For Wall I, the maximum estimated lateral pressure ($53.18kN/m^2$) occurred at height 2.19m from the bottom, when the fresh concrete level was at height 6.25m from the bottom. The measured maximum average lateral pressure ($50.86kN/m^2$) occurred at height 1.83m from the bottom, when the fresh concrete level was at height 4.94m from the bottom of Wall I. The maximum estimated lateral pressure ($60.05kN/m^2$) for Wall II occurred at height 2.68m from the bottom, when the fresh concrete level was at height 6.25m from the bottom. The measured maximum average lateral pressure ($42.67kN/m^2$) occurred at height 1.89m from the bottom, when the fresh concrete level was at height 4.63m from the bottom of Wall II.

Table 4.18 and Figs. 4.15 and 4.16 show the comparison between field measurements and Rodin's method, ACI method, CIRIA method, Gardner's method and the present method. Rodin's method underestimates the maximum lateral pressures by 29% for Wall I and has a slight difference, 4% higher, in estimating the maximum lateral pressure for Wall II. ACI method underestimates the maximum lateral pressures by 43% for Wall I and has a slight difference, 3% lower, in estimating the maximum lateral pressure for Wall II. CIRIA method underestimates the maximum lateral pressures by 28% for Wall I and

Table 4.14 Concrete Mix Quantities per Cubic Yard for Example 2 of Wall Formwork (Johnston, Kahn and Phillips, 1989)

Mix (1)	Quantity Wall I (2)	 Wall II (3)
Cement (Type II)	900 lb.	640 lb.
Water	305 lb.	246 lb.
Sand	1,145 lb.	1,294 lb.
Coarse aggregate	1,349 lb.	1,621 lb.
Water reducing retarder (Protex PDA-25R, 4oz/100lb)	36 oz	26 oz
Air entraining agent (Protex AES)	11 oz, (1.25 oz/100 lb.)	3.2 oz, (0.5 oz/100 lb.)
Unit weight	137 pcf	141 pcf

Table 4.15 Test Data for Wall I and Wall II

	Wall I	Wall II
Wall height (m)	6.25	6.25
Wall thickness (m)	1.52	1.22
Average concrete temperature (C°)	28.89	21.11
Average rate of pour (m/hr)	1.07	1.89
Average unit weight (kN/m^2)	21.53	22.16
Average slump (cm)	10.0	11.0
Vertical reinforcement	No.11@ 0.15 m c/c	No.11 @ 0.15 m c/c
Form tie diameter (m)	0.019	0.019
Immersed depth of vibrator (m)	0.51	0.51
Maximum average lateral pressure (kN/m^2)	50.86	42.67
Maximum individual lateral pressure (kN/m^2)	N.A	N.A
Concrete height at which average maximum pressure occurred (m)	4.94	4.63
Form height at which maximum average pressure occurred (m)	1.83	1.89

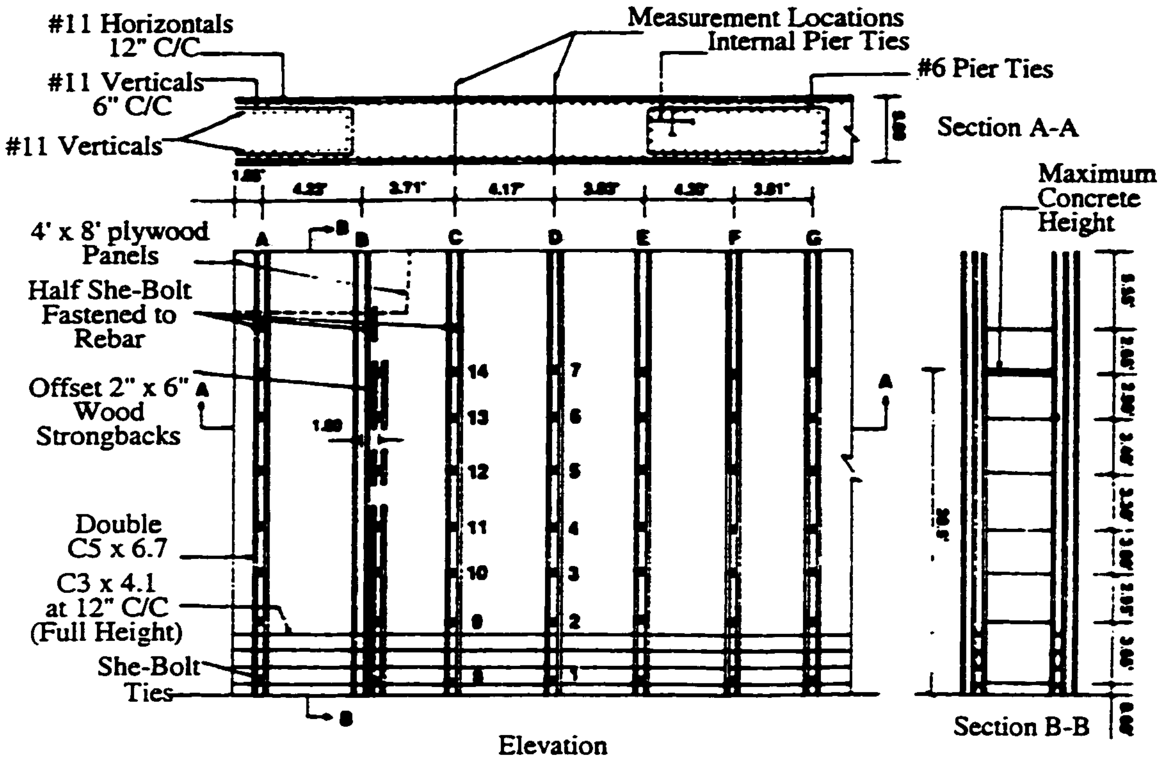


Fig. 4.13 Wall I Form and Reinforcement Layout (Johnston, Kahn and Phillips, 1989)

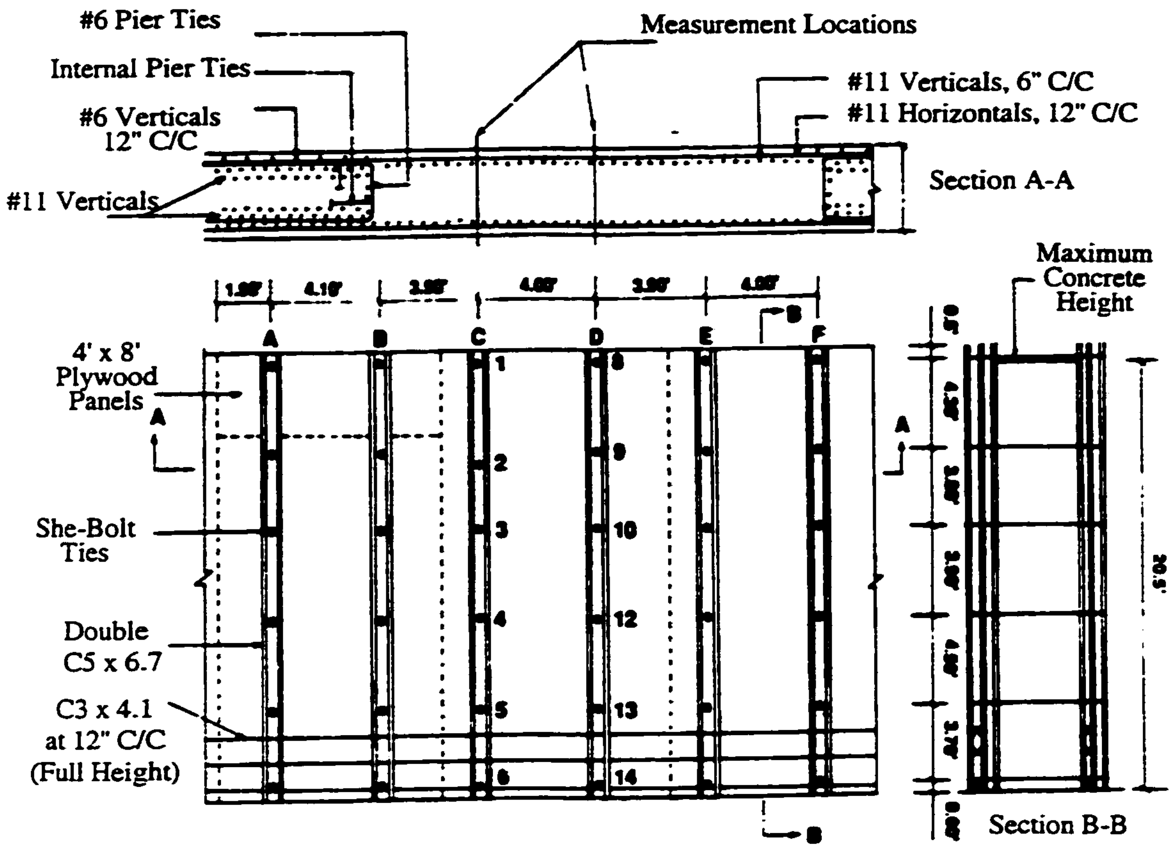


Fig. 4.14 Wall II Form and Reinforcement Layout
(Johnston, Kahn and Phillips, 1989)

Table 4.16 Lateral Pressures on Wall I

Form Height (m)	Maximum Lateral Pressures (kN/m^2)	Lateral Pressures at placing the last lift (kN/m^2)
0.18	10.32	10.32
1.29	38.44	38.44
2.19	(53.18)	53.18
3.10	48.22	48.22
4.11	36.07	36.07
5.15	23.82	23.82
6.03	11.45	11.45
6.25*	0.00	0.00

( ) = Maximum pressure
* = Concrete height when the maximum pressure occurred

Table 4.17 Lateral Pressures on Wall II

Form Height (m)	Maximum Lateral Pressures (kN/m^2)	Lateral Pressures at placing the last lift (kN/m^2)
0.18	17.08	17.08
1.31	53.38	53.38
2.68	(60.05)	60.05
3.87	45.36	45.36
5.03	29.24	29.24
6.25*	0.00	0.00

( ) = Maximum pressure
* = Concrete height when the maximum pressure occurred

Table 4.18 Comparison of Theoretical and Measured Pressures of Example 2 of Wall Formwork

	Measured	Rodin's Method	ACI Metod	CIRIA Method	Gardner's Method	Present Method	Comparison				
	1	2	3	4	5	6	7	8	9	10	11
	P_{max} (kN/m^2)	P_{max} (kN/m^2)	P_{max} (kN/m^2)	P_{max} (kN/m^2)	P_{max} (kN/m^2)	P_{max} (kN/m^2)	2/1	3/1	4/1	5/1	6/1
Wall I	50.86	35.88	28.74	36.40	63.80	53.18	0.71	0.57	0.57	1.25	1.05
Wall II	42.67	44.54	44.06	51.34	62.88	60.05	1.04	0.97	0.97	1.47	1.41

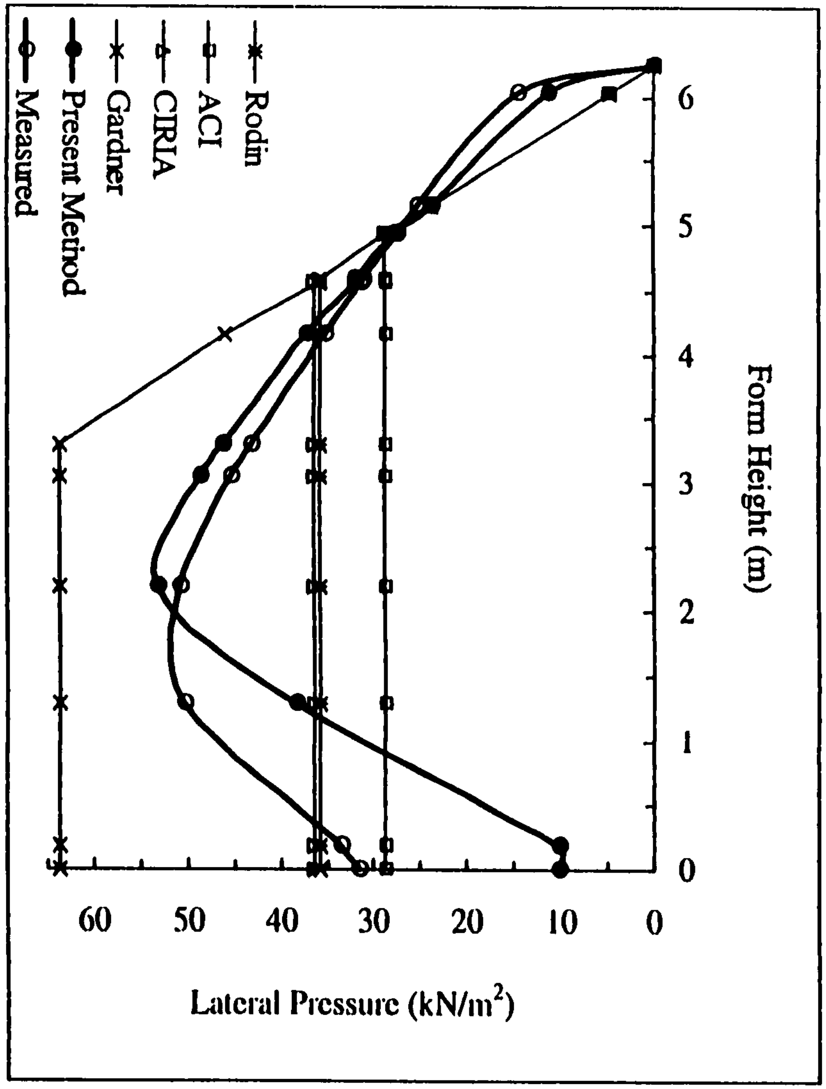


Fig. 4.15 Concrete Pressure Envelopes of Wall I

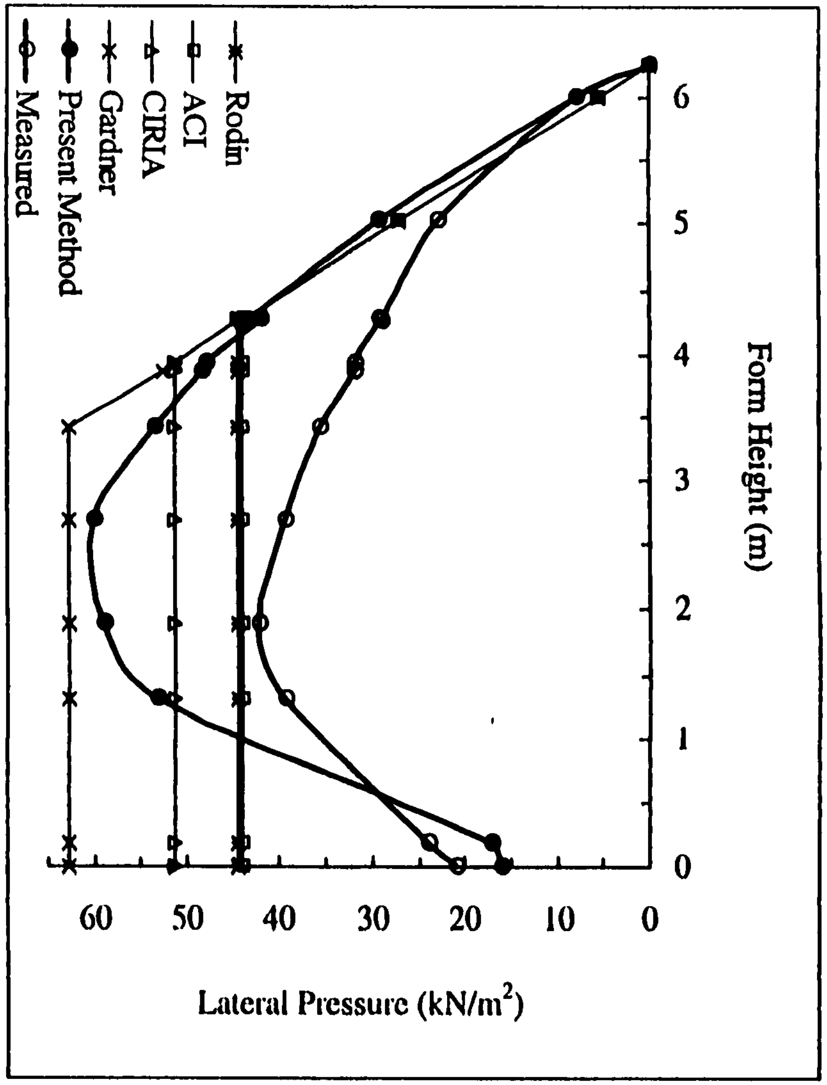


Fig. 4.16 Concrete Pressure Envelopes of Wall II

overestimates the maximum lateral pressure by 20% for Wall II. Gardner's method overestimates the maximum lateral pressures by 25% and 47% for Walls I and II respectively. The present method has a slight difference, 5% higher, in estimating the maximum lateral pressure for Wall I and overestimates the maximum lateral pressure by 41% for Wall II.

Concrete pressure envelopes of the above two examples reveal that the pressure envelopes drawn from the present method results start from the top surface of fresh concrete, initially, follow the hydrostatic line and then deviate from the hydrostatic condition with a low rate until reaching the maximum lateral pressure. The pressure envelopes, after reaching the maximum, start decreasing gradually up to the lowest lateral pressure at the form base. These shapes are similar to the measured pressure envelopes. ACI, CIRIA and Gardner's methods assume the lateral pressure envelope to be a full liquid head from the top of placement to a maximum calculated pressure and to be remaining constant at that maximum value at greater depths. The present method gives a reasonable estimation for the maximum lateral pressures on all walls except Wall II in Example 2. For Wall II, all major variables led to a higher estimation of the maximum lateral pressure. The low measured maximum lateral pressure on Wall II may refer to the contribution of aggregate type; air content and cement content effects on concrete stiffening time (Johnston, Kahn and Phillips, 1989). For this study range, the present method neglects the effect of concrete properties on the initial pressure envelope by assuming the coefficient C in Equation 3.2 to be equal 1, which seems to be less than 1 for a case like Wall II

The present method is a computer-aided analysis while the others are empirical equations derived from experimental data. This advantage allows the present method to estimate the lateral pressure for every form tie at any time during concrete placing which in turn allows the maximum lateral pressures to be determined. ACI method is inadequate for estimating the maximum lateral pressure, especially on Wall Segment 1. CIRIA method gives a reasonable estimation for the maximum lateral pressure. Gardner and Rodin methods give somehow reasonable estimation for lateral pressure.

CHAPTER V

PARAMETRIC STUDY

5.1 General

Two examples are investigated to determine: 1) the influence of different parameters affecting the construction load distribution among shores and interconnected slabs during construction of multistory buildings; and 2) the lateral pressures on wall formwork during concrete placing. A parametric study is conducted by changing one or more of the influencing parameters, and studying the effect of this change on the final results. The results are presented in a tabular and graphical form.

5.2 Shoring System

In this section, the examined factors influencing construction load distribution are: negligence of change in construction load during construction cycles; slab stiffness; shore stiffness; slab boundary conditions; construction live load; construction procedure; and using the slab moments for determining slab loads. The slab load history predicted by the simplified method is included for comparison purposes. The example used for parametric study is that introduced by Liu, et. al (1986), and described earlier in Chapter 4.

5.2.1 Influence of Neglecting the Change in Construction Load During Construction Cycles

The present method considers the change in construction load during construction cycles by calculating the construction load distribution twice: first at the beginning and then at the end of each construction step accounting for the effects of shrinkage and creep on concrete elements. In order to investigate that effect, the analysis was carried out without considering the change in construction load as well as the effects of shrinkage and creep on concrete elements. The results of comparison are shown in Table 5.1 and Fig. 5.1.The constant load analysis overestimates the absolute maximum slab load by 9%, and underestimates the absolute maximum shore load by 4%. In general, the constant load analysis increases slightly the construction load distribution. It can be concluded that the influence of neglecting the change in construction load during construction cycles has little effect on construction load distribution. This conclusion complies with the assumptions of the simplified method.

5.2.2 Influence of Slab Stiffness

The flexural stiffness of an uncracked section is only slightly affected by the percentage of steel, and may be taken directly proportional to the modulus of elasticity of concrete. The modulus of elasticity of concrete is proportional to the square root of concrete strength, f_c' (ACI 318, 1989). The slab stiffness can vary with time-dependent concrete strength, the introduction of additional structural features like drop panels or beams, and the change of slab thickness. The effect of slab stiffness is investigated here by analyzing the shores system using two different concrete strengths, $f_c' = 41$MPa and 28MPa.

The results of analysis are shown in Table 5.2 and Fig. 5.2. The comparison of the two cases shows that using lower concrete strength increases the absolute maximum slab load by 3%, and decreases the absolute maximum shore load by 3%. This indicates that the influence of slab stiffness, especially due to concrete strength, has insignificant effect on construction load distribution.

Table 5.1 Effect of Neglecting the Change in Construction Load during Construction Cycles on Slab Load Ratio

		Present Method		Constant Load*		Comparison	
		1	2	3	4	5	6
Step	Level	F_{max} (D)	V_{max} (D)	F_{max} (D)	V_{max} (D)	3/1	4/2
1	1	1.00	0.04	1.00	0.04	1.00	1.00
2	1	(2.16)	0.34	(2.07)	0.37	0.96	1.09
	2	1.00	0.05	1.00	0.05	1.00	1.00
3,4	1	-	1.33	-	1.40	-	1.05
	2	0.56	0.67	0.56	0.60	1.00	0.90
5	1	1.04	1.49	0.78	1.54	0.75	1.03
	2	1.06	1.09	1.23	1.00	1.16	0.92
	3	1.00	0.07	1.00	0.05	1.00	0.71
6	1	-	1.69	-	1.78	-	1.05
	2	0.87	1.11	0.79	1.15	0.91	1.04
	3	0.87	0.20	0.99	0.07	1.14	0.35
7,8	1	-	1.00	-	1.00	-	1.00
	2	-	1.57	-	1.66	-	1.06
	3	0.60	0.43	0.79	0.34	1.32	0.79
9	1	-	1.29	-	1.25	-	0.97
	2	0.45	1.83	0.52	1.90	1.16	1.04
	3	1.54	0.90	1.95	0.80	1.27	0.89
	4	1.00	0.07	1.00	0.05	1.00	0.71
10	1	-	1.00	-	1.00	-	1.00
	2	-	(1.92)	-	(2.09)	-	1.09
	3	1.16	0.92	1.64	0.92	1.41	1.00
	4	0.90	0.16	1.21	0.00	1.34	0.00

D = weight of concrete slab + weight of formwork for the tributary area of a given shore
F = shore load
V = slab load (shear force)
( ) = maximum load
* = as assumed in simplified, improved simplified and refined (Liu et al.) methods

Table 5.2 Influence of Slab Stiffness on Slab Load Ratio

		$f_c' = 41$ MPa		$f_c' = 28$ MPa		Comparison	
		1	2	3	4	5	6
Step	Level	F_{max} (D)	V_{max} (D)	F_{max} (D)	V_{max} (D)	3/1	4/2
1	1	1.00	0.04	1.00	0.03	1.00	0.75
2	1	(2.16)	0.34	(2.10)	0.26	0.97	0.76
	2	1.00	0.05	1.02	0.04	1.02	0.80
3,4	1	-	1.33	-	1.23	-	0.92
	2	0.56	0.67	0.46	0.79	0.82	1.18
5	1	1.04	1.49	1.00	1.45	0.96	0.97
	2	1.06	1.09	1.25	1.15	1.18	1.06
	3	1.00	0.07	1.00	0.05	1.00	0.71
6	1	-	1.69	-	1.67	-	0.99
	2	0.87	1.11	0.71	1.12	0.82	1.01
	3	0.87	0.20	0.92	0.20	1.06	1.00
7,8	1	-	1.00	-	1.00	-	1.00
	2	-	1.57	-	1.54	-	0.98
	3	0.60	0.43	0.72	0.47	1.20	1.09
9	1	-	1.29	-	1.20	-	0.93
	2	0.45	1.83	0.24	1.88	0.53	1.03
	3	1.54	0.90	1.14	0.91	0.74	1.01
	4	1.00	0.07	1.00	0.05	1.00	0.71
10	1	-	1.00	-	1.00	-	1.00
	2	-	(1.92)	-	(1.98)	-	1.03
	3	1.16	0.92	1.00	0.92	0.86	1.00
	4	0.90	0.16	0.96	0.10	1.07	0.63

D = weight of concrete slab + weight of formwork for the tributary area of a given shore
F = shore load
V = slab load (shear force)
( ) = maximum load

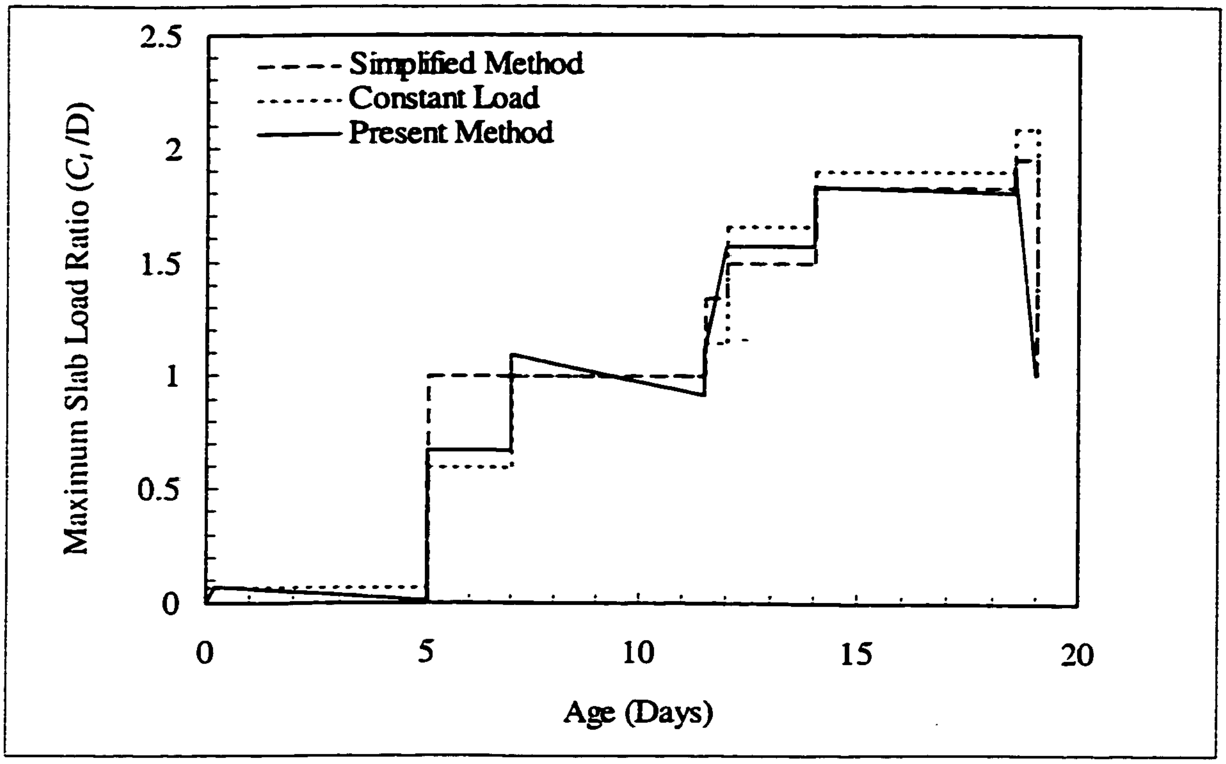


Fig. 5.1 Influence of Neglecting the Change in Construction load on Slab Load Ratio

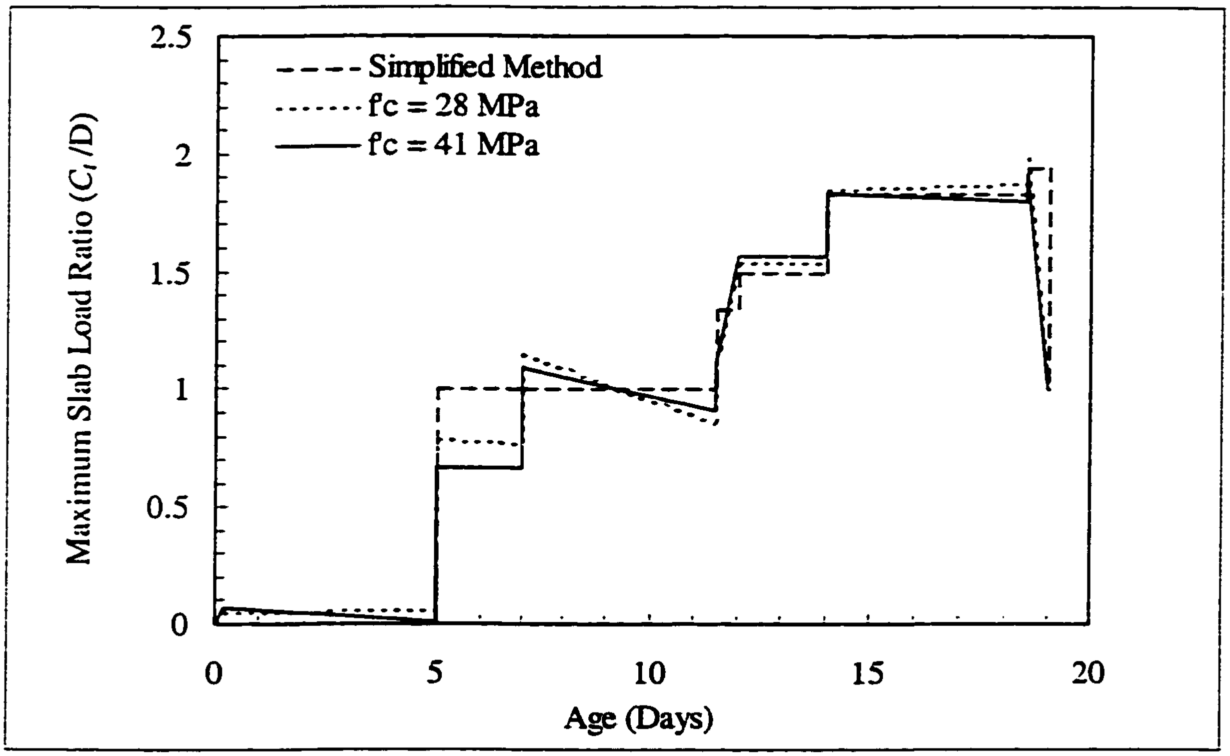


Fig. 5.2 Influence of Slab Stiffness on Slab Load Ratio

It can be seen that the simplified method is closer to the present method in the case of 28MPa concrete strength.

5.2.3 Influence of Axial Shore Stiffness

The influence of axial shore stiffness is examined by using wooden and steel shores. In both cases the axial stiffness is finite, as given by AE/L where A, E and L are the cross sectional area, the modulus of elasticity and the length of shore, respectively. The comparison results are shown in Table 5.3 and Fig. 5.3. Based on these results, it can be noted that the maximum slab and shore loads using wooden shores are slightly higher and lower than those obtained when using steel shores for the first five construction steps, respectively, and vice versa for the rest. This is a consequence of the contribution between shores and their support stiffness. In the first construction steps, the shores are supported by ground, which is basically rigid, while in the others, after removal of the first level of shores, the shores are supported by slabs, which can be considered as flexible supports. The rigid shore analysis overestimates the absolute maximum slab load by 2%, but does not influence the absolute maximum shore load.

According to the results obtained, the shore stiffness has little effect on the construction load distribution. Small differences in results refer to the small difference in the axial shore stiffness between the two used cases in which the axial stiffness of steel shores is about 3.47 times of that of wooden shores. This difference is very small when comparing with infinite shore stiffness. The case of the simplified method, which assumes infinite shore stiffness, shows consistency with the present method.

Table 5.3 Influence Shore Stiffness on Slab Load Ratio

		Flexible Shores (Wood)		Rigid Shores (Steel)		Comparison	
		1	2	3	4	5	6
Step	Level	F_{max} *(D)*	V_{max} *(D)*	F_{max} *(D)*	V_{max} *(D)*	3/1	4/2
1	1	1.00	0.04	1.02	0.01	1.02	0.25
2	1	(2.16)	0.34	(2.16)	0.16	1.00	0.47
	2	1.00	0.05	1.05	0.01	1.05	0.20
3,4	1	-	1.33	-	1.18	-	0.89
	2	0.56	0.67	0.34	0.83	0.61	1.24
5	1	1.04	1.49	1.42	1.29	1.37	0.87
	2	1.06	1.09	1.30	1.06	1.23	0.97
	3	1.00	0.07	1.00	0.02	1.00	0.29
6	1	-	1.69	-	1.59	-	0.94
	2	0.87	1.11	0.80	1.11	0.92	1.00
	3	0.87	0.20	0.69	0.31	0.79	1.55
7,8	1	-	1.00	-	1.00	-	1.00
	2	-	1.57	-	1.50	-	0.96
	3	0.60	0.43	0.55	0.50	0.92	1.16
9	1	-	1.29	-	1.41	-	1.09
	2	0.45	1.83	0.43	1.84	0.96	1.01
	3	1.54	0.90	1.26	0.88	0.82	0.98
	4	1.00	0.07	1.00	0.03	1.00	0.43
10	1	-	1.00	-	1.00	-	1.00
	2	-	(1.92)	-	(1.96)	-	1.02
	3	1.16	0.92	1.02	0.94	0.88	1.02
	4	0.90	0.16	0.90	0.11	1.00	0.69

D = weight of concrete slab + weight of formwork for the tributary area of a given shore
F = shore load
V = slab load (shear force)
( ) = maximum load

5.2.4 Influence of Slab Boundary Conditions

In order to determine the influence of slab boundary conditions on construction load distribution, two boundary conditions, fixed-ended slab and simply supported slab, were considered in the analysis of the shoring system and then compared with present method that considers the vertical slab deflection at the column joints. Comparison of the results is shown in Table 5.4 and Fig. 5.4. Figure 5.4 shows that slab load history of present method fits between the fixed-ended and simply supported slab load histories.

The analysis assuming fixed-ended slab yields a decrease in the absolute maximum slab and shore loads by 5% and 6% respectively. On the other hand, the analysis assuming simply supported slab yields an increase in the absolute maximum slab and shore loads by 3% and 12%, respectively. From Fig. 5.4, it can be seen the slab load histories predicted by the simplified method and by the analysis assuming simply supported slabs are very close. It can be concluded that the boundary conditions have little effect on construction load distribution.

5.2.5 Influence of Construction Live Load

Although the construction live load imposed on a temporary support system is transient in nature, this short-term peak load may be critical to the safety of the overall building. Experiences from structural collapses involving formwork failures occurred during placement of concrete slabs, while workmen and concreting equipment are still on top of the new deck (Mossalam and Chen, 1991). The construction live load was included in the

Table 5.4 Influence of Slab Boundary Conditions on Slab Load Ratio

		Present Method		Fixed-Ended Slab		Simply Supported Slab		Comparison			
		1	2	3	4	5	6	7	8	9	10
Step	Level	F_{max} (D)	V_{max} (D)	F_{max} (D)	V_{max} (D)	F_{max} (D)	V_{max} (D)	3/1	4/2	5/1	6/2
1	1	1.00	0.04	1.00	0.06	1.08	0.00	1.00	1.50	1.08	0.00
2	1	(2.16)	0.34	(2.02)	0.39	(2.41)	0.09	0.94	1.15	1.12	0.26
	2	1.00	0.05	1.00	0.09	1.08	0.00	1.00	1.80	1.08	0.00
3,4	1	-	1.33	-	1.34	-	1.19	-	1.01	-	0.89
	2	0.56	0.67	0.50	0.68	0.43	0.84	0.89	1.01	0.77	1.25
5	1	1.04	1.49	0.66	1.47	1.37	1.24	0.63	0.99	1.32	0.83
	2	1.06	1.09	1.02	1.11	1.25	1.03	0.96	1.02	1.18	0.94
	3	1.00	0.07	1.00	0.12	1.08	0.00	1.00	1.71	1.08	0.00
6	1	-	1.69	-	1.63	-	1.56	-	0.96	-	0.92
	2	0.87	1.11	0.68	1.17	0.76	1.24	0.78	1.05	0.87	1.12
	3	0.87	0.20	0.81	0.20	0.99	0.20	0.93	1.00	1.14	1.00
7,8	1	-	1.00	-	1.00	-	1.00	-	1.00	-	1.00
	2	-	1.57	-	1.59	-	1.55	-	1.01	-	0.99
	3	0.60	0.43	0.65	0.44	0.62	0.47	1.08	1.02	1.03	1.09
9	1	-	1.29	-	1.19	-	1.34	-	0.92	-	1.04
	2	0.45	1.83	0.38	1.80	0.55	1.82	0.84	0.98	1.22	0.99
	3	1.54	0.90	1.53	0.92	1.70	0.91	0.99	1.02	1.10	1.01
	4	1.00	0.07	1.00	0.12	1.06	0.00	1.00	1.71	1.06	0.00
10	1	-	1.00	-	1.00	-	1.00	-	1.00	-	1.00
	2	-	(1.92)	-	(1.83)	-	(1.97)	-	0.95	-	1.03
	3	1.16	0.92	0.96	0.96	1.21	0.96	0.83	1.04	1.04	1.04
	4	0.90	0.16	0.79	0.21	1.01	0.07	0.88	1.31	1.12	0.44

D = weight of concrete slab + weight of formwork for the tributary area of a given shore
F = shore load
V = slab load (shear force)
( ) = maximum load

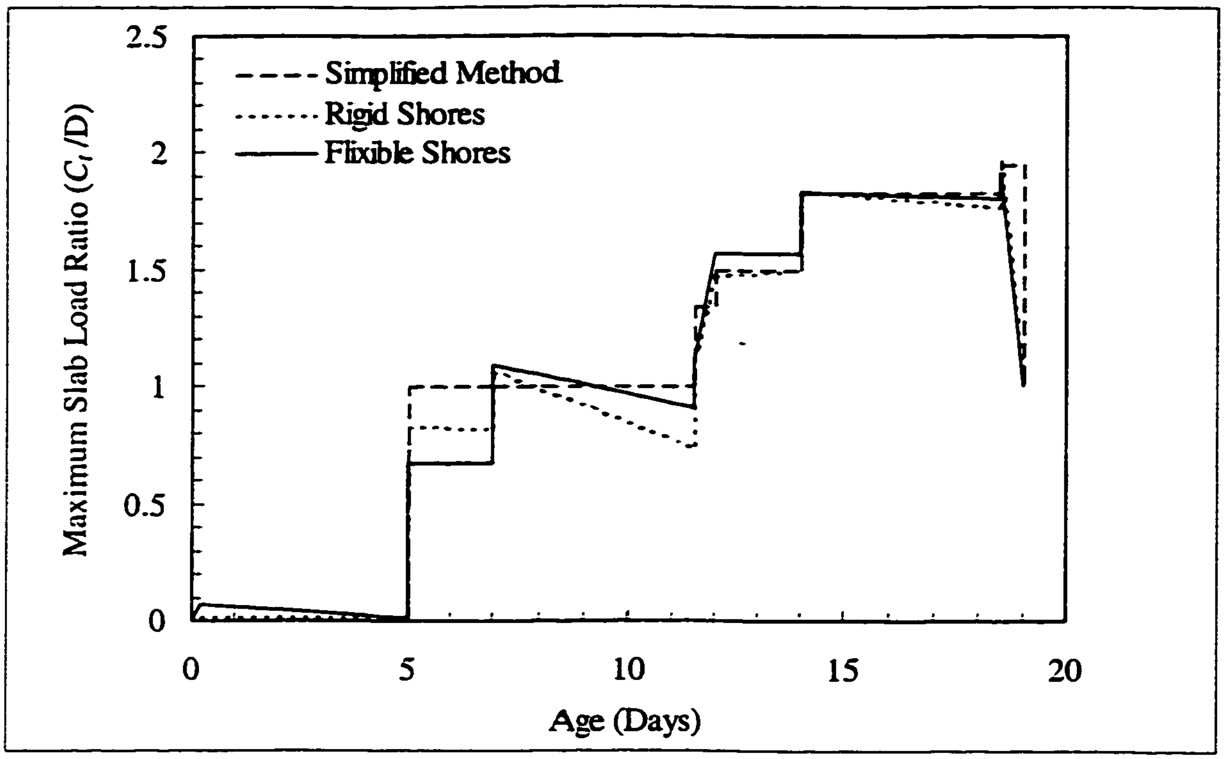


Fig. 5.3 Influence of Shore Stiffness on Slab Load Ratio

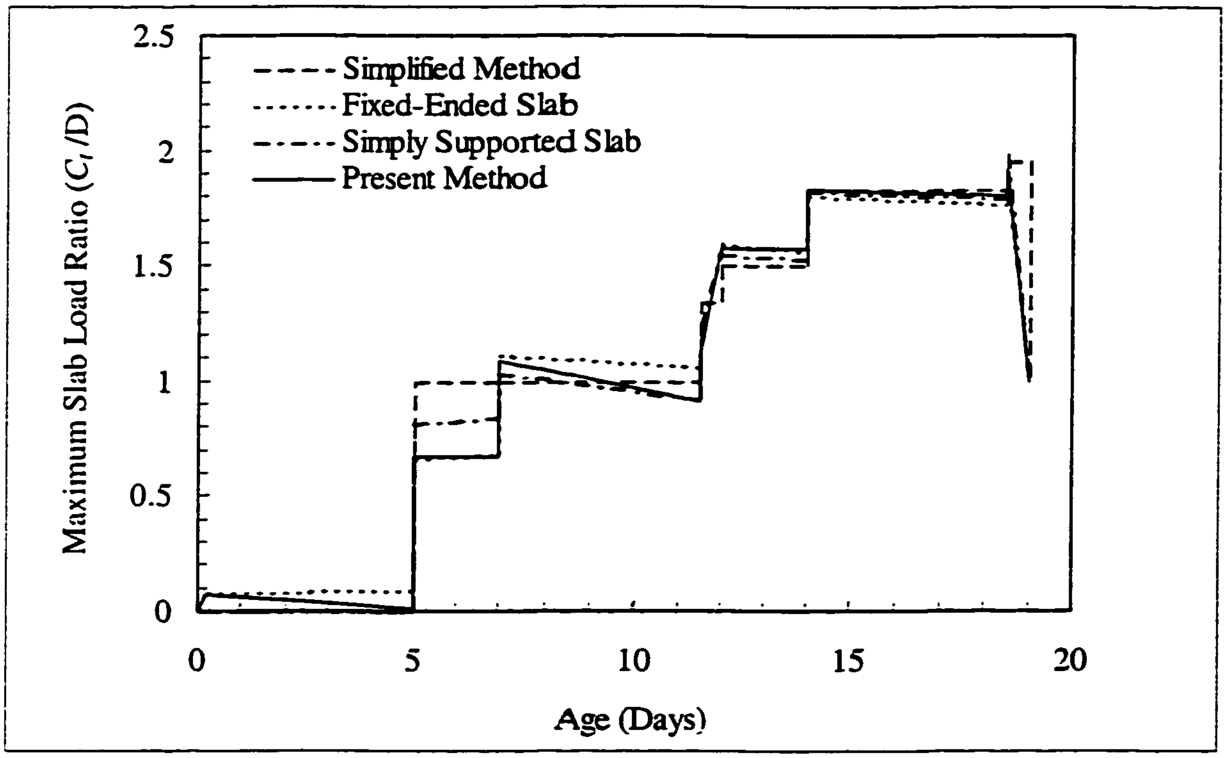


Fig. 5.4 Influence of Boundary Conditions on Slab Load Ratio

analysis of shores system in order to investigate its effect on construction load distribution.

From Table 5.5, it can be noted that the analysis including construction live load increases the absolute maximum slab and shore loads by 9% and 30%, respectively. Grundy-Kabila's simplified method gives the construction load distribution due to only dead load, but Hurd (1995) modified that by including construction live load in the analysis of shores system. The slab load histories with and without inclusion of construction live load predicted by the present method and the simplified method are shown in Fig. 5.5. Based on the above findings, the consideration of construction live load is important, especially, when calculating shores' loads. The simplified method agrees with the present method as to where and when the absolute maximum construction loads occur.

5.2.6 Influence of Construction Procedures

In order to investigate the influence of construction procedure on construction load distribution, the present analysis was carried out for two construction procedures: a) two levels of shores and one level of reshores; and b) three levels of shores. The analysis results of three levels of shores procedure are given in Table 5.6. The comparison of slab load histories using the above two procedures is shown in Fig. 5.6. In spite of the difference between construction steps of two procedures, the comparison is done regardless where and when the maximum construction loads would occur. The absolute maximum slab and shore loads using three levels of shores procedure are higher than that

Table 5.5 Influence of Construction Live Load on Slab Load Ratio

		Dead Load		Dead and Live Loads		Comparison	
		1	2	3	4	5	6
Step	Level	F_{max} (D)	V_{max} (D)	F_{max} (D)	V_{max} (D)	3/1	4/2
1	1	1.00	0.04	1.60	0.23	1.60	5.75
2	1	(2.16)	0.34	(2.80)	0.55	1.30	1.62
	2	1.00	0.05	1.60	0.25	1.60	5.00
3,4	1	-	1.33	-	1.40	-	1.05
	2	0.56	0.67	0.64	0.70	1.14	1.04
5	1	1.04	1.49	1.18	1.67	1.13	1.12
	2	1.06	1.09	1.53	1.38	1.44	1.27
	3	1.00	0.07	1.60	0.26	1.60	3.71
6	1	-	1.69	-	1.79	-	1.06
	2	0.87	1.11	0.98	1.19	1.13	1.07
	3	0.87	0.20	0.96	0.22	1.10	1.10
7,8	1	-	1.00	-	1.00	-	1.00
	2	-	1.57	-	1.66	-	1.06
	3	0.60	0.43	0.72	0.44	1.2	1.02
9	1	-	1.29	-	1.35	-	1.05
	2	0.45	1.83	1.82	(2.09)	4.04	1.14
	3	1.54	0.90	2.25	1.22	1.46	1.36
	4	1.00	0.07	1.60	0.26	1.60	3.71
10	1	-	1.00	-	1.00	-	1.00
	2	-	(1.92)	-	1.99	-	1.04
	3	1.16	0.92	1.66	0.99	1.43	1.08
	4	0.90	0.16	0.99	0.22	1.10	1.38

D = weight of concrete slab + weight of formwork for the tributary area of a given shore
F = shore load
V = slab load (shear force)
( ) = maximum load

Table 5.6 Construction Load Distribution for Three Levels of Shores Procedure

		Three Levels of Shores	
		1	2
Step	Level	F_{max} (D)	V_{max} (D)
1	1	1.00	0.04
2	1	2.24	0.34
	2	1.00	0.05
3	1	(3.12)	0.24
	2	1.96	0.44
	3	1.00	0.06
4	1	-	1.59
	2	0.88	1.00
	3	0.85	0.49
5	1	-	1.79
	2	0.90	1.31
	3	1.11	0.94
	4	1.00	0.08
6	1	-	1.00
	2	-	1.81
	3	1.06	1.03
	4	0.90	0.24
7	2	-	2.05
	3	1.14	1.29
	4	1.49	0.69
	5	1.00	0.06
8	2	-	1.00
	3	-	1.89
	4	0.98	0.92
	5	0.84	0.21
9	3	-	(2.11)
	4	1.15	1.22
	5	1.36	0.68
	6	1.00	0.06
10	3	-	1.00
	4	-	1.86
	5	0.93	0.92
	6	0.80	0.21

D = weight of concrete slab + weight of formwork for the tributary area of a given shore
F = shore load
V = slab load (shear force)
( ) = maximum load

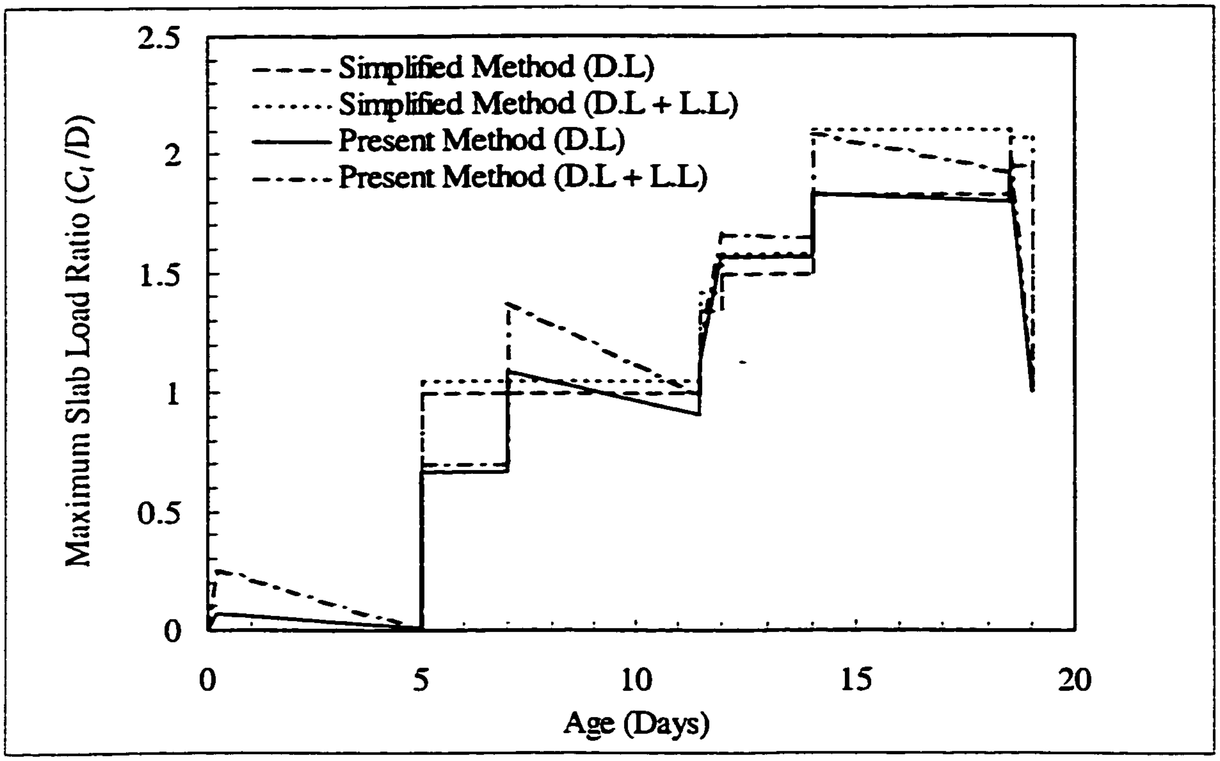
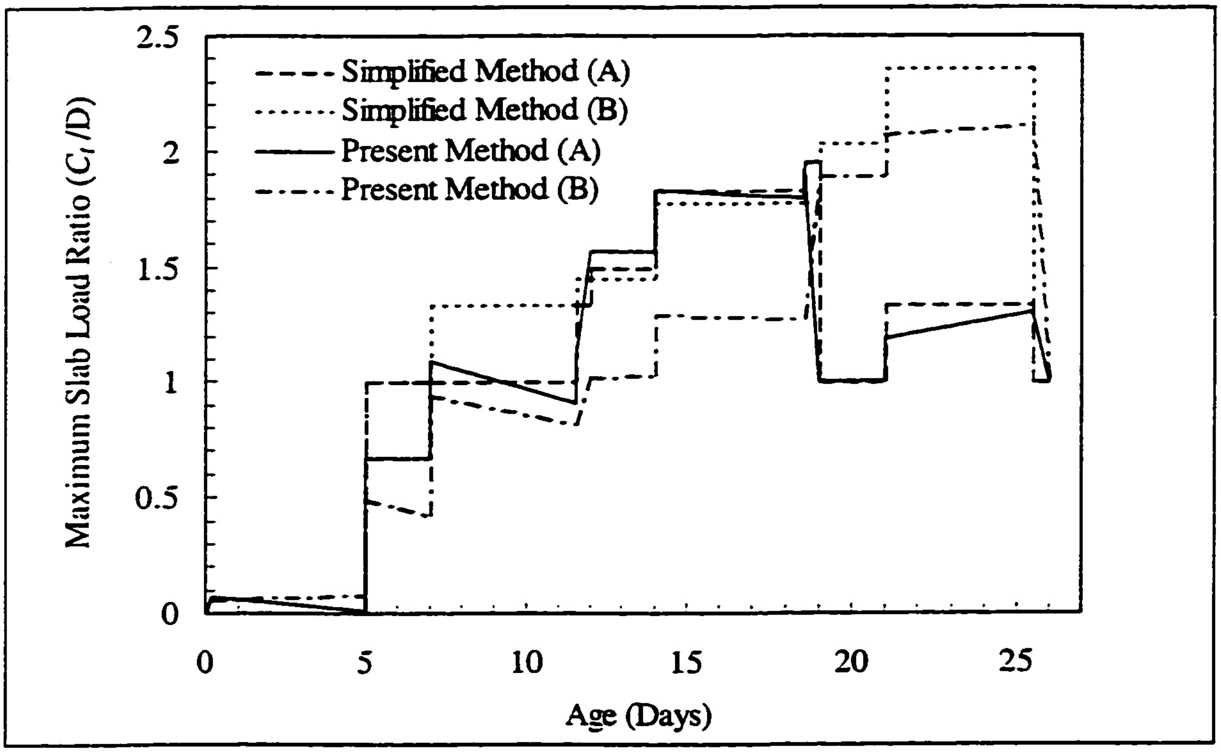


Fig. 5.5 Influence of Construction Live Load on Slab Load Ratio



A = two levels of shores and one level of reshores; B = three levels of shores

Fig. 5.6 Influence of Construction Procedure Type on Slab Load Ratio

using two levels of shores and one level of reshores procedure by 10% and 44%, respectively. From Fig. 5.6, it can be seen that both the simplified and present methods show that the type of construction procedure has an important influence on construction load distribution.

5.2.7 Influence of Using Slab Moment for Determining the Slab Loads

In all previous analyses, the present method used shear force for determining the slab loads. The comparison of the analysis used shear forces and slab moment was conducted in order to investigate their effect on estimating the slab loads. The results are shown in Table 5.7 and Fig. 5.7. The slab moments shown in Table 5.7 have been normalized with respect to the same slab subjected to a uniformly distributed dead load at the corresponding locations.

Based on the results obtained, it can be concluded that the absolute maximum slab load calculated based on slab moment is higher than that based on shear force by 20% and occurs at step 9. The results also show that the simplified method is very close to the present method when using the shear force. Thus when reporting slab loads one should state clearly whether these loads are calculated based on moment or shear.

5.3 Wall Formwork

The parameters examined are concrete placement rate, concrete temperature, wall thickness, form tie stiffness, form stiffness, neglecting the concrete part in the analysis, concrete density, amount of reinforcement, and load pattern.

Table 5.7 Influence of Using Slab Moment for Determining Slab Loads on Slab Load Ratio

		Slab Load		Comparison
		1	2 -	3
Step	Level	V_{max} (D)	M_{max} (D)	1/2
1	1	0.04	0.13	3.23
2	1	0.34	0.34	1.00
	2	0.05	0.20	4.00
3,4	1	1.33	1.28	0.96
	2	0.67	0.96	1.43
5	1	1.49	1.46	0.98
	2	1.09	1.19	1.09
	3	0.07	0.16	2.29
6	1	1.69	1.60	0.95
	2	1.11	1.31	1.18
	3	0.20	0.28	1.40
7,8	1	1.00	1.00	1.00
	2	1.57	1.74	1.11
	3	0.43	0.47	1.09
9	1	1.29	1.43	1.11
	2	1.83	(2.31)	1.26
	3	0.90	0.84	0.93
	4	0.07	0.18	2.57
10	1	1.00	1.00	1.00
	2	(1.92)	1.90	0.99
	3	0.92	1.18	1.28
	4	0.16	0.23	1.44

D = weight of concrete slab + weight of formwork for the tributary area of a given shore
M = slab load (moment)
V = slab load (shear force)
( ) = maximum load

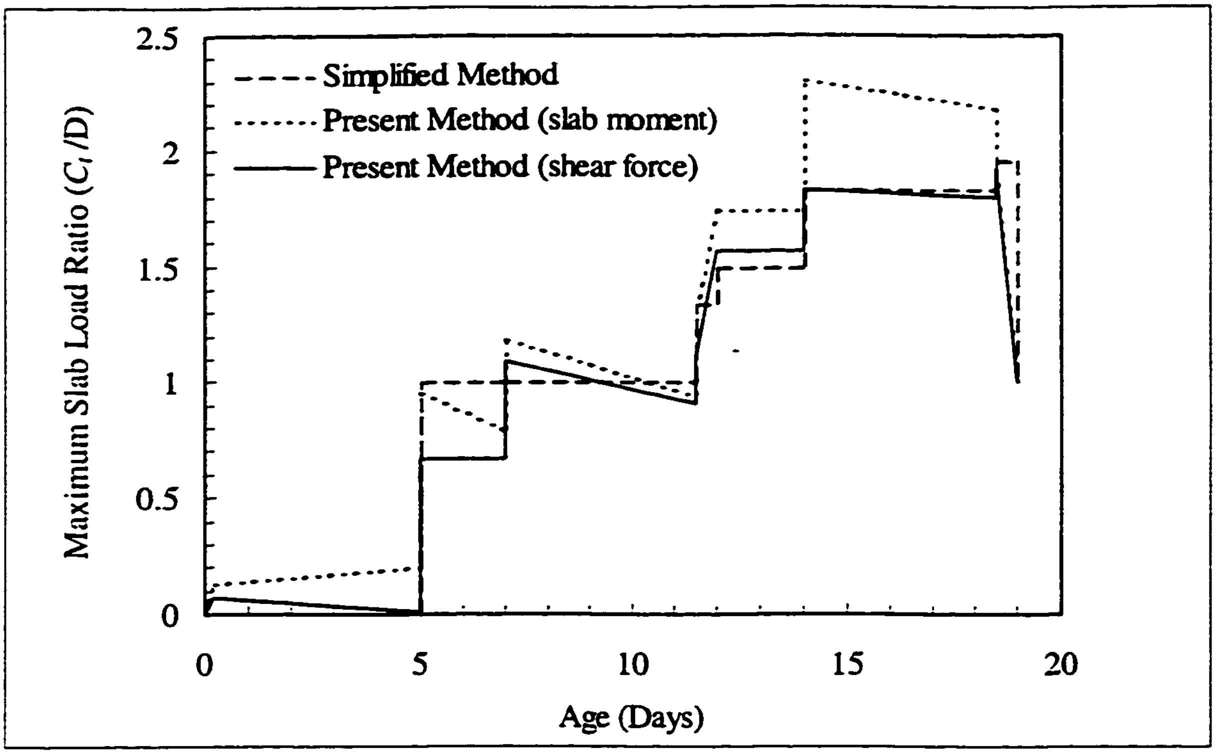


Fig. 5.7 Influence of Using Slab Moment for Determining Slab Loads on Slab Load Ratio

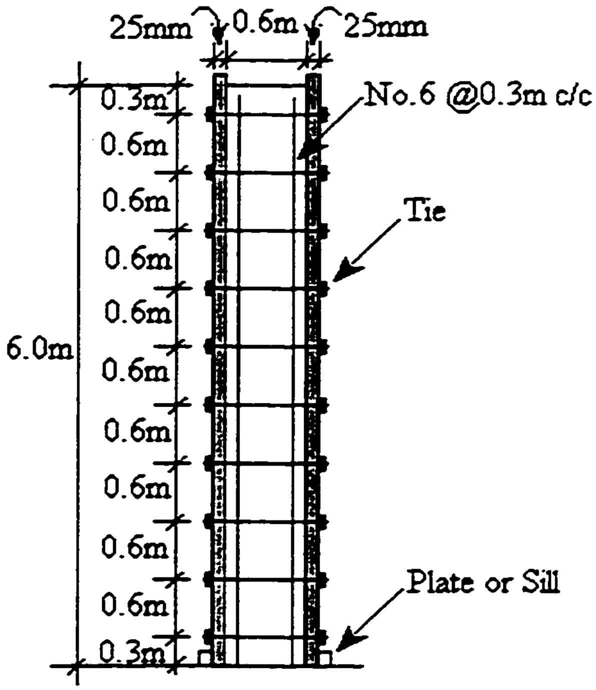


Fig. 5.8 2-D Concrete Wall Model

The results of concrete lateral pressure at placing the last lift are presented in tabular and graphical forms, while the results of the maximum lateral pressure on every tie are presented only in tabular forms.

The basic configuration of 2-D concrete wall model is shown in Fig. 5.8. The wall has a height of 6.0m (19.7ft), thickness of 0.6m (2ft.), and length of 6m (19.7ft.). Form faces are 25mm (1in.) steel panels supported by ties. The form ties consist of 38mm (1.5in.) diameter she-bolts with a 25mm (1in.) diameter coil threaded inner rod. The wall is reinforced on each face with No.6 reinforcing bars at a cover of 61mm (2.4in.) and spaced 300mm(12in.) on centre both horizontally and vertically. The fresh concrete having 22.90 kN/m^3 (146 pcf) density and 17.22 ^{o}C (63.0 ^{o}F) is placed in lifts of 0.6m (2ft.) at average rate of 1.74m/hr (5.7ft/hr). The placed concrete is consolidated using internal vibrators of 62.5mm (2.5in.) head diameter and 0.5m (19in.) head length. The modulus of elasticity, E_c, and the 28-day cylinder strength of concrete, f_c', for the wall are 2.6 x 10^4MPa (3.8 x 10^6psi) and 28MPa (4000psi), respectively.

5.3.1 Influence of Concrete Placement Rate

The actual rate of pour, R = 1.74m/hr (5.7ft/hr), was increased to double its value, R = 3.48m/hr (11.42ft/hr), in order to check the effect of this change on concrete lateral pressure. Comparison of the results is shown in Table 5.8 and Fig. 5.9. The analysis using R = 3.48m/hr increases the maximum lateral pressure by 26%. The maximum lateral pressure occurred at a form height of 1.5m (4.92ft.) from bottom for both cases, and when the total concrete height from bottom was 5.1m (16.73ft.) and 5.7m (18.70ft.) for the analyses using R = 1.74m/hr and R = 3.48m/hr, respectively. From the above results, it can be concluded that the concrete placement rate has significant effect on the estimation of concrete lateral pressure.

5.3.2 Influence of Concrete Temperature

The temperature of concrete has an important influence on lateral pressures as it affects the setting time of concrete (Hurd, 1995). In other word, the lateral pressure decreases as the concrete temperature increases and vice versa. In order to investigate the effect of temperature on concrete lateral pressure, two different concrete temperatures, 17.22 ^{o}C (63 ^{o}F) and 28.00 ^{o}C (28 ^{o}F), were used in the analysis. The results of analyses are compared in Table 5.9 and Fig. 5.10. Both cases show consistency for estimating the lateral pressure. Based on the obtained results, it can be seen that the concrete temperature has insignificant effect on concrete lateral pressure.

This can be attributed to the fact that the present method does not consider the influence of concrete temperature change in the first process (initial pressure envelope), which accounts for the effect of concrete setting time. But, the effect of temperature on concrete strength is considered by the present method in the second process (restrain of lateral pressure), which has a little effect on the concrete lateral pressure.

5.3.3 Influence of Wall Thickness

In order to investigate the effect of wall thickness of form on the lateral pressure, the wall thickness of d = 0.6m (2ft.), was increased to d = 1.20m (4ft.). The results of the analyses

Table 5.8 Influence of Concrete Placement Rate on Lateral Pressure

	Placement Rate, R = 1.74 m/hr		Placement Rate, R = 3.48 m/hr		Comparison	
	1	2	3	4	5	6
Form Height (m)	Maximum Lateral Pressure (kN/m^2)	Lateral Pressure at placing the last lift (kN/m^2)	Maximum Lateral Pressure (kN/m^2)	Lateral Pressure at placing the last lift (kN/m^2)	3/1	4/2
0.00	41.80	41.27	52.38	51.96	1.25	1.26
0.30	41.80	41.27	52.38	51.96	1.25	1.26
0.90	60.72	59.95	76.22	75.77	1.26	1.26
1.50	(62.42)	61.92	(78.45)	78.40	1.26	1.27
2.10	60.07	60.00	75.27	75.27	1.25	1.25
2.70	59.07	59.07	70.88	70.88	1.20	1.20
3.30	56.07	56.07	61.87	61.87	1.10	1.10
3.90	47.73	47.73	48.80	48.80	1.02	1.02
4.50	34.95	34.95	34.48	34.48	0.99	0.99
5.10	20.75	20.75	20.33	20.33	0.98	0.98
5.70	6.88	6.88	6.75	6.75	0.98	0.98
6.00	0.00	0.00	0.00	0.00	-	-

( ) = maximum pressure

Table 5.9 Influence of Concrete Temperature on Lateral Pressure

	Temperature, T = 17.22 °C		Temperature, T = 28.00 °C		Comparison	
	1	2	3	4	5	6
Form Height (m)	Maximum Lateral Pressure (kN/m^2)	Lateral Pressure at placing the last lift (kN/m^2)	Maximum Lateral Pressure (kN/m^2)	Lateral Pressure at placing the last lift (kN/m^2)	3/1	4/2
0.00	41.80	41.27	40.80	40.27	0.98	0.98
0.30	41.80	41.27	40.80	40.27	0.98	0.98
0.90	60.72	59.95	60.17	59.45	0.99	0.99
1.50	(62.42)	61.92	(62.67)	62.22	1.00	1.00
2.10	60.07	60.00	60.30	60.28	1.00	1.00
2.70	59.07	59.07	59.02	59.02	1.00	1.00
3.30	56.07	56.07	55.85	55.85	1.00	1.00
3.90	47.73	47.73	47.58	47.58	1.00	1.00
4.50	34.95	34.95	34.97	34.97	1.00	1.00
5.10	20.75	20.75	20.83	20.83	1.00	1.00
5.70	6.88	6.88	6.97	6.97	1.01	1.01
6.00	0.00	0.00	0.00	0.00	-	-

( ) = maximum pressure

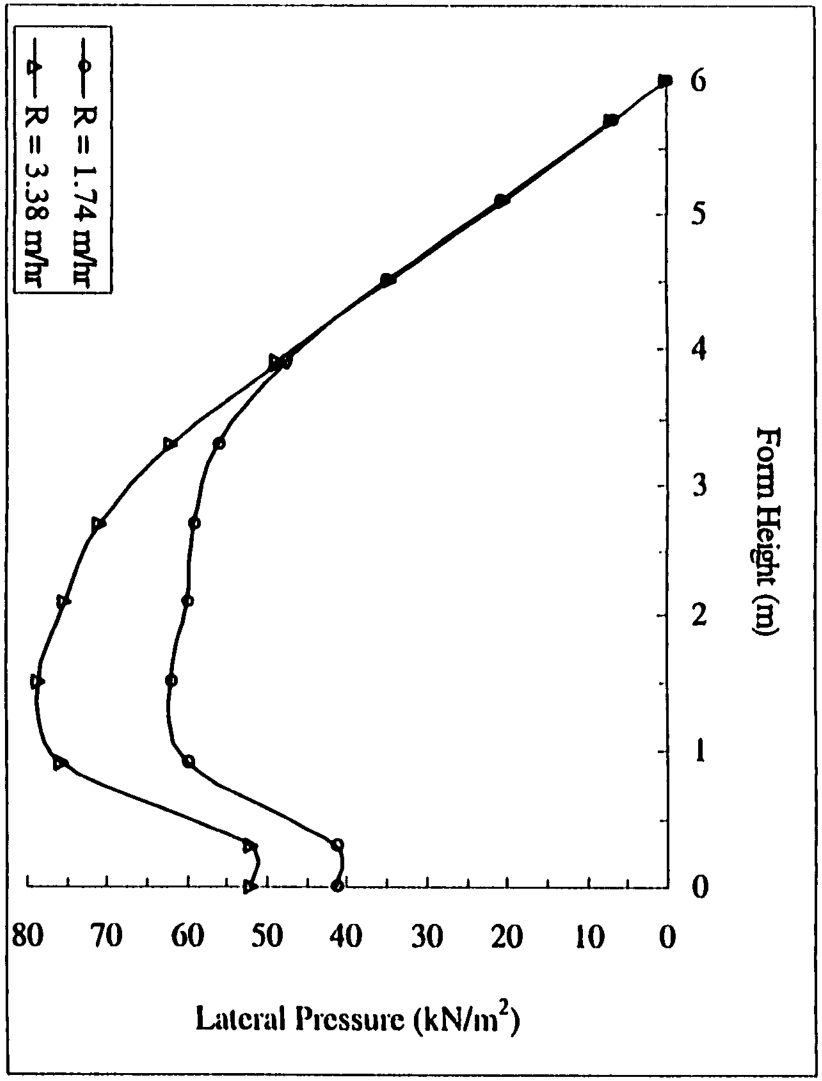


Fig. 5.9 Influence of Concrete Placement Rate

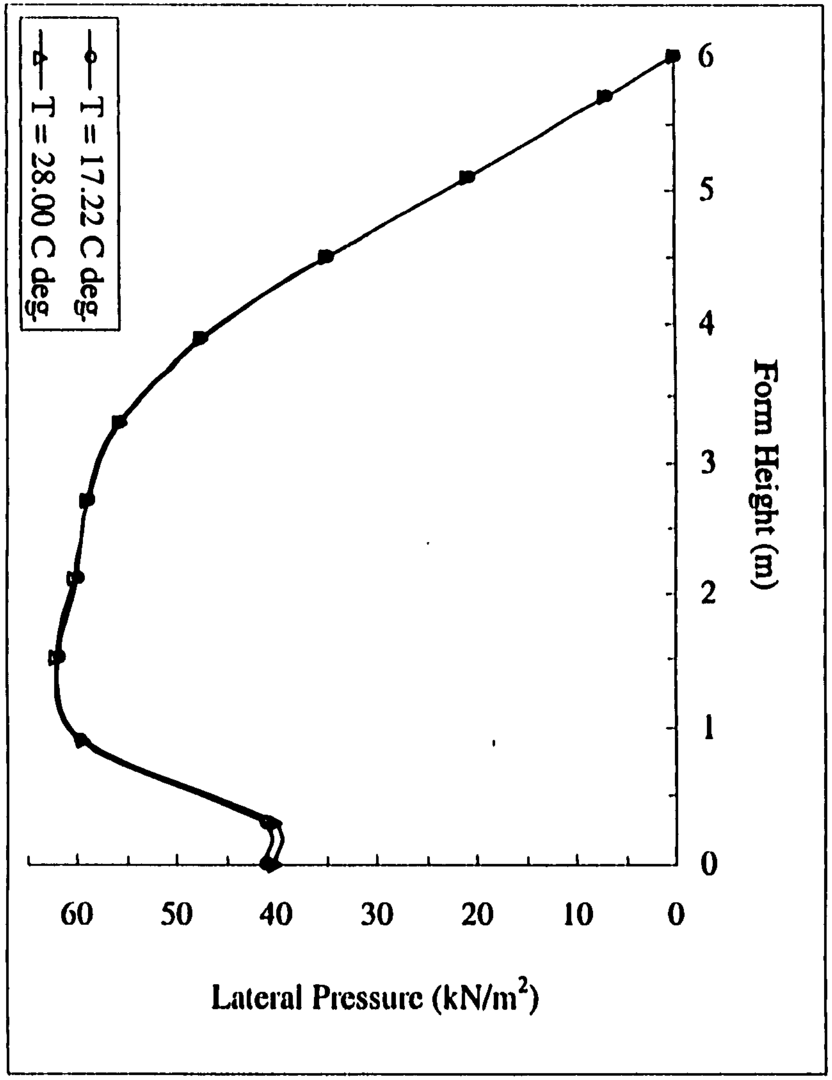


Fig. 5.10 Influence of Concrete Temperature

are compared in Table 5.10 and Fig. 5.11. The maximum lateral pressure of 1.2m thick wall thick is higher than that on the 0.6m thick wall by 2%, and occurred at a form height on the 2.10m (6.9ft.) from bottom when the total concrete height was 6.0m from the bottom. This increase in the maximum lateral pressure may be higher for walls of larger height. Based on the above findings, the wall thickness has little effect on concrete lateral pressure.

5.3.4 Influence of Form Tie Stiffness

The influence of form tie stiffness is examined by using two different ties varying in diameter. The results of comparison are shown in Table 5.11 and Fig. 5.12. Both cases show consistency in estimation of lateral pressure. Based on these results, it can be noted that the tie stiffness has insignificant effect on concrete lateral pressure.

5.3.5 Influence of Form Stiffness

The form thickness, $t_f' = 25$mm (1in.), was increased to the double, $t_f' = 50$mm (2in.), in order to check the influence of form stiffness on lateral pressure. Analysis results are compared in Table 5.12 and Fig. 5.13. Both cases agree on the location and time where and when the maximum lateral pressures occurred. The analysis using 50mm thick form yielded a slightly higher (2%) maximum lateral pressure. Based on observations, it can be concluded that the form stiffness has little effect on concrete lateral pressure.

Table 5.10 Influence of Wall Thickness on Lateral Pressure

	Wall Thickness, $d = 0.60m$		Wall Thickness, $d = 1.20m$		Comparison	
	1	2	3	4	5	6
Form Height (m)	Maximum Lateral Pressure (kN/m^2)	Lateral Pressure at placing the last lift (kN/m^2)	Maximum Lateral Pressure (kN/m^2)	Lateral Pressure at placing the last lift (kN/m^2)	3/1	4/2
0.00	41.80	41.27	30.78	30.78	0.74	0.75
0.30	41.80	41.27	30.78	30.78	0.74	0.75
0.90	60.72	59.95	52.80	52.80	0.87	0.88
1.50	(62.42)	61.92	63.05	63.05	1.01	1.02
2.10	60.07	60.00	(63.57)	63.57	1.06	1.06
2.70	59.07	59.07	59.43	59.43	1.01	1.01
3.30	56.07	56.07	53.12	53.12	0.95	0.95
3.90	47.73	47.73	44.93	44.93	0.94	0.94
4.50	34.95	34.95	34.62	34.62	0.99	0.99
5.10	20.75	20.75	22.57	22.57	1.09	1.09
5.70	6.88	6.88	8.90	8.90	1.29	1.29
6.00	0.00	0.00	0.00	0.00	-	-

( ) = maximum pressure

Table 5.11 Influence of Form Tie Stiffness on Lateral Pressure

	Tie Diameter, $\phi_{tie} = 25.0mm$		Tie Diameter, $\phi_{tie} = 19.0mm$		Comparison	
	1	2	3	4	5	6
Form Height (m)	Maximum Lateral Pressure (kN/m^2)	Lateral Pressure at placing the last lift (kN/m^2)	Maximum Lateral Pressure (kN/m^2)	Lateral Pressure at placing the last lift (kN/m^2)	3/1	4/2
0.00	41.80	41.27	37.42	36.93	0.90	0.89
0.30	41.80	41.27	37.42	36.93	0.90	0.89
0.90	60.72	59.95	58.78	58.18	0.97	0.97
1.50	(62.42)	61.92	(63.18)	62.92	1.01	1.02
2.10	60.07	60.00	60.95	60.95	1.01	1.02
2.70	59.07	59.07	58.97	58.97	1.00	1.00
3.30	56.07	56.07	55.37	55.37	0.99	0.99
3.90	47.73	47.73	47.22	47.22	0.99	0.99
4.50	34.95	34.95	34.90	34.90	1.00	1.00
5.10	20.75	20.75	20.98	20.98	1.01	1.01
5.70	6.88	6.88	7.33	7.33	1.07	1.07
6.00	0.00	0.00	0.00	0.00	-	-

( ) = maximum pressure

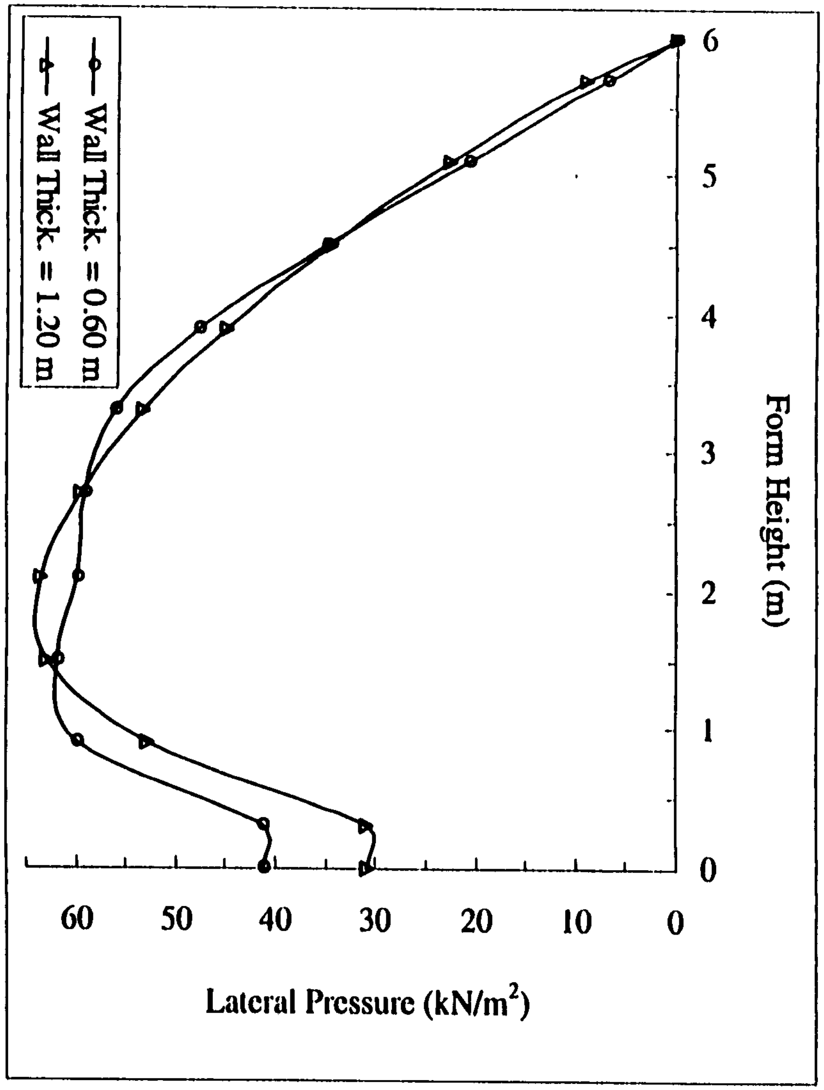


Fig. 5.11 Influence of Minimum Dimension of Form

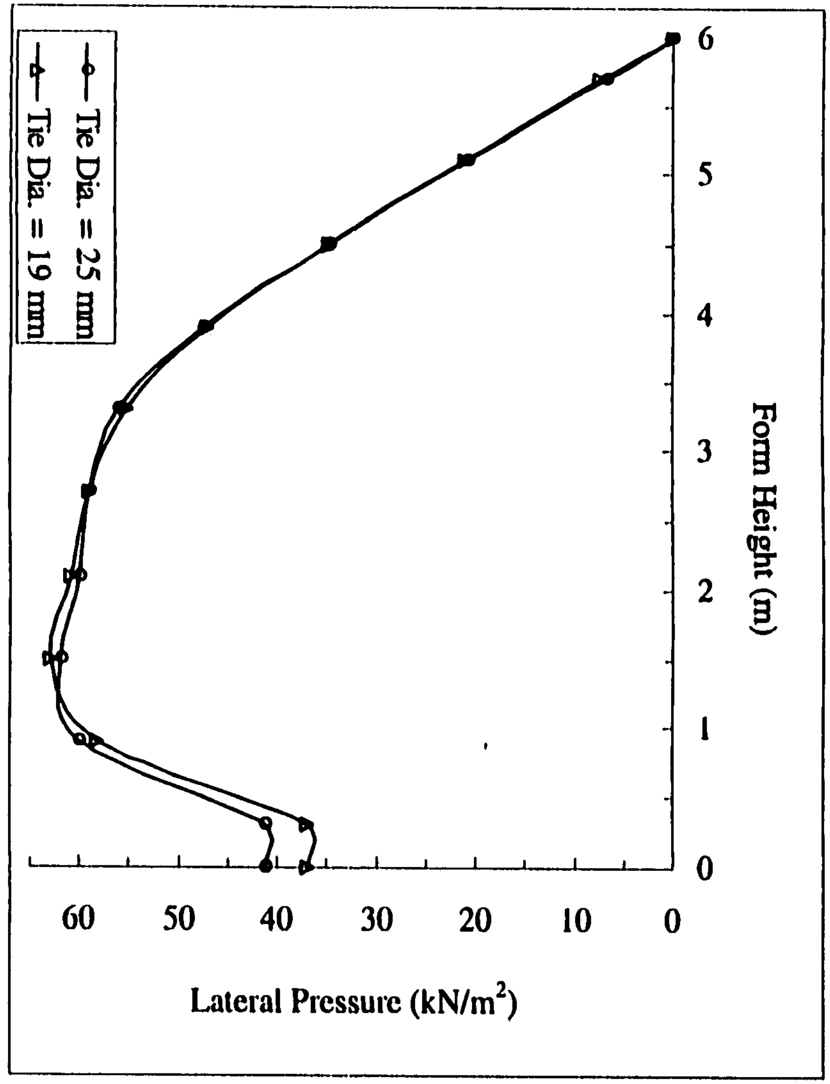


Fig. 5.12 Influence of Form Tie Stiffness

5.3.6 Influence of Neglecting the Concrete Part in the Analysis

In order to investigate that assumption, the analysis was done without considering the concrete part (inner part of composite wall). The results of comparison are shown in Table 5.13 and Fig. 5.14. The analysis without consideration of the concrete part produced a slightly higher (3%) maximum lateral pressure. The maximum lateral pressure occurred at a height of 0.30m (1ft.) from the form bottom, when the placed concrete level was at a height of 3.30m (10.8ft.) from the form bottom. Based on the above findings, it can be concluded that the concrete part has little effect on concrete lateral pressure.

5.3.7 Influence of Concrete Density

In order to investigate the effect of this parameter, two different concrete densities, $\gamma =$ 22.90kN/m^3 (146pcf) and $\gamma = 24.00$kN/m^3 (153pcf), were used in the analysis. Results of the comparison are shown in Table 5.14 and Fig. 5.15. The small increment in concrete density, 1.10kN/m^3 (7pcf), increased the maximum lateral pressure by 5%. Both cases agreed on the location and time where and when the maximum lateral pressures occurred. Based on the above observations, it can be concluded that the concrete density has significant effect on concrete lateral pressure.

5.3.8 Influence of Amount of Reinforcement

The present method considers the steel reinforcement in the analysis. In order to investigate that effect, the amount of reinforcement was increased to double its value in one case and neglected in another. The results of comparison are shown in Table 5.15 and

Table 5.12 Influence of Form Stiffness on Lateral Pressure

	Form Thickness, $t_f' = 25.0$mm		Form Thickness, $t_f' = 50.0$mm		Comparison	
	1	2	3	4	5	6
Form Height (m)	Maximum Lateral Pressure (kN/m^2)	Lateral Pressure at placing the last lift (kN/m^2)	Maximum Lateral Pressure (kN/m^2)	Lateral Pressure at placing the last lift (kN/m^2)	3/1	4/2
0.00	41.80	41.27	36.96	36.42	0.88	0.88
0.30	41.80	41.27	36.96	36.42	0.88	0.88
0.90	60.72	59.95	59.07	58.40	0.97	0.97
1.50	(62.42)	61.92	(63.42)	63.02	1.02	1.02
2.10	60.07	60.00	60.72	60.65	1.01	1.01
2.70	59.07	59.07	58.98	58.98	1.00	1.00
3.30	56.07	56.07	55.63	55.63	0.99	0.99
3.90	47.73	47.73	47.42	47.42	0.99	0.99
4.50	34.95	34.95	34.90	34.90	1.00	1.00
5.10	20.75	20.75	21.07	21.07	1.02	1.02
5.70	6.88	6.88	7.00	7.00	1.02	1.02
6.00	0.00	0.00	0.00	0.00	-	-

( ) = maximum pressure

Table 5.13 Influence of Neglecting the Concrete Part in the Analysis on Lateral Pressure

	Present Method		Without Concrete Part		Comparison	
	1	2	3	4	5	6
Form Height (m)	Maximum Lateral Pressure (kN/m^2)	Lateral Pressure at placing the last lift (kN/m^2)	Maximum Lateral Pressure (kN/m^2)	Lateral Pressure at placing the last lift (kN/m^2)	3/1	4/2
0.00	41.80	41.27	64.49	63.78	1.54	1.55
0.30	41.80	41.27	(64.49)	63.78	1.54	1.55
0.90	60.72	59.95	61.70	61.25	1.02	1.02
1.50	(62.42)	61.92	58.55	58.10	0.94	0.94
2.10	60.07	60.00	59.18	58.70	0.99	0.98
2.70	59.07	59.07	59.07	58.73	1.00	0.99
3.30	56.07	56.07	58.22	58.22	1.04	1.04
3.90	47.73	47.73	48.47	48.47	1.02	1.02
4.50	34.95	34.95	34.18	34.18	0.98	0.98
5.10	20.75	20.75	21.05	21.05	1.01	1.01
5.70	6.88	6.88	5.95	5.95	0.86	0.86
6.00	0.00	0.00	0.00	0.00	-	-

( ) = maximum pressure

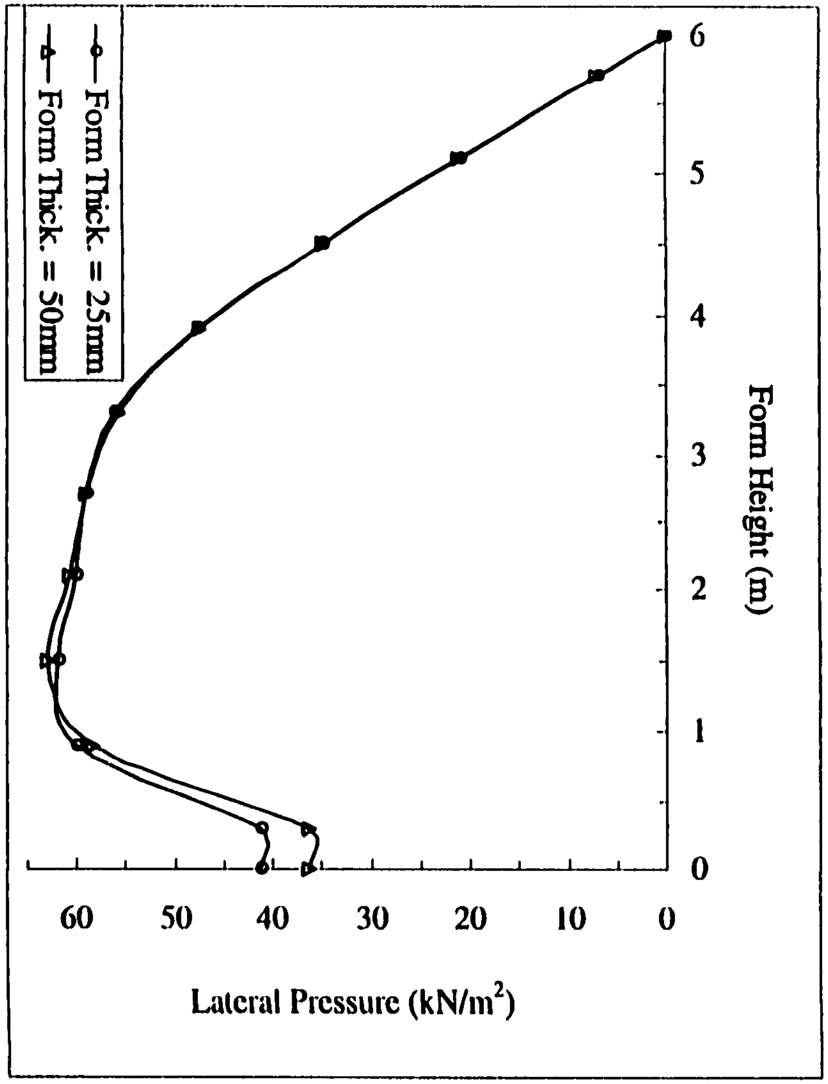


Fig. 5.13 Influence of Form Stiffness

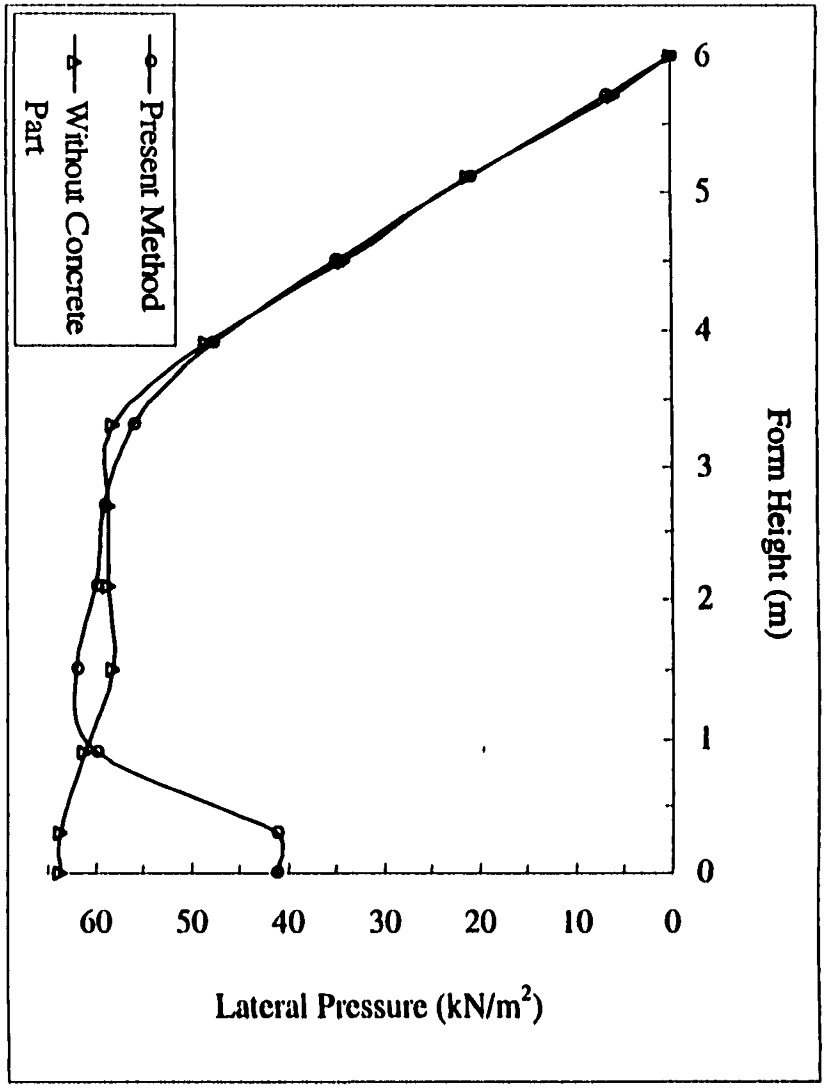


Fig. 5.14 Influence of Neglecting Concrete Part in the Analysis

Table 5.14 Influence of Concrete Density on Lateral Pressure

	Concrete Density, $\gamma = 22.90$ kN/m^3		Concrete Density, $\gamma = 24.00$ kN/m^3		Comparison	
	1	2	3	4	5	6
Form Height (m)	Maximum Lateral Pressure (kN/m^2)	Lateral Pressure at placing the last lift (kN/m^2)	Maximum Lateral Pressure (kN/m^2)	Lateral Pressure at placing the last lift (kN/m^2)	3/1	4/2
0.00	41.80	41.27	43.80	43.27	1.05	1.05
0.30	41.80	41.27	43.80	43.27	1.05	1.05
0.90	60.72	59.95	63.63	62.83	1.05	1.05
1.50	(62.42)	61.92	(65.42)	64.90	1.05	1.05
2.10	60.07	60.00	62.97	62.88	1.05	1.05
2.70	59.07	59.07	61.90	61.90	1.05	1.05
3.30	56.07	56.07	58.75	58.75	1.05	1.05
3.90	47.73	47.73	50.02	50.02	1.05	1.05
4.50	34.95	34.95	36.62	36.62	1.05	1.05
5.10	20.75	20.75	21.73	21.73	1.05	1.05
5.70	6.88	6.88	7.22	7.22	1.05	1.05
6.00	0.00	0.00	0.00	0.00	-	-

( ) = maximum pressure

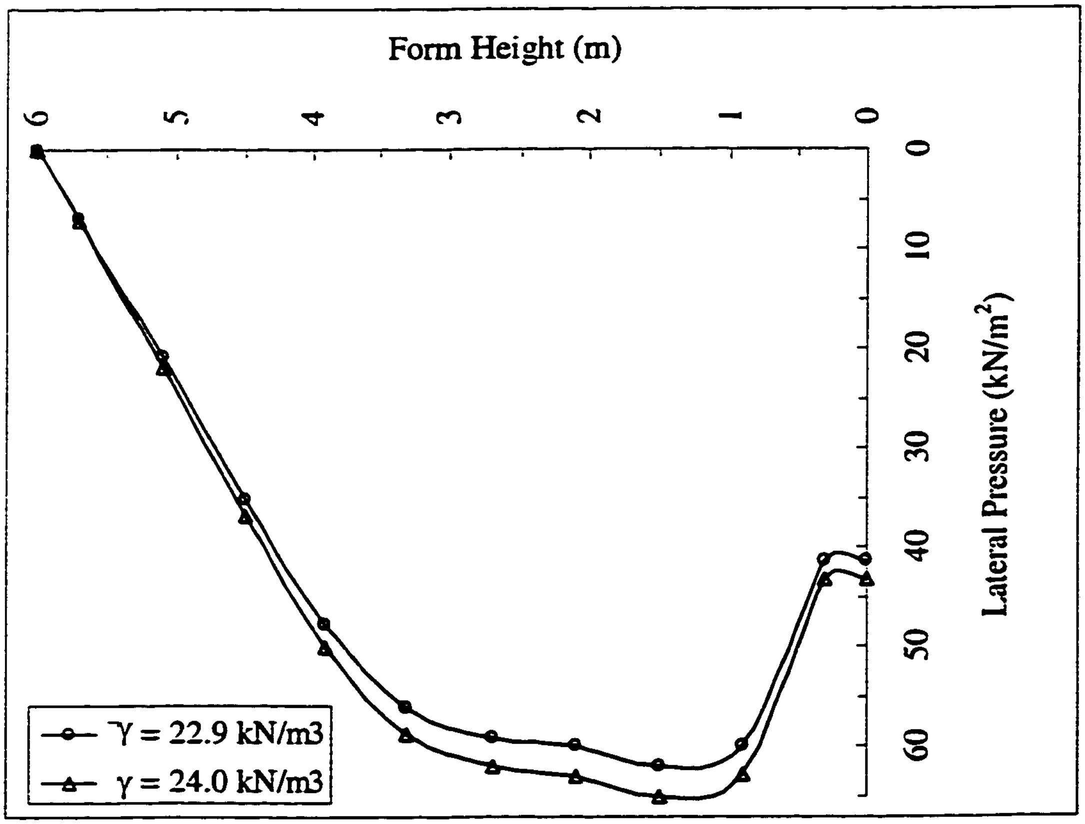


Fig. 5.15 Influence of Concrete Density

Table 5.15 Influence of the Amount of Reinforcement on Lateral Pressure

	Reference Amount of Reinforcement		Double Area of Steel Reinforcement		Zero Steel Reinforcement		Comparison			
	1	2	3	4	5	6	7	8	9	10
Form Height (m)	Maximum Lateral Pressure (kN/m^2)	Lateral Pressure at Placing the Last Lift (kN/m^2)	Maximum Lateral Pressure (kN/m^2)	Lateral Pressure at Placing the Last Lift (kN/m^2)	Maximum Lateral Pressure (kN/m^2)	Lateral Pressure at Placing the Last Lift (kN/m^2)	3/1	4/2	5/1	6/2
0.00	41.80	41.27	40.27	39.73	44.64	44.11	0.96	0.96	1.07	1.07
0.30	41.80	41.27	40.27	39.73	44.64	44.11	0.96	0.96	1.07	1.07
0.90	60.72	59.95	60.42	59.65	61.08	60.32	1.00	0.99	1.01	1.01
1.50	(62.42)	61.92	(62.62)	62.10	(62.08)	61.58	1.00	1.00	0.99	0.99
2.10	60.07	60.00	60.17	60.10	59.95	59.88	1.00	1.00	1.00	1.00
2.70	59.07	59.07	59.07	59.07	59.05	59.05	1.00	1.00	1.00	1.00
3.30	56.07	56.07	56.05	56.05	56.07	56.07	1.00	1.00	1.00	1.00
3.90	47.73	47.73	47.70	47.70	47.72	47.72	1.00	1.00	1.00	1.00
4.50	34.95	34.95	34.88	34.88	35.03	35.03	1.00	1.00	1.00	1.00
5.10	20.75	20.75	20.60	20.60	21.13	21.13	0.99	0.99	1.02	1.02
5.70	6.88	6.88	7.10	7.10	6.43	6.43	1.03	1.03	0.93	0.93
6.00	0.00	0.00	0.00	0.00	0.00	0.00	-	-	-	-

( ) = maximum pressure

Fig. 5.16. All cases show consistency in estimating the lateral pressure. Based on the above results, the amount of reinforcement has negligible effect on concrete lateral pressure.

5.3.9 Influence of Load Pattern

In the present analysis, the initial pressure envelope exerts loads on the composite wall structure. This action is a function of the behavior, characteristics, and placing rate of fresh concrete. Figures 5.17, 5.18 and 5.19 illustrate the three different load patterns used to investigate the influence of variation of load pattern on lateral pressure. All of three cases have the same pouring time. The results of comparison are shown in Table 5.16 and Fig. 5.20.

The maximum lateral pressure of load pattern due to low and high rate of pouring is higher than that due to average rate of pouring by 23%, and occurred at a height of 1.50m (4.92ft.) from form bottom when the total concrete height from the form bottom was 5.70m (18.7ft). The maximum lateral pressure of load pattern due to high and low rate of pouring is lower than that due to average rate of pouring by 5%, and occurred at a height of 0.90m (2.95ft.) from form bottom when the total concrete height from the form bottom was 4.50m (14.76ft.). Based on the above observations, the load pattern has significant effect on concrete lateral pressure.

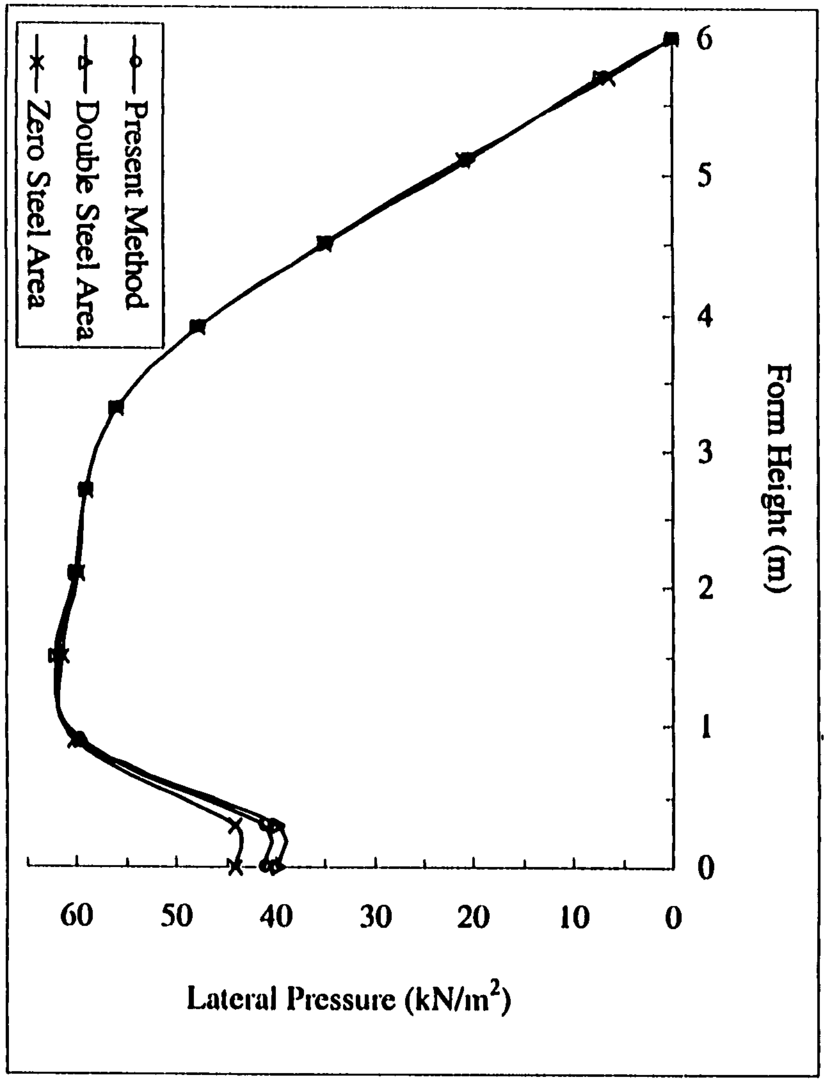


Fig. 5.16 Influence of the Amount of Reinforcement

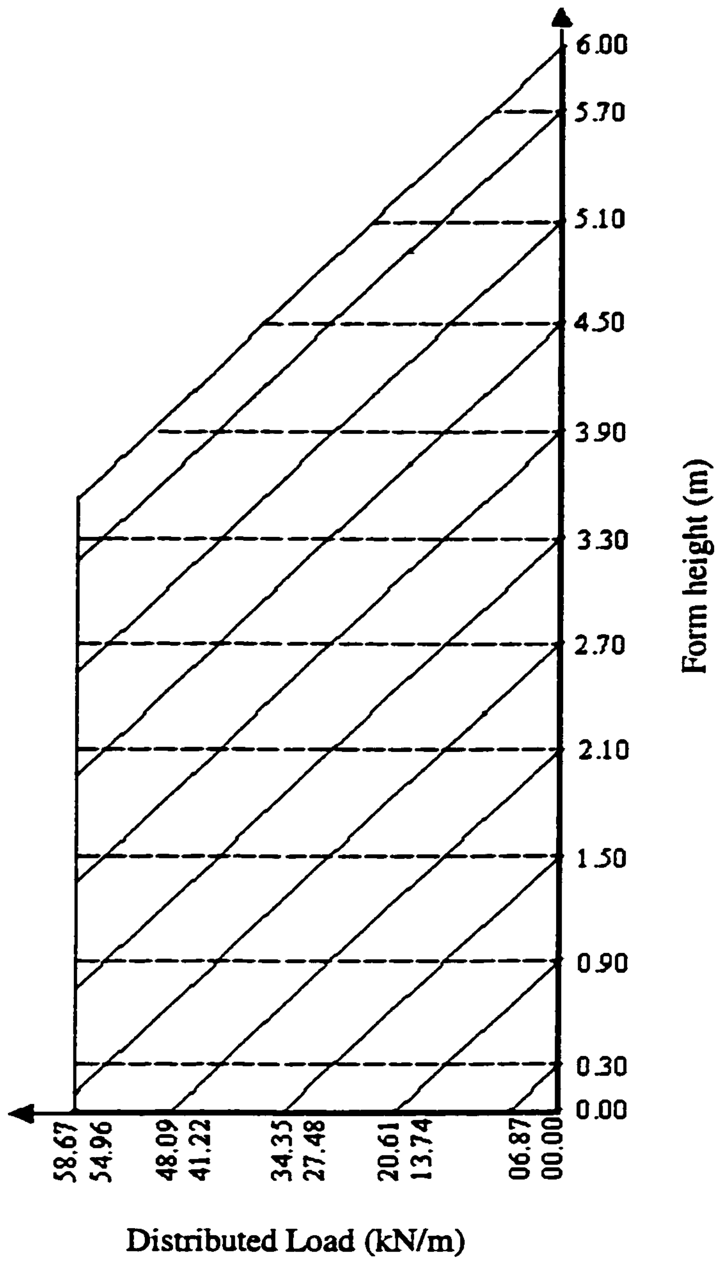


Fig. 5.17 Load Pattern Due to Average Rate of Pour

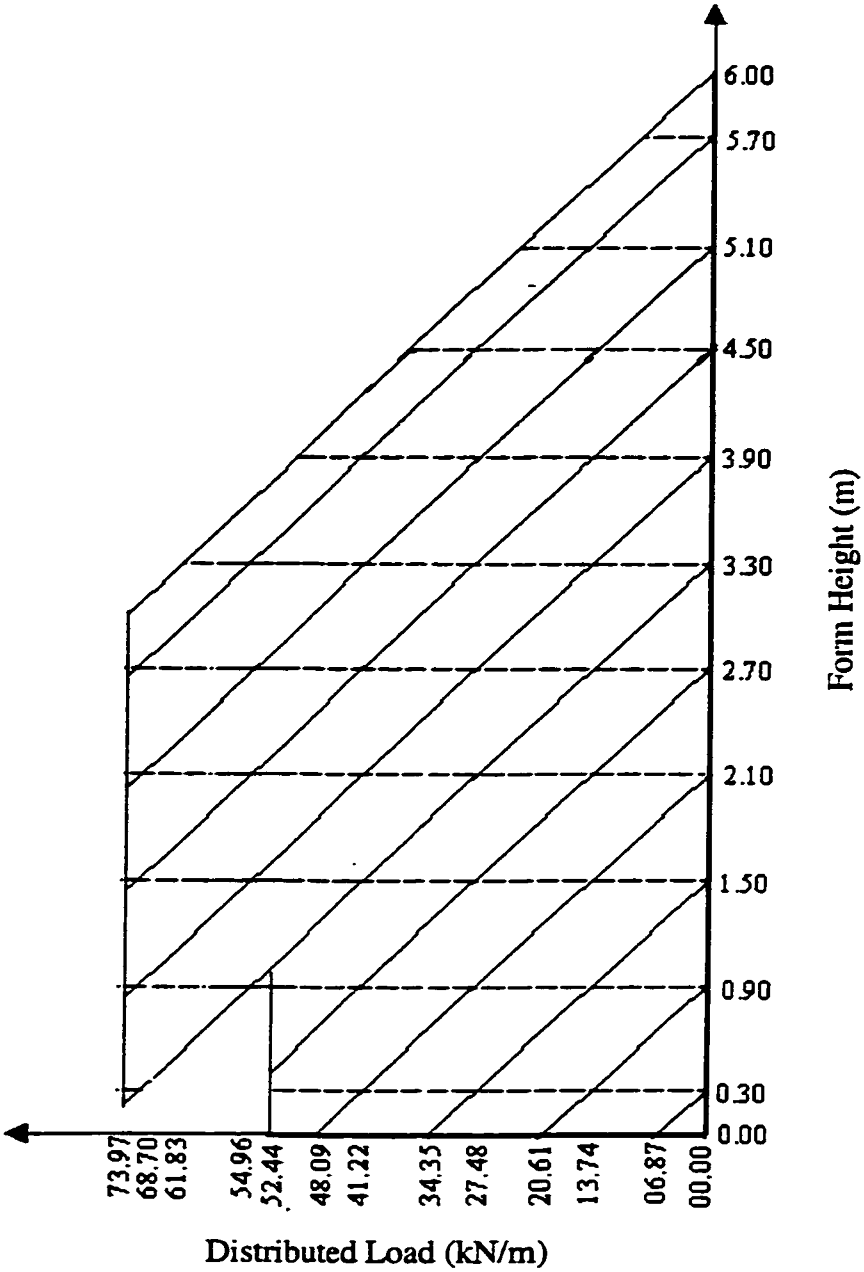


Fig. 5.18 Load Pattern Due to Low and High Rates of Pour

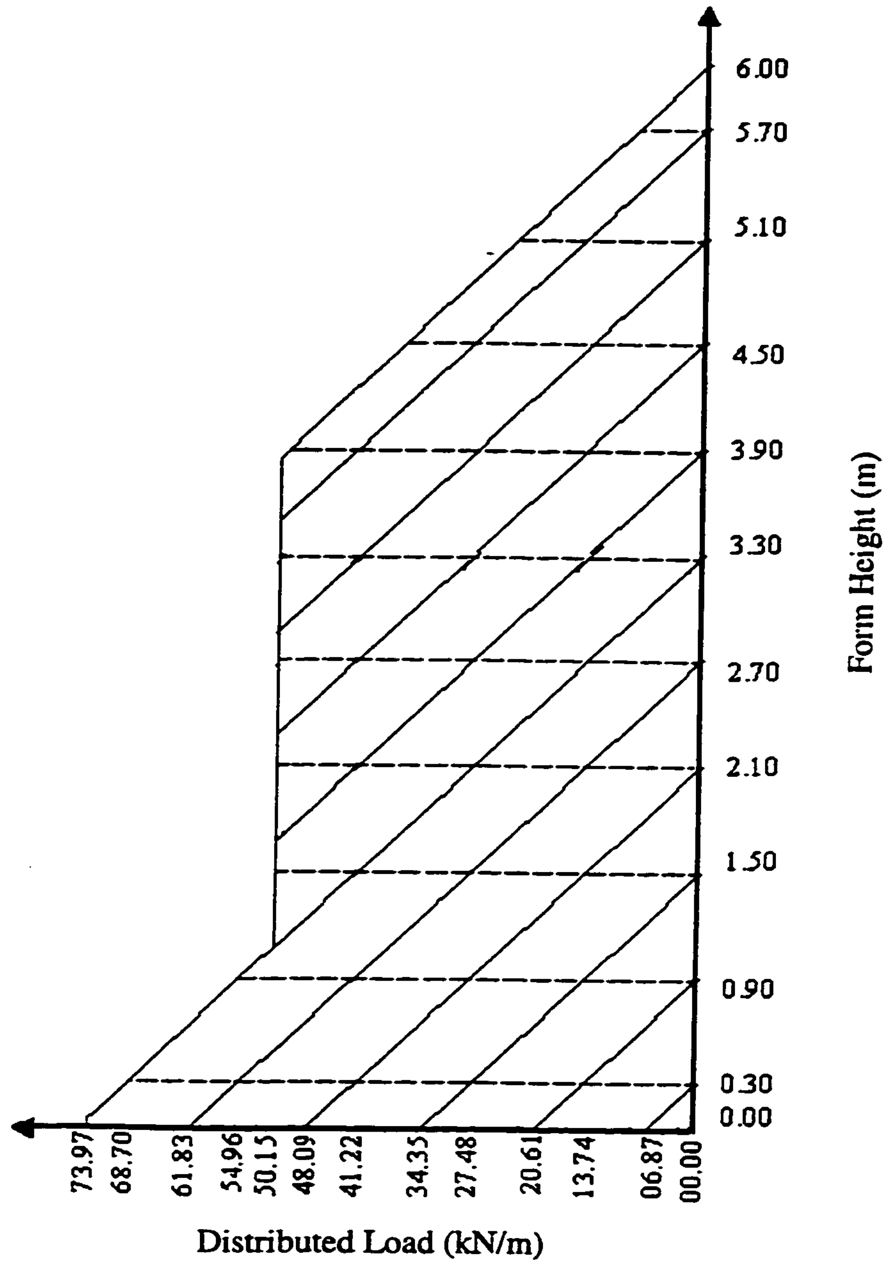


Fig. 5.19 Load Pattern Due to High and Low Rates of Pour

Table 5.16 Influence of Load Pattern on Lateral Pressure

	Average Rate of Pour, R = 1.74 m/hr		Low and High Rate of Pour, R = 1.24 and 3.48 m/hr		High and Low Rate of Pour, R = 3.48 and 1.08 m/hr		Comparison			
	1	2	3	4	5	6	7	8	9	10
Form Height (m)	Maximum Lateral Pressure (kN/m^2)	Lateral Pressure at Placing the Last Lift (kN/m^2)	Maximum Lateral Pressure (kN/m^2)	Lateral Pressure at Placing the Last Lift (kN/m^2)	Maximum Lateral Pressure (kN/m^2)	Lateral Pressure at Placing the Last Lift (kN/m^2)	3/1	4/2	5/1	6/2
0.00	41.80	41.27	45.56	45.18	44.11	43.47	1.09	1.09	1.06	1.05
0.30	41.80	41.27	45.56	45.18	44.11	43.47	1.09	1.09	1.06	1.05
0.90	60.72	59.95	70.32	69.97	(59.08)	58.27	1.16	1.17	0.97	0.97
1.50	(62.42)	61.92	(76.93)	76.90	55.40	54.77	1.23	1.24	0.89	0.88
2.10	60.07	60.00	75.52	75.52	51.17	51.02	1.26	1.26	0.85	0.85
2.70	59.07	59.07	71.37	71.37	50.43	50.43	1.21	1.21	0.85	0.85
3.30	56.07	56.07	62.02	62.02	49.75	49.75	1.11	1.11	0.89	0.89
3.90	47.73	47.73	48.82	48.82	44.92	44.92	1.02	1.02	0.94	0.94
4.50	34.95	34.95	34.48	34.48	34.58	34.58	0.99	0.99	0.99	0.99
5.10	20.75	20.75	20.33	20.33	21.13	21.13	0.98	0.98	1.02	1.02
5.70	6.88	6.88	6.75	6.75	7.10	7.10	0.98	0.98	1.03	1.03
6.00	0.00	0.00	0.00	0.00	0.00	0.00	-	-	-	-

( ) = maximum pressure

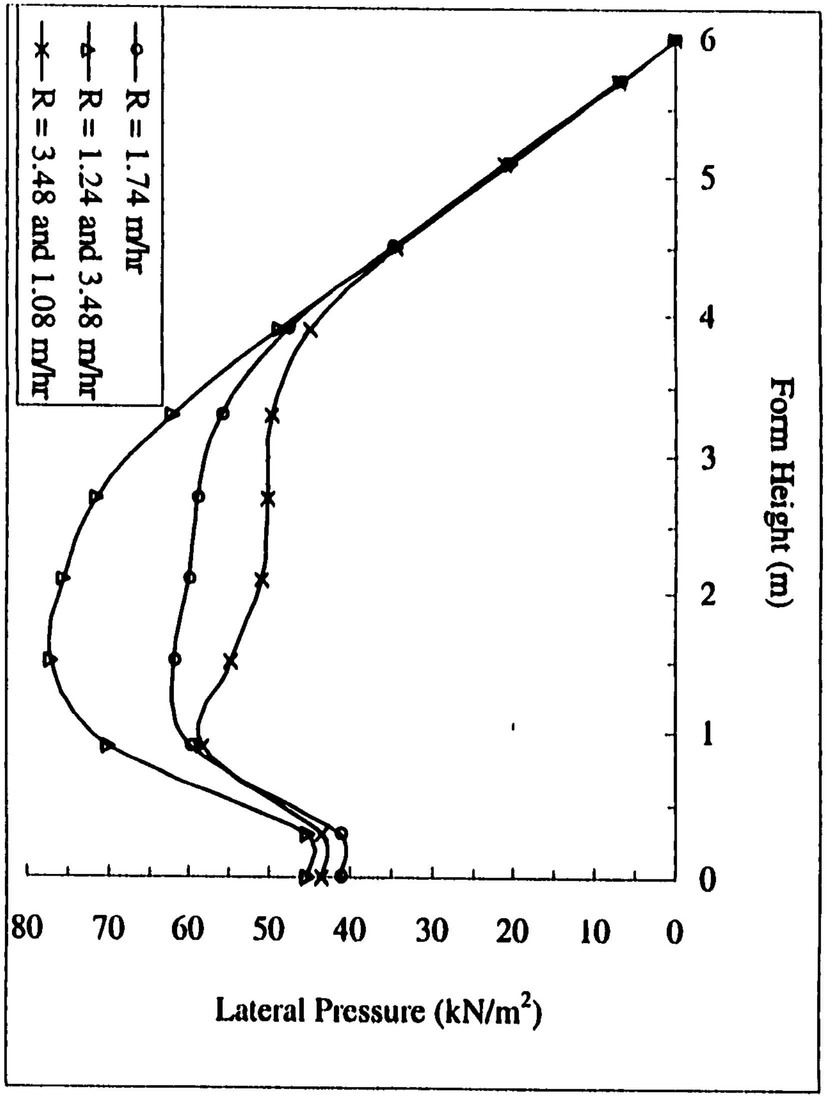


Fig. 5.20 Influence of Load Pattern

CHAPTER VI

SUMMARY, CONCLUSIONS AND RECOMENDATIONS

6.1 Summary

Over the past years, numerous efforts have been conducted in attempt to find or improve the usable procedures needed for determining the load distribution among shores and/or reshores and interconnected slabs during the construction of multistory concrete buildings and the concrete lateral pressure on vertical formwork during concrete placing. Grunday-Kabaila's simplified method has become the basis for most other developed methods used in shoring system. These methods employed either 2-D or 3-D computer-aided models for single-bay and/or multi-bay frames. Based on experimental data several recommended and proposed procedures for empirically estimating concrete lateral pressures on wall formwork have been developed. Each method assumes that for design purposes the envelope of the lateral pressure exerted by fresh concrete against a vertical form face can be considered as hydrostatic from free surface to a maximum value and thereafter constant at the maximum value. However, the value of maximum pressure varies significantly with a variation of considered parameters in those methods.

This study was aimed at improving the understanding of formwork behavior during the construction of concrete structures and the developing of practical computer-based models to determine the construction load distribution between shoring system and interconnected slabs of multistory concrete buildings, the lateral pressures exerted by fresh concrete on wall formwork. The computer program CPF is a reasonable analytical tool for accomplishing these goals because it has wide range of applicability for idealization of both the shoring system and the wall formwork models, and provides instantaneous and time-dependent analyses. The analysis by CPF accounts for time-dependent effects caused by creep and shrinkage of concrete and for the effects of sequence of construction, loading and change in geometry and support conditions and addition or removal of structural elements. The maturity-based model proposed by Gross and Lew (1986) was also employed in the present investigation to determine the development of concrete strength for both the shoring system and the wall formwork models.

For shoring systems, 2-D model employed single-bay frame idealization has been developed in order to determine the load distribution among shores and/or reshores and interconnected slabs during the construction of multistory concrete buildings. The shoring system model was checked against the field measurements (Agarwal and Gardner, 1974), the simplified method (Grundy and Kabaila, 1963), the refined method (Liu, et al., 1986), the improved simplified method (Duan and Chen, 1995a) and the 2-D and 3-D methods advanced by Mosallam and Chen (1991 and 1992) to verify the present method and to investigate the range of applicability of the others. A parametric study was carried out to determine the influence of different factors on load distribution of a multistory concrete building under construction. The examined parameters are: neglecting the change in construction load during construction cycles; slab stiffness; shore stiffness; slab boundary conditions; construction live load; construction procedure; and use of slab moments for determining slab loads.

In order to determine the concrete lateral pressure on wall formwork during concrete placing, 2-D model has been also developed. The wall-formwork model idealization is based on two main processes acting against each other. The initial pressure envelope represents the first process, which is a function of the behavior, characteristics and placing rate of fresh concrete. The second process is the restrain of the lateral forces that is a function of the rate of concrete shear strength development and the characteristics of form elements. Wall formwork model was checked against the field measurements (Dunston, Johnston and McCain, 1994) and (Johnston, Khan and Philips, 1989), Rodin's method (Rodin, 1953), ACI method (ACI Committee 347R-94, 1994), CIRIA method (Clear and Harrison, 1985) and Gardner's method (Gardner, 1980) in order to verify the present method and to investigate the assumptions and limitations of the others. A parametric study was carried out to investigate the influence of different factors on the concrete lateral pressure estimation during concrete placing into the wall forms. The examined parameters are: concrete placement rate; concrete temperature; minimum dimension of form; form tie stiffness; form stiffness; neglecting the concrete part in the analysis; concrete density; amount of reinforcement; and load pattern.

6.2 Conclusions

Based on the present study, the following conclusions and observations can be stated for both the shoring system and the wall formwork analyses:

a) **Shoring System**

1- A comparison of the present method with others shows that the construction load distribution predicted by the present method is within ranges of -7% to +22% for slab loads and +7% to +30% for shore loads.

2- The simplified method is adequate for predicting the location of the absolute maximum slab and shore loads. However, it generally overestimates slightly the slab load ratios.

3- The differences in the results between the present method and simplified method are mostly attributed to the fundamental assumptions in the simplified method. These assumptions include infinite axial stiffness of shores and reshores and neglecting the effects of the change in construction load during construction cycles, and the effects of creep and shrinkage on concrete structure.

4- The change in construction load during construction cycles attributed to the effects of time-dependent effects caused by creep and shrinkage of concrete during construction cycles has little effect on the construction load distribution.

5- The influence of slab stiffness acquired from different concrete strengths has insignificant effect on construction load distribution.

6- A comparison between the analyses using wooden and steel shores and reshores shows that the influence of axial shore stiffness has little effect on construction load distribution. This is due to a small difference in axial stiffness between two examined cases where the axial stiffness of steel shore is about 3.47 times of that of wooden shore.

7- The boundary conditions of slab have, in general, little effect on the construction load distribution. However, the boundary conditions affect the maximum shore load more than the slab load.

8- The use of three levels of shores procedure leads to increase the absolute maximum slab and shore loads by 10% and 40% respectively with respect to the procedure of two levels of shores and one level of reshores.

9- The use of slab moment for determining the slab loads gives higher estimate for the absolute maximum slab load than that using shear force.

b) Wall Formwork

1- The present method is a computer-aided analysis, while Rodin's, ACI, CIRIA and Gardner's methods are empirical equations derived from experimental data.

2- The pressure envelope obtained by the present method is similar in shape to the typical pressure envelope.

3- A comparison between the field measurements and the present method shows that the assumptions involved in the present method are valid and the analysis can be applied within a reasonable level of accuracy.

4- The ACI, Rodin's and Gardener's recommendations are inadequate for predicting the maximum lateral pressures on the examined cases, tall and thick concrete walls.

5- The CIRIA recommendations give reasonable and conservative estimates for the maximum lateral pressures.

6- The parameters involved in the first process of the present method, the initial pressure envelope, have significant effect on the magnitude of the maximum lateral pressure. These parameters are the concrete placement rate, concrete density and load patterns.

7- The parameters involved in the second process of the present method, the restrain of lateral forces, have little effect on the magnitude of the maximum lateral pressure. However, they play a role in distributing the lateral pressures on wall forms. These parameters are concrete temperature, minimum dimension of form, form tie stiffness, form stiffness, neglecting the concrete part in the analysis, and the amount of reinforcement.

6.3 Recommendations for Future Work

Based on the results of the present study, the following recommendations in both the shoring system and wall formwork areas can be made:

a) **Shoring System**

- Additional field data, especially continuous shore and reshore measurements during construction cycles, are needed to develop a good data base for a reliable assessment of load fluctuations on shoring systems, to establish construction load factors and to verify the accuracy of the existing load analysis procedures.
- The model for shoring system can be extended to 3-D model.
- The effect of non-linearity produced by cracking of concrete should be investigated. CPF can be used for this purpose.

b) **Wall Formwork**

- Additional field studies are needed to develop a good database for a reliable assessment of load fluctuations on vertical formwork, to develop the formulation of concrete lateral pressure criteria in design and to verify the accuracy of using the existing procedures for determining concrete lateral pressures.

- The present method neglects the effects of concrete properties on the first process, the initial pressure envelope, by assuming the concrete properties coefficient, C, to be equal one. Therefore, the effects of concrete temperature, type of cement and workability (water cement ratio, aggregate size, and cement content) could be included for better results.

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APPENDIX A

INPUT DATA FILE FOR SHORING SYSTEM

TITEL: TWO LEVELES OF WODDEN SHORES AND ONE LEVEL OF WODDEN RESHORES PROCEDURE (CONSTRUCTION EXAMPLE USED IN SECTION 5.2)

```
C**** SET 1 :CONTROL PARAMETERS.
IOUT = 1  IUNITS = 1 NPOIN = 16  NELEM = 21  NSD = 7  NCTYP = 13
NSTYP  = 1 MNSC = 3   MNCL = 1  MNPSL = 0  MNSL = 2 NLSTG = 13
IOWT = 0  IFRCT = 0  ITPND = 1  NONLIN = 0 IRELAX = 0 ITDATA = 0

C**** SET 2: MATERIAL PROPERTIES.
C**** FOR CONCRETE:
C**   I  ISTG  GAMA ALFAT  FCT(N/m2)
      1   1    0.0   0.0    2.5E6
      2   2    0.0   0.0    2.3E6
      3   2    0.0   0.0    2.5E6
      4   3    0.0   0.0    2.5E6
      5   4    0.0   0.0    2.3E6
      6   4    0.0   0.0    2.5E6
      7   5    0.0   0.0    2.5E6
      8   7    0.0   0.0    2.3E6
      9   7    0.0   0.0    2.5E6
     10   8    0.0   0.0    2.5E6
     11  11    0.0   0.0    2.3E6
     12  11    0.0   0.0    2.5E6
     13  12    0.0   0.0    2.5E6
         END-OF-GROUP
C**** FOR STEEL:
C**    I    ES(N/m2)   BETA1
       1    200.0E9     1.0
         END-OF-GROUP

C**** SET 3: COORDINATES OF NOODES.
C**   I       XI(I)(m)      YI(I)(m)
      1         0.0          14.0
      2         1.5          14.0
      3         3.0          14.0
      4         0.0          11.2
      5         1.5          11.2
      6         3.0          11.2
      7         0.0           8.4
      8         1.5           8.4
      9         3.0           8.4
     10         0.0           5.6
     11         1.5           5.6
```

```
     12          3.0            5.6
     13          0.0            2.8
     14          1.5            2.8
     15          3.0            2.8
     16          0.0            0.0
            END-OF-GROUP

C**** SET 5: TOPOLOGY OF MEMBERS AND SECTION DIMENSIONS.
C**** ELEMENT 1:
C**** CONTROL INFORMATION.
C**    IE  NOD(IE,1) NOD(IE,2) NSEC NCL NPSL NSL INTG ILSTG DO(m) SRM(m)
       1     1         4         3    1    0  . 2   1     1   0.250
C**    NC      DX
       2    0.5
C**    IWSC(IE,I), I=1,NSEC
       1   2   3
C**** CONCRETE DIMENSIONS INFORMATION.
C**    PART 1:  NC     ICTYP(1,1)     ICTYP(1,2)
                1        1               0
C**  NSTRT     NEND      BTL (m)     BBL (m)     BTR     BBR     BTC     BBC
                1         3            0.5        0.5
C**  NSTRT     NEND      DCTL (m)    DL (m)      DCTR     DR     DCTC    DC
                1         3            0.0        0.5
                END-OF-GROUP
C**** LAYOUT AND PROPERTIES OF NONPRESTRESSED STEEL LAYERS.
C** I INSTYP(I,1) INSTYP(I,2) NSTRT NEND  ASL(m2)    DSL(m) DSR DSC
    1      1            1         1    3   0.0018     0.05
    2      1            1         1    3   0.0018     0.45

C**** ELEMENT 2:
C**** CONTROL INFORMATION.
C** IE NOD(IE,1) NOD(IE,2) NSEC NCL NPSL NSL INTG ILSTG DO(m) SRM(m)
     2     2         5        3   1    0    0    1     2   0.025
C**    NC      DX
       2      0.5
C**    IWSC(IE,I), I=1,NSEC
       1   2   3
C**** CONCRETE DIMENSIONS INFORMATION.
C**    PART 1:  NC     ICTYP(1,1)     ICTYP(1,2)
                1         2              1
C**  NSTRT     NEND      DCTL (m)    DL (m)      DCTR     DR     DCTC     DC
                1         3          0.0         0.05
C**  N     AC(m2)      YC(m)       AIC(m4)
     1      0.005       0.025       1.0E-59   GEN 1,3
            END-OF-GROUP

C**** ELEMENT 3:
C**** CONTROL INFORMATION.
C** IE NOD(IE,1) NOD(IE,2) NSEC NCL NPSL NSL INTG ILSTG DO(m) SRM(m)
    3     3         6        3    1   0     0   1     2     0.0
C**    NC      DX
       2      0.5
C**    IWSC(IE,I), I=1,NSEC
       1   2   3
C**** CONCRETE DIMENSIONS INFORMATION.
C**    PART 1:  NC     ICTYP(1,1)     ICTYP(1,2)
                1         2              1
```

```
C**  NSTRT    NEND     DCTL (m)  DL (m)     DCTR     DR     DCTC     DC
      1        3       0.0       0.025
C**  N     AC(m2)     YC(m)      AIC(m4)
     1      0.0025     0.0125     1.0E-59    GEN 1,3
          END-OF-GROUP

C**** ELEMENT 4:
C**** CONTROL INFORMATION.
C**    IE  NOD(IE,1) NOD(IE,2)NSEC NCL NPSL NSL INTG ILSTG DO(m)SRM(m)
       4      4        5        3   1    0    2    1     2   0.09
C**    NC      DX
       2      0.5
C**    IWSC(IE,I), I=1,NSEC
       1   2   3
C**** CONCRETE DIMENSIONS INFORMATION.
C**    PART 1:  NC    ICTYP(1,1)    ICTYP(1,2)
                1         3             0
C** NSTRT    NEND     BTL (m)    BBL (m)     BTR     BBR     BTC     BBC
     1        3       0.9        0.9
C** NSTRT    NEND     DCTL (m)  DL (m)     DCTR     DR     DCTC     DC
     1        3       0.0       0.18
          END-OF-GROUP
C**** LAYOUT AND PROPERTIES OF NONPRESTRESSED STEEL LAYERS.
C**   I INSTYP(I,1)INSTYP(I,2) NSTRT NEND ASL(m2)DSL(m)DSR DSC
     1        1         1        1    3   0.0008  0.02
     2        1         1        1    3   0.0004  0.16

C**** ELEMENT 5:
C**** CONTROL INFORMATION.
C** IE NOD(IE,1) NOD(IE,2) NSEC NCL NPSL NSL INTG ILSTG DO(m) SRM(m)
     5   5          6         3   1    0   1    1    2   0.09
C**    NC      DX
       2      0.5
C**    IWSC(IE,I), I=1,NSEC
       1   2   3
C**** CONCRETE DIMENSIONS INFORMATION.
C**    PART 1:  NC    ICTYP(1,1)    ICTYP(1,2)
                1         3             0
C**  NSTRT    NEND     BTL (m)    BBL (m)     BTR     BBR     BTC     BBC
      1        3       0.9         0.9
C**  NSTRT    NEND     DCTL (m)  DL (m)     DCTR     DR     DCTC     DC
      1        3       0.0        0.18
          END-OF-GROUP
C**** LAYOUT AND PROPERTIES OF
C**  I  INSTYP(I,1) INSTYP(I,2) NSTRT NEND ASL(m2) DSL(m) DSR DSC
     1      1           1         1    3 0.0008   0.16

C**** ELEMENT 6:
C**** CONTROL INFORMATION.
C**    IE  NOD(IE,1) NOD(IE,2) NSEC NCL NPSL NSL INTG ILSTG DO(m) SRM(m)
       6    4          7         3    1    0   2    1    3  0.250
C**    NC      DX
       2     0.5
C**    IWSC(IE,I), I=1,NSEC
       1   2   3
C**** CONCRETE DIMENSIONS INFORMATION.
C**    PART 1:  NC    ICTYP(1,1)    ICTYP(1,2)
```

```
                  1            4                0
C**   NSTRT      NEND         BTL  (m)     BBL  (m)      BTR       BBR      BTC       BBC
       1          3           0.5          0.5
C**   NSTRT      NEND         DCTL  (m)    DL  (m)       DCTR      DR       DCTC      DC
       1          3           0.0          0.5
                  END-OF-GROUP
C**** LAYOUT AND PROPERTIES OF NONPRESTRESSED STEEL LAYERS.
C** I INSTYP(I,1) INSTYP(I,2) NSTRT NEND  ASL(m2)      DSL(m) DSR DSC
    1        1           1        1    3   0.0018       0.05
    2        1           1        1    3   0.0018       0.45

C**** ELEMENT 7:
C**** CONTROL INFORMATION.
C** IE NOD(IE,1) NOD(IE,2) NSEC NCL NPSL NSL INTG ILSTG DO(m) SRM(m)
    7    5          8        3    1    0    0    1     4    0.025
C**     NC        DX
        2        0.5
C**     IWSC(IE,I),  I=1,NSEC
        1   2   3
C**** CONCRETE DIMENSIONS INFORMATION.
C**     PART 1:  NC       ICTYP(1,1)      ICTYP(1,2)
                 1            5               1
C**   NSTRT      NEND         DCTL  (m)   DL  (m)       DCTR      DR       DCTC      DC
        1         3             0.0         0.05
C**   N      AC(m2)       YC(m)        AIC(m4)
      1      0.005        0.025        1.0E-59     GEN 1,3
             END-OF-GROUP

C**** ELEMENT 8:
C**** CONTROL INFORMATION.
C** IE NOD(IE,1) NOD(IE,2) NSEC NCL NPSL NSL INTG ILSTG DO(m) SRM(m)
    8    6          9        3    1    0    0    1     4    0.0
C**     NC        DX
        2        0.5
C**     IWSC(IE,I),  I=1,NSEC
        1   2   3
C**** CONCRETE DIMENSIONS INFORMATION.
C**     PART 1:  NC       ICTYP(1,1)      ICTYP(1,2)
                 1            5               1
C**   NSTRT      NEND         DCTL  (m)   DL  (m)       DCTR      DR       DCTC      DC
        1         3           0.0         0.025
C**   N      AC(m2)       YC(m)        AIC(m4)
      1      0.0025       0.0125       1.0E-59     GEN 1,3
      END-OF-GROUP
C**** ELEMENT 9:
C**** CONTROL INFORMATION.
C**     IE  NOD(IE,1) NOD(IE,2)NSEC NCL NPSL NSL  INTG ILSTG DO(m)SRM(m)
        9       7          8       3    1    0    2    1     4    0.09
C**     NC        DX
        2        0.5
C**     IWSC(IE,I),  I=1,NSEC
        1   2   3
C**** CONCRETE DIMENSIONS INFORMATION.
C**     PART 1:  NC       ICTYP(1,1)      ICTYP(1,2)
                 1            6               0
C** NSTRT      NEND         BTL  (m)     BBL  (m)      BTR       BBR      BTC       BBC
      1         3           0.9          0.9
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C** NSTRT     NEND      DCTL (m)   DL (m)      DCTR     DR     DCTC     DC
     1         3        0.0         0.18
          END-OF-GROUP
C**** LAYOUT AND PROPERTIES OF NONPRESTRESSED STEEL LAYERS.
C**   I INSTYP(I,1)INSTYP(I,2) NSTRT NEND ASL(m2)DSL(m)DSR DSC

      1      1              1            1    3    0.0008      0.02
      2      1              1            1    3    0.0004       0.16

C**** ELEMENT 10:
C**** CONTROL INFORMATION.
C** IE NOD(IE,1) NOD(IE,2) NSEC NCL NPSL NSL INTG ILSTG DO(m) SRM(m)
    10      8          9         3   1   0     1     1     4    0.09
C**    NC      DX
       2       0.5
C**    IWSC(IE,I), I=1,NSEC
       1   2   3
C**** CONCRETE DIMENSIONS INFORMATION.
C**    PART 1:  NC     ICTYP(1,1)     ICTYP(1,2)
                1          6              0
C**   NSTRT     NEND      BTL (m)     BBL (m)     BTR      BBR     BTC      BBC
       1         3        0.9         0.9
C**   NSTRT     NEND      DCTL (m)   DL (m)      DCTR     DR     DCTC     DC
       1         3        0.0        0.18
                END-OF-GROUP
C**** LAYOUT AND PROPERTIES OF NONPRESTRESSED STEEL LAYERS.
C**   I  INSTYP(I,1) INSTYP(I,2) NSTRT NEND ASL(m2) DSL(m) DSR DSC
     1      1             1        1     3   0.0008   0.16

C**** ELEMENT 11:
C**** CONTROL INFORMATION.
C**    IE  NOD(IE,1) NOD(IE,2) NSEC NCL NPSL NSL INTG ILSTG DO(m) SRM(m)
       11       7         10        3    1    0   2     1     5  0.250
C**    NC       DX
       2     0.5
C**    IWSC(IE,I), I=1,NSEC
       1   2   3
C**** CONCRETE DIMENSIONS INFORMATION.
C**    PART 1:  NC     ICTYP(1,1)     ICTYP(1,2)
                1          7              0
C**   NSTRT     NEND      BTL (m)     BBL (m)     BTR      BBR     BTC      BBC
       1         3        0.5          0.5
C**   NSTRT     NEND      DCTL (m)   DL (m)      DCTR     DR     DCTC     DC
       1         3        0.0          0.5
                END-OF-GROUP
C**** LAYOUT AND PROPERTIES OF NONPRESTRESSED STEEL LAYERS.
C** I INSTYP(I,1) INSTYP(I,2) NSTRT NEND  ASL(m2)    DSL(m) DSR DSC
    1        1             1      1     3    0.0018      0.05
    2        1             1      1     3    0.0018      0.45

C**** ELEMENT 12:
C**** CONTROL INFORMATION.
C** IE NOD(IE,1) NOD(IE,2) NSEC NCL NPSL NSL INTG ILSTG DO(m) SRM(m)
    12     8           11         3   1    0     0    1     7    0.025
C**    NC      DX
       2       0.5
C**    IWSC(IE,I), I=1,NSEC
```

```
      1   2    3
C**** CONCRETE DIMENSIONS INFORMATION.
C**    PART 1:  NC     ICTYP(1,1)     ICTYP(1,2)
                1          8              1
C**   NSTRT     NEND      DCTL (m)   DL (m)      DCTR     DR     DCTC     DC
        1         3        0.0        0.05
C**   N     AC(m2)      YC(m)        AIC(m4)
      1     0.005       0.025      1.0E-59   GEN 1,3
            END-OF-GROUP

C**** ELEMENT 13:
C**** CONTROL INFORMATION.
C** IE NOD(IE,1) NOD(IE,2) NSEC NCL NPSL NSL INTG ILSTG DO(m) SRM(m)
    13    9          12        3    1    0    0    1     7     0.0
C**    NC       DX
       2        0.5
C**    IWSC(IE,I), I=1,NSEC
       1   2   3
C**** CONCRETE DIMENSIONS INFORMATION.
C**    PART 1:  NC     ICTYP(1,1)     ICTYP(1,2)
                1          8              1
C**   NSTRT     NEND      DCTL (m)   DL (m)      DCTR     DR     DCTC     DC
        1         3        0.0        0.025
C**   N     AC(m2)      YC(m)        AIC(m4)
      1     0.0025     0.0125      1.0E-59   GEN 1,3
            END-OF-GROUP

C**** ELEMENT 14:
C**** CONTROL INFORMATION.
C**    IE   NOD(IE,1) NOD(IE,2)NSEC NCL NPSL NSL INTG ILSTG DO(m)SRM(m)
      14      10          11       3    1    0    2    1     7   0.09
C**    NC       DX
       2        0.5
C**    IWSC(IE,I), I=1,NSEC
       1   2   3
C**** CONCRETE DIMENSIONS INFORMATION.
C**    PART 1:  NC     ICTYP(1,1)     ICTYP(1,2)
                1          9              0
C** NSTRT     NEND      BTL (m)     BBL (m)     BTR      BBR      BTC     BBC
      1         3        0.9         0.9
C** NSTRT     NEND      DCTL (m)   DL (m)      DCTR     DR     DCTC     DC
      1         3        0.0       0.18
            END-OF-GROUP
C**** LAYOUT AND PROPERTIES OF NONPRESTRESSED STEEL LAYERS.
C**   I INSTYP(I,1)INSTYP(I,2) NSTRT NEND ASL(m2)DSL(m)DSR DSC
      1       1           1         1     3   0.0008   0.02
      2       1           1         1     3   0.0004   0.16

C**** ELEMENT 15:
C**** CONTROL INFORMATION.
C** IE NOD(IE,1) NOD(IE,2) NSEC NCL NPSL NSL INTG ILSTG DO(m) SRM(m)
    15    11         12        3    1    0    1    1     7    0.09
C**    NC       DX
       2        0.5
C**    IWSC(IE,I), I=1,NSEC
       1   2   3
C**** CONCRETE DIMENSIONS INFORMATION.
```

```
C**    PART 1:  NC     ICTYP(1,1)     ICTYP(1,2)
                1        9             0
C**   NSTRT     NEND      BTL (m)     BBL (m)     BTR     BBR     BTC     BBC
       1         3        0.9          0.9
C**   NSTRT     NEND      DCTL (m)   DL (m)      DCTR     DR     DCTC     DC
       1         3        0.0          0.18
                END-OF-GROUP
C**** LAYOUT AND PROPERTIES OF
C**   I  INSTYP(I,1) INSTYP(I,2) NSTRT NEND ASL(m2) DSL(m) DSR DSC
      1       1            1        1    3    0.0008   0.16

C**** ELEMENT 16:
C**** CONTROL INFORMATION.
C**    IE  NOD(IE,1) NOD(IE,2) NSEC NCL NPSL NSL INTG ILSTG DO(m) SRM(m)
       16     10          13       3   1    0    2    1     8  0.250
C**    NC      DX
       2    0.5
C**    IWSC(IE,I), I=1,NSEC
       1   2   3
C**** CONCRETE DIMENSIONS INFORMATION.
C**    PART 1:  NC     ICTYP(1,1)     ICTYP(1,2)
                1        10             0
C**   NSTRT     NEND      BTL (m)     BBL (m)     BTR     BBR     BTC     BBC
        1        3        0.5          0.5
C**   NSTRT     NEND      DCTL (m)   DL (m)      DCTR     DR     DCTC     DC
        1        3        0.0          0.5
                END-OF-GROUP
C**** LAYOUT AND PROPERTIES OF NONPRESTRESSED STEEL LAYERS.
C** I INSTYP(I,1) INSTYP(I,2) NSTRT NEND  ASL(m2)    DSL(m) DSR DSC
    1       1           1        1    3    0.0018     0.05
    2       1           1        1    3    0.0018     0.45

C**** ELEMENT 17:
C**** CONTROL INFORMATION.
C** IE NOD(IE,1) NOD(IE,2) NSEC NCL NPSL NSL INTG ILSTG DO(m) SRM(m)
    17     11        14        3   1    0    0    1    11    0.025
C**    NC      DX
       2       0.5
C**    IWSC(IE,I), I=1,NSEC
     1   2   3
C**** CONCRETE DIMENSIONS INFORMATION.
C**    PART 1:  NC     ICTYP(1,1)     ICTYP(1,2)
                1        11              1
C**   NSTRT     NEND      DCTL (m)   DL (m)      DCTR     DR     DCTC     DC
       1         3        0.0      0.05
C**   N     AC(m2)      YC(m)        AIC(m4)
      1     0.005      0.025       1.0E-59    GEN 1,3
      END-OF-GROUP

C**** ELEMENT 18:
C**** CONTROL INFORMATION.
C** IE NOD(IE,1) NOD(IE,2) NSEC NCL NPSL NSL INTG ILSTG DO(m) SRM(m)
    18     12         15       3   1    0    0    1    11   0.0
C**    NC      DX
       2       0.5
C**    IWSC(IE,I), I=1,NSEC
       1   2   3
```

```
C**** CONCRETE DIMENSIONS INFORMATION.
C**    PART 1:  NC     ICTYP(1,1)     ICTYP(1,2)
                1          11              1
C**  NSTRT     NEND      DCTL (m)  DL (m)      DCTR     DR     DCTC     DC
       1        3         0.0       0.025
C**  N     AC(m2)       YC(m)        AIC(m4)
     1      0.0025       0.0125       1.0E-59   GEN 1,3
            END-OF-GROUP

C**** ELEMENT 19:
C**** CONTROL INFORMATION.
C**    IE   NOD(IE,1) NOD(IE,2)NSEC NCL NPSL NSL INTG ILSTG DO(m)SRM(m)
       19      13        14      3    1    0    2    1    11    0.09
C**    NC       DX
       2        0.5
C**    IWSC(IE,I), I=1,NSEC
       1   2   3
C**** CONCRETE DIMENSIONS INFORMATION.
C**    PART 1:  NC     ICTYP(1,1)     ICTYP(1,2)
                1          12              0
C** NSTRT     NEND      BTL (m)     BBL (m)      BTR     BBR     BTC     BBC
       1        3         0.9         0.9
C** NSTRT     NEND      DCTL (m)  DL (m)      DCTR     DR     DCTC     DC
       1        3         0.0       0.18
            END-OF-GROUP
C**** LAYOUT AND PROPERTIES OF NONPRESTRESSED STEEL LAYERS.
C**  I INSTYP(I,1)INSTYP(I,2) NSTRT NEND ASL(m2)DSL(m)DSR DSC
     1      1            1        1    3   0.0008  0.02
     2      1            1        1    3   0.0004  0.16

C**** ELEMENT 20:
C**** CONTROL INFORMATION.
C** IE NOD(IE,1) NOD(IE,2) NSEC NCL NPSL NSL INTG ILSTG DO(m) SRM(m)
    20    14          15       3    1    0    1    1    11    0.09
C**    NC       DX
       2        0.5
C**    IWSC(IE,I), I=1,NSEC
       1   2   3
C**** CONCRETE DIMENSIONS INFORMATION.
C**    PART 1:  NC     ICTYP(1,1)     ICTYP(1,2)
                1          12              0
C**  NSTRT     NEND      BTL (m)     BBL (m)      BTR     BBR     BTC     BBC
       1        3         0.9         0.9
C**  NSTRT     NEND      DCTL (m)  DL (m)      DCTR     DR     DCTC     DC
       1        3         0.0       0.18
            END-OF-GROUP
C**** LAYOUT AND PROPERTIES OF
C**  I  INSTYP(I,1) INSTYP(I,2) NSTRT NEND ASL(m2) DSL(m) DSR DSC
     1      1             1         1    3  0.0008    0.16

C**** ELEMENT 21:
C**** CONTROL INFORMATION.
C**    IE  NOD(IE,1) NOD(IE,2) NSEC NCL NPSL NSL INTG ILSTG DO(m) SRM(m)
       21     13        16        3    1    0    2    1    12   0.250
C**    NC       DX
       2     0.5
C**    IWSC(IE,I), I=1,NSEC
```

```
      1   2   3
C**** CONCRETE DIMENSIONS INFORMATION.
C**    PART 1:  NC      ICTYP(1,1)     ICTYP(1,2)
               1         13             0
C**   NSTRT     NEND      BTL (m)      BBL (m)      BTR      BBR      BTC      BBC
      1         3         0.5          0.5
C**   NSTRT     NEND      DCTL (m)   DL (m)       DCTR      DR      DCTC      DC
      1         3         0.0          0.5
          END-OF-GROUP
C**** LAYOUT AND PROPERTIES OF NONPRESTRESSED STEEL LAYERS.
C** I INSTYP(I,1) INSTYP(I,2) NSTRT NEND  ASL(m2)     DSL(m) DSR DSC
    1       1            1        1     3   0.0018      0.05
    2       1            1        1     3   0.0018      0.45
          END-OF-GROUP

C**** SET 6: TIME-DEPENDENT PARAMETERS.
C**** FOR TIME INTERVAL NO. 1:
C**** CONTROL PARAMETERS.
  IBC = 1 ILOAD = 1 ITPND = 1 ITMP = 0 IFORM = 0 JOUT = 0
C**** BOUNDARY CONDITIONS.
C**     NBC     NBD(1)     NBD(2)     NBD(3)      BD(1)      BD(2)      BD(3)
        1        0          0          0
              END-OF-GROUP
C**** TIME-DEPENDENT MATERIAL PROPERTIES.
C**    FOR CONCRETE:
C**    I        EC(T0)  (N/m2)  ! FOR COLUMN
       1         1.0E5
C**    PHI(Ti, Tj)
        0.0
C**     CHI(Ti, Tj)      S(Ti, Tj)
        0.0               0.0
C**** APPLIED LOADS:
DISTRIBUTED LOADS:
C** IELEM   IDIR  DIST1    DIST2(m)     QL(N/m)  QR      QC
     1       1      0.0       2.8        6.0E3
          END-OF-GROUP

C**** FOR TIME INTERVAL NO. 2:
C**** CONTROL PARAMETERS.
  IBC = 1 ILOAD = 1 ITPND = 0 ITMP = 0 IFORM = 0 JOUT = 0
C**** BOUNDARY CONDITIONS.
C**     NBC     NBD(1)     NBD(2)     NBD(3)      BD(1)      BD(2)
BD(3)
        2        0          0          1
        3        0          0          0
        6        0          1          0
          END-OF-GROUP
C**** TIME-DEPENDENT MATERIAL PROPERTIES.
C**    FOR CONCRETE:
C**   I       EC(T0)  (N/m2)! FOR COLUMN
      1       2.76E10
C**   I       EC(T0)  (N/m2)! FOR SHORE
      2       7.75E9
C**   I       EC(T0)  (N/m2)!FOR SLAB
      3       1.0E5
C**** APPLIED LOADS:
DISTRIBUTED LOADS:
```

```
C** IELEM  IDIR  DIST1    DIST2(m)    QL(N/m)   QR     QC
     4       0      0.0      1.5        3.89E3
     5       0      0.0      1.5        3.89E3
         END-OF-GROUP

C**** FOR TIME INTERVAL NO. 3:
C**** CONTROL PARAMETERS.
  IBC = 0 ILOAD = 1 ITPND = 1 ITMP = 0 IFORM = 0 JOUT = 0
C**** TIME-DEPENDENT MATERIAL PROPERTIES.
C**    FOR CONCRETE:
C**    I       EC(T0) (N/m2) ! FOR COLUMN
       1       2.76E10
C**    PHI(Ti, Tj)
        0.0 0.595 0.595
C**     CHI(Ti, Tj)     S(Ti, Tj)
        0.730        -0.87121E-04
C**    I       EC(T0) (N/m2)!FOR SHORE
       2       7.75E9
C**    PHI(Ti, Tj)
        0.0 0.0
C**     CHI(Ti, Tj)     S(Ti, Tj)
        0.0             0.0
C**    I       EC(T0) (N/m2)!FOR SLAB
       3       1.0E5
C**    PHI(Ti, Tj)
        0.0 0.0
C**     CHI(Ti, Tj)     S(Ti, Tj)
        0.0                0.0
C**    I       EC(T0) (N/m2) ! FOR COLUMN
       4       1.0E5
C**    PHI(Ti, Tj)
        0.0
C**     CHI(Ti, Tj)     S(Ti, Tj)
        0.0               0.0
C**** APPLIED LOADS:
DISTRIBUTED LOADS:
C** IELEM  IDIR  DIST1    DIST2(m)    QL(N/m)   QR     QC
       6      1     0.0      2.8         6.0E3
           END-OF-GROUP

C**** FOR TIME INTERVAL NO. 4:
C**** CONTROL PARAMETERS.
  IBC = 1 ILOAD = 1 ITPND = 0 ITMP = 0 IFORM = 0 JOUT = 0
C**** BOUNDARY CONDITIONS.
C**    NBC     NBD(1)     NBD(2)     NBD(3)      BD(1)      BD(2)       BD(3)
        9       0          1          0
          END-OF-GROUP
C**** TIME-DEPENDENT MATERIAL PROPERTIES.
C**    FOR CONCRETE:
C**    I       EC(T0) (N/m2)! FOR COLUMN
       1        3.02E10
C**    I       EC(T0) (N/m2)! FOR SHORE
       2       7.75E9
C**    I       EC(T0) (N/m2)!FOR SLAB
       3       2.76E10
C**    I       EC(T0) (N/m2)! FOR COLUMN
       4       2.76E10
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C**    I       EC(T0)  (N/m2)! FOR SHORE
       5       7.75E9
C**    I       EC(T0)  (N/m2)!FOR SLAB
       6       1.0E5
C**** APPLIED LOADS:
DISTRIBUTED LOADS:
C** IELEM  IDIR  DIST1   DIST2(m)   QL(N/m)  QR    QC
     9     0     0.0      1.5       3.89E3
    10     0     0.0      1.5       3.89E3
    END-OF-GROUP

C**** FOR TIME INTERVAL NO. 5:
C**** CONTROL PARAMETERS.
 IBC = 0 ILOAD = 1 ITPND = 1 ITMP = 0 IFORM = 0 JOUT = 0
C**** TIME-DEPENDENT MATERIAL PROPERTIES.
C**    FOR CONCRETE:
C**    I       EC(T0)  (N/m2)   ! FOR COLUMN
       1        3.02E10
C**    PHI(Ti, Tj)
        0.0 0.752 0.752 0.469 0.469
C**     CHI(Ti, Tj)     S(Ti, Tj)
        0.785     -0.46341E-04
C**    I       EC(T0)  (N/m2)! FOR SHORE
       2       7.75E9
C**    PHI(Ti, Tj)
        0.0 0.0 0.0 0.0
C**     CHI(Ti, Tj)     S(Ti, Tj)
        0.0            0.0
C**    I       EC(T0)  (N/m2)!FOR SLAB
       3       2.76E10
C**    PHI(Ti, Tj)
        0.0 0.0  0.509 0.509
C**     CHI(Ti, Tj)     S(Ti, Tj)
        0.728   -0.65341E-04
C**    I       EC(T0)  (N/m2)
       4       2.76E10  (N/m2)! FOR COLUMN
C**    PHI(Ti, Tj)
         0.0  0.509 0.509
C**     CHI(Ti, Tj)     S(Ti, Tj)
        0.728   -0.65341E-04
C**    I       EC(T0)  (N/m2)! FOR SHORE
       5       7.75E9.
C**    PHI(Ti, Tj)
        0.0 0.0
C**     CHI(Ti, Tj)     S(Ti, Tj)
        0.0     0.0
C**    I       EC(T0)  (N/m2)!FOR SLAB
       6       1.0E5
C**    PHI(Ti, Tj)
        0.0 0.0
C**     CHI(Ti, Tj)     S(Ti, Tj)
        0.0     0.0
C**    I       EC(T0)  (N/m2)  ! FOR COLUMN
       7       1.0E5
C**    PHI(Ti, Tj)
        0.0
C**     CHI(Ti, Tj)     S(Ti, Tj)
```

```
        0.0      0.0
C** APPLIED LOADS
DISTRIBUTED LOADS:
C** IELEM  IDIR  DIST1   DIST2(m)   QL(N/m)  QR    QC
     11     1     0.0     2.8        6.0E3
         END-OF-GROUP

C**** FOR TIME INTERVAL NO. 6:
C**** CONTROL PARAMETERS.
IBC = 0 ILOAD = 0 ITPND = 1 ITMP = 0 IFORM = 0 JOUT = 0
C**** TIME-DEPENDENT MATERIAL PROPERTIES.
C**    FOR CONCRETE:
C**    I       EC(T0) (N/m2) ! FOR COLUMN
       1       3.11E10
C**    PHI(Ti, Tj)
         0.0 0.801 0.801 0.548 0.548 0.286
C**     CHI(Ti, Tj)     S(Ti, Tj)
        0.773    -0.15888E-04
C**    I       EC(T0) (N/m2)! FOR SHORE
       2       7.75E9
C**    PHI(Ti, Tj)
        0.0 0.0 0.0 0.0 0.0
C**     CHI(Ti, Tj)     S(Ti, Tj)
        0.0      0.0
C**    I       EC(T0) (N/m2)  !FOR SLAB
       3       2.97E10
C**    PHI(Ti, Tj)
        0.0 0.0 0.595 0.595 0.302
C**     CHI(Ti, Tj)     S(Ti, Tj)
        0.758     -0.21780E-04
C**    I       EC(T0) (N/m2) ! FOR COLUMN
       4       2.97E10
C**    PHI(Ti, Tj)
         0.0 0.595 0.595 0.302
C**     CHI(Ti, Tj)     S(Ti, Tj)
        0.758     -0.21780E-04
C**    I       EC(T0) (N/m2)! FOR SHORE
       5       7.75E9
C**    PHI(Ti, Tj)
        0.0 0.0 0.0
C**     CHI(Ti, Tj)     S(Ti, Tj)
        0.0    0.0
C**    I       EC(T0) (N/m2)!FOR SLAB
       6       2.59E10
C**    PHI(Ti, Tj)
        0.0 0.0 0.335
C**     CHI(Ti, Tj)     S(Ti, Tj)
        0.687  -0.31680E-04
C**    I       EC(T0) (N/m2) ! FOR COLUMN
       7       2.59E10
C**    PHI(Ti, Tj)
         0.0 0.335
C**     CHI(Ti, Tj)     S(Ti, Tj)
        0.687  -0.31680E-04

C**** FOR TIME INTERVAL NO. 7:
C**** CONTROL PARAMETERS.
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    IBC = 1 ILOAD = 1 ITPND = 0 ITMP = 0 IFORM = 0 JOUT = 0
C**** BOUNDARY CONDITIONS.
C**   NBC    NBD(1)    NBD(2)    NBD(3)    BD(1)    BD(2)    BD(3)
       12      0         1         0
          END-OF-GROUP
C**** TIME-DEPENDENT MATERIAL PROPERTIES.
C**   FOR CONCRETE:
C**   I      EC(T0) (N/m2) ! FOR COLUMN
      1      3.13E10
C**   I      EC(T0) (N/m2)! FOR SHORE
      2      7.75E9
C**   I      EC(T0) (N/m2)!FOR SLAB
      3      3.02E10
C**   I      EC(T0) (N/m2)! FOR COLUMN
      4      3.02E10
C**   I      EC(T0) (N/m2)! FOR SHORE
      5      7.75E9
C**   I      EC(T0) (N/m2)!FOR SLAB
      6      2.76E10
C**   I      EC(T0) (N/m2) ! FOR COLUMN
      7      2.76E10
C**   I      EC(T0) (N/m2)! FOR SHORE
      8      7.75E9
C**   I      EC(T0) (N/m2) ! FOR SLAB
      9      1.0E5
C**** APPLIED LOADS:
DISTRIBUTED LOADS:
C** IELEM  IDIR  DIST1   DIST2(m)   QL(N/m)  QR    QC
     14     0     0.0     1.5       3.89E3
     15     0     0.0     1.5       3.89E3
       END-OF-GROUP

C**** FOR TIME INTERVAL NO. 8:
C**** CONTROL PARAMETERS.
IBC = 0 ILOAD = 1  ITPND = 1 ITMP = 0 IFORM = 0 JOUT = 0
C**** TIME-DEPENDENT MATERIAL PROPERTIES.
C**   FOR CONCRETE:
C**   I      EC(T0) (N/m2) ! FOR COLUMN
      1      3.13E10
C**   PHI(Ti, Tj)
        0.0 0.902 0.902 0.693 0.693 0.529 0.447 0.447
C**    CHI(Ti, Tj)    S(Ti, Tj)
       0.801  -0.34572E-04
C**   I      EC(T0) (N/m2)! FOR SHORE
      2      7.75E9
C**   PHI(Ti, Tj)
       0.0 0.0 0.0 0.0 0.0 0.0 0.0
C**    CHI(Ti, Tj)    S(Ti, Tj)
       0.0    0.0
C**   I      EC(T0) (N/m2)!FOR SLAB
      3      3.02E10
C**   PHI(Ti, Tj)
       0.0 0.0 0.752 0.752 0.558 0.469 0.469
C**    CHI(Ti, Tj)    S(Ti, Tj)
       0.785  -0.463141E-04
C**   I      EC(T0) (N/m2) ! FOR COLUMN
      4      3.02E10
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C**   PHI(Ti, Tj)
       0.0 0.752 0.752 0.558 0.469 0.469
C**    CHI(Ti, Tj)     S(Ti, Tj)
       0.785  -0.463141E-04
C**   I       EC(T0)  (N/m2)! FOR SHORE
      5       7.75E9
C**   PHI(Ti, Tj)
       0.0 0.0 0.0 0.0 0.0
C**    CHI(Ti, Tj)     S(Ti, Tj)
       0.0    0.0
C**   I       EC(T0)  (N/m2)!FOR SLAB
      6       2.76E10
C**   PHI(Ti, Tj)
       0.0 0.0 0.619 0.509 0.509
C**    CHI(Ti, Tj)     S(Ti, Tj)
       0.728    -0.65341E-04
C**   I       EC(T0)  (N/m2)  ! FOR COLUMN
      7       2.7610
C**   PHI(Ti, Tj)
        0.0 0.619 0.509 0.509
C**    CHI(Ti, Tj)     S(Ti, Tj)
       0.728    -0.65341E-04
C**   I       EC(T0)  (N/m2)! FOR SHORE
      8       7.75E9
C**   PHI(Ti, Tj)
       0.0 0.0
C**    CHI(Ti, Tj)     S(Ti, Tj)
       0.0    0.0
C**   I       EC(T0)  (N/m2)  ! FOR SLAB
      9       1.0E5
C**   PHI(Ti, Tj)
       0.0 0.0
C**    CHI(Ti, Tj)     S(Ti, Tj)
       0.0    0.0
C**   I       EC(T0)  (N/m2)  ! FOR COLUMN
      10      1.0E5
C**   PHI(Ti, Tj)
       0.0
C**    CHI(Ti, Tj)     S(Ti, Tj)
       0.0    0.0
C**** APPLIED LOADS:
DISTRIBUTED LOADS:
C** IELEM  IDIR  DIST1   DIST2(m)   QL(N/m)  QR    QC
     16     1     0.0     2.8        6.0E3
       END-OF-GROUP

C**** FOR TIME INTERVAL NO. 9:
C**** CONTROL PARAMETERS.
IBC = 0  ILOAD = 0  ITPND = 0 ITMP = 0 IFORM = 0 JOUT = 0
C**** TIME-DEPENDENT MATERIAL PROPERTIES.
C**   FOR CONCRETE:
C**   I       EC(T0)  (N/m2)  ! FOR COLUMN
      1       3.18E10
C**   I       EC(T0)  (N/m2)! FOR SHORE
      2       7.75E9
C**   I       EC(T0)  (N/m2)!FOR SLAB
      3       3.11E10
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C**    I       EC(T0)  (N/m2) ! FOR COLUMN
       4       3.11E10
C**    I       EC(T0)  (N/m2)! FOR SHORE
       5       7.75E9
C**    I       EC(T0)  (N/m2)!FOR SLAB
       6       2.97E10
C**    I       EC(T0)  (N/m2) ! FOR COLUMN
       7       2.97E10
C**    I       EC(T0)  (N/m2)! FOR SHORE
       8       7.75E9
C**    I       EC(T0)  (N/m2) ! FOR SLAB
       9      2.59E10
C**    I       EC(T0)  (N/m2) ! FOR COLUMN
       10      2.59E10

C**** FOR TIME INTERVAL NO. 10:
C**** CONTROL PARAMETERS.
IBC = 0  ILOAD = 0  ITPND = 1 ITMP = 0 IFORM = 0 JOUT = 0
C**** TIME-DEPENDENT MATERIAL PROPERTIES.
C**    FOR CONCRETE:
C**    I       EC(T0)  (N/m2) ! FOR COLUMN
       1       3.18E10
C**    PHI(Ti, Tj)
        0.0 0.937 0.937 0.738 0.738 0.591 0.522 0.522 0.276 0.276
C**     CHI(Ti, Tj)     S(Ti, Tj)
        0.778  -0.12100E-04
C**    I       EC(T0)  (N/m2)! FOR SHORE
       2       7.75E9
C**    PHI(Ti, Tj)
        0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0
C**     CHI(Ti, Tj)     S(Ti, Tj)
        0.0    0.0
C**    I       EC(T0)  (N/m2)!FOR SLAB
       3       3.11E10
C**    PHI(Ti, Tj)
        0.0 0.0 0.801 0.801 0.624 0.548 0.548 0.286 0.286
C**     CHI(Ti, Tj)     S(Ti, Tj)
        0.773   -0.15888E-04
C**    I       EC(T0)  (N/m2) ! FOR COLUMN
       4       3.11E10
C**    PHI(Ti, Tj)
        0.0 0.801 0.801 0.624 0.548 0.548 0.286 0.286
C**     CHI(Ti, Tj)     S(Ti, Tj)
        0.773   -0.15888E-04
C**    I       EC(T0)  (N/m2)! FOR SHORE
       5       7.75E9
C**    PHI(Ti, Tj)
        0.0 0.0 0.0 0.0 0.0 0.0 0.0
C**     CHI(Ti, Tj)     S(Ti, Tj)
        0.0    0.0
C**    I       EC(T0)  (N/m2)!FOR SLAB
       6       2.97E10
C**    PHI(Ti, Tj)
        0.0 0.0 0.692 0.595 0.595 0.302 0.302
C**     CHI(Ti, Tj)     S(Ti, Tj)
        0.758  -0.21780E-04
C**    I       EC(T0)  (N/m2) ! FOR COLUMN
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    7       2.97E10
C**   PHI(Ti, Tj)
       0.0 0.692 0.595 0.595 0.302 0.302
C**    CHI(Ti, Tj)     S(Ti, Tj)
       0.758  -0.21780E-04
C**   I         EC(T0) (N/m2)! FOR SHORE
    8       7.75E9
C**   PHI(Ti, Tj)
       0.0 0.0 0.0 0.0
C**    CHI(Ti, Tj)     S(Ti, Tj)
       0.0    0.0
C**   I         EC(T0) (N/m2) ! FOR SLAB
    9      2.59E10
C**   PHI(Ti, Tj)
       0.0 0.0 0.335 0.335
C**    CHI(Ti, Tj)     S(Ti, Tj)
       0.687   -0.31680E-04
C**   I         EC(T0) (N/m2) ! FOR COLUMN
      10        2.59E10
C**   PHI(Ti, Tj)
       0.0 0.335 0.335
C**    CHI(Ti, Tj)     S(Ti, Tj)
       0.687   -0.31680E-04

C**** FOR TIME INTERVAL NO. 11:
C**** CONTROL PARAMETERS.
    IBC = 1 ILOAD = 1 ITPND = 0 ITMP = 0 IFORM = 0 JOUT = 0
C**** BOUNDARY CONDITIONS.
C**   NBC     NBD(1)    NBD(2)    NBD(3)     BD(1)    BD(2)      BD(3)
      15       0          1         0
          END-OF-GROUP
C**** TIME-DEPENDENT MATERIAL PROPERTIES.
C**   FOR CONCRETE:
C**   I         EC(T0) (N/m2) ! FOR COLUMN
      1         3.19E10
C**   I         EC(T0) (N/m2) ! FOR SHORE
      2         7.75E9
C**   I         EC(T0) (N/m2) ! FOR  SLAB
      3         3.13E10
C**   I         EC(T0) (N/m2) ! FOR COLUMN
      4         3.13E10
C**   I         EC(T0) (N/m2) ! FOR SHORE
      5         7.75E9
C**   I         EC(T0) (N/m2) ! FOR  SLAB
      6         3.02E10
C**   I         EC(T0) (N/m2) ! FOR COLUMN
      7         3.02E10
C**   I         EC(T0) (N/m2) ! FOR SHORE
      8         7.75E9
C**   I         EC(T0) (N/m2) ! FOR SLAB
      9        2.76E10
C**   I         EC(T0) (N/m2) ! FOR COLUMN
      10       2.76E10
C**   I         EC(T0) (N/m2) ! FOR SHORE
      11         7.75E9
C**   I         EC(T0) (N/m2) ! FOR SLAB
      12         1.0E5
```

```
C**** APPLIED LOADS:
DISTRIBUTED LOADS:
C** IELEM  IDIR  DIST1    DIST2(m)    QL(N/m)   QR     QC
     19     0     0.0      1.5         3.89E3
     20     0     0.0      1.5         3.89E3
          END-OF-GROUP

C**** FOR TIME INTERVAL NO. 12:
C**** CONTROL PARAMETERS.
   IBC = 0  ILOAD = 1   ITPND = 1 ITMP = 0 IFORM = 0 JOUT = 0
C**** TIME-DEPENDENT MATERIAL PROPERTIES.
C**    FOR CONCRETE:
C**    I      EC(T0)  (N/m2) ! FOR COLUMN
       1      3.19E10
C**    PHI(Ti, Tj)
        0.0 1.012 1.012 0.832 0.832 0.712 0.660 0.660 0.509 0.509 0.432
0.432
C**     CHI(Ti, Tj)     S(Ti, Tj)
        0.807  -0.26779E-04
C**    I      EC(T0)  (N/m2) ! FOR SHORE
       2      7.75E9
C**    PHI(Ti, Tj)
        0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0
C**     CHI(Ti, Tj)     S(Ti, Tj)
        0.0   0.0
C**    I      EC(T0)  (N/m2) ! FOR SLAB
       3      3.13E10
C**    PHI(Ti, Tj)
       0.0 0.0 0.902 0.902 0.752 0.693 0.693 0.529 0.529 0.447 0.447
C**     CHI(Ti, Tj)     S(Ti, Tj)
        0.801    -0.34572E-04
C**    I      EC(T0)  (N/m2) ! FOR COLUMN
       4      3.13E10
C**    PHI(Ti, Tj)
       0.0 0.902 0.902 0.752 0.693 0.693 0.529 0.529 0.447 0.447
C**     CHI(Ti, Tj)     S(Ti, Tj)
        0.801    -0.34572E-04
C**    I      EC(T0)  (N/m2) ! FOR SHORE
       5      7.75E9
C**    PHI(Ti, Tj)
        0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0
C**     CHI(Ti, Tj)     S(Ti, Tj)
        0.0   0.0
C**    I      EC(T0)  (N/m2) ! FOR SLAB
       6      3.02E10
C**    PHI(Ti, Tj)
        0.0 0.0 0.833 0.752 0.752 0.558 0.558 0.469 0.469
C**     CHI(Ti, Tj)     S(Ti, Tj)
        0.785    -0.46341E-04
C**    I      EC(T0)  (N/m2) ! FOR COLUMN
       7      3.02E10
C**    PHI(Ti, Tj)
        0.0 0.833 0.752 0.752 0.558 0.558 0.469 0.469
C**     CHI(Ti, Tj)     S(Ti, Tj)
        0.785    -0.46341E-04
C**    I      EC(T0)  (N/m2) ! FOR SHORE
       8      7.75E9
```

```
C**    PHI(Ti, Tj)
        0.0 0.0 0.0 0.0 0.0 0.0
C**     CHI(Ti, Tj)     S(Ti, Tj)
        0.0    0.0
C**    I        EC(T0)  (N/m2)  ! FOR SLAB
       9      2.76E10
C**    PHI(Ti, Tj)
        0.0 0.0 0.619 0.619 0.509 0.509
C**     CHI(Ti, Tj)     S(Ti, Tj)
        0.728   -0.65341E-04
C**    I        EC(T0)  (N/m2)  (N/m2)  ! FOR COLUMN
       10      2.76E10
C**    PHI(Ti, Tj)
         0.0 0.619 0.619 0.509 0.509
C**     CHI(Ti, Tj)     S(Ti, Tj)
        0.728   -0.65341E-04
C**    I        EC(T0)  (N/m2)  ! FOR SHORE
       11       7.75E9
C**    PHI(Ti, Tj)
        0.0 0.0
C**     CHI(Ti, Tj)     S(Ti, Tj)
        0.0    0.0
C**    I        EC(T0)  (N/m2)  ! FOR SLAB
       12       1.0E5
C**    PHI(Ti, Tj)
        0.0 0.0
C**     CHI(Ti, Tj)     S(Ti, Tj)
        0.0    0.0
C**    I        EC(T0)  (N/m2)  ! FOR COLUMN
       13       1.0E5
C**    PHI(Ti, Tj)
        0.0
C**     CHI(Ti, Tj)     S(Ti, Tj)
        0.0    0.0
C**** APPLIED LOADS:
DISTRIBUTED LOADS:
C** IELEM  IDIR  DIST1   DIST2(m)   QL(N/m)  QR    QC
     21     1     0.0     2.8        6.0E3
          END-OF-GROUP

C**** FOR TIME INTERVAL NO. 13:
C**** CONTROL PARAMETERS.
   IBC = 0  ILOAD = 0   ITPND = 0 ITMP = 0 IFORM = 0 JOUT = 0
C**** TIME-DEPENDENT MATERIAL PROPERTIES.
C**    FOR CONCRETE:
C**    I        EC(T0)  (N/m2)  ! FOR COLUMN
       1        3.22E10
C**    I        EC(T0)  (N/m2)  ! FOR SHORE
       2        7.75E9
C**    I        EC(T0)  (N/m2)  ! FOR SLAB
       3        3.18E10
C**    I        EC(T0)  (N/m2)  ! FOR COLUMN
       4        3.18E10
C**    I        EC(T0)  (N/m2)  ! FOR SHORE
       5        7.75E9
C**    I        EC(T0)  (N/m2)  ! FOR SLAB
       6        3.11E10
```

```
C**     I       EC(T0) (N/m2) ! FOR COLUMN
        7       3.11E10
C**     I       EC(T0) (N/m2) ! FOR SHORE
        8       7.75E9
C**     I       EC(T0) (N/m2) ! FOR SLAB
        9       2.97E10
C**     I       EC(T0) (N/m2) ! FOR COLUMN
       10      2.97E10
C**     I       EC(T0) (N/m2) ! FOR SHORE
       11       7.75E9
C**     I       EC(T0) (N/m2) ! FOR SLAB
       12       2.59E10
C**     I       EC(T0) (N/m2) ! FOR COLUMN
       13       2.59E10
             END-OF-GROUP
```

APPENDIX B

INPUT DATA FILE FOR WALL FORMWORK

```
TITEL: WALL FORMWORK (CONSTRUCTION EXAMPLE USED
       IN SECTION 5.3)

C**** SET 1 :CONTROL PARAMETERS.
IOUT = 1  IUNITS = 1  NPOIN = 12  NELEM = 11  NSD = 11   NCTYP= 12
NSTYP = 1 MNSC = 3   MNCL = 2    MNPSL = 0   MNSL = 1   NLSTG = 11
IOWT = 0  IFRCT = 0  ITPND = 0   NONLIN = 0  IRELAX = 0 ITDATA = 0

C**** SET 2: MATERIAL PROPERTIES.
C**** FOR CONCRETE:
C**   I   ISTG   GAMA  ALFAT   FCT(N/m2)
      1    1     0.0    0.0     450E6
      2    1     0.0    0.0     2.5E6
      3    2     0.0    0.0     2.5E6
      4    3     0.0    0.0     2.5E6
      5    4     0.0    0.0     2.5E6
      6    5     0.0    0.0     2.5E6
      7    6     0.0    0.0     2.5E6
      8    7     0.0    0.0     2.5E6
      9    8     0.0    0.0     2.5E6
     10    9     0.0    0.0     2.5E6
     11   10     0.0    0.0     2.5E6
     12   11     0.0    0.0     2.5E6
         END-OF-GROUP
C**** FOR STEEL:
C**    I     ES(N/m2)    BETA1   ALFAT
       1     200.0E9      1.0
     END-OF-GROUP

C**** SET 3: COORDINATES OF NOODES.

C**   I        XI(I)(m)      YI(I)(m)
      1         0.00          0.00
      2         0.30          0.00
      3         0.90          0.00
      4         1.50          0.00
      5         2.10          0.00
      6         2.70          0.00
      7         3.30          0.00
      8         3.90          0.00
      9         4.50          0.00
     10         5.10          0.00
     11         5.70          0.00
     12         6.00          0.00
```

```
      END-OF-GROUP

C**** SET 5: TOPOLOGY OF MEMBERS AND SECTION DIMENSIONS.
C**** ELEMENT 1:
C**** CONTROL INFORMATION.
C**     IE  NOD(IE,1) NOD(IE,2) NSEC NCL NPSL NSL INTG ILSTG DO(m) SRM(m)
        1    1          2         3    2    0    1    1    1   0.1625
C**     NC      DX
        2     0.5
C**     IWSC(IE,I), I=1,NSEC
        1   2   3
C**     PART 1:  NC     ICTYP(1,1)     ICTYP(1,2)
                  1         2              0
C**   NSTRT     NEND      BTL (m)     BBL (m)     BTR     BBR     BTC     BBC
      1         3         1.000         1.000
C**   NSTRT     NEND      DCTL (m)  DL (m)      DCTR     DR     DCTC     DC
      1         3          0.000     0.300
C**     PART 2:  NC     ICTYP(1,1)     ICTYP(1,2)
                  1         1              0
C**   NSTRT     NEND      BTL (m)     BBL (m)     BTR     BBR     BTC     BBC
      1         3         1.000         1.000
C**   NSTRT     NEND      DCTL (m)  DL (m)      DCTR     DR     DCTC     DC
       1        3         0.300     0.025
                          END-OF-GROUP
C**** LAYOUT AND PROPERTIES OF NONPRESTRESSED STEEL LAYERS.
C** I INSTYP(I,1) INSTYP(I,2) NSTRT NEND  ASL(m2)    DSL(m) DSR DSC
    1       1          1             1     3   0.00114     0.24

C**** ELEMENT 2:
C**** CONTROL INFORMATION.
C**     IE  NOD(IE,1) NOD(IE,2) NSEC NCL NPSL NSL INTG ILSTG DO(m) SRM(m)
        2      2          3         3     2    0    1    1    1   0.1625
C**     NC      DX
        2     0.5
C**     IWSC(IE,I), I=1,NSEC
        1   2   3
C**     PART 1:  NC     ICTYP(1,1)     ICTYP(1,2)
                  1         3              0
C**   NSTRT     NEND      BTL (m)     BBL (m)     BTR     BBR     BTC     BBC
       1        3         1.000       1.000
C**   NSTRT     NEND      DCTL (m)  DL (m)      DCTR     DR     DCTC     DC
       1        3         0.000     0.300
C**     PART 2:  NC     ICTYP(1,1)     ICTYP(1,2)
                  1         1              0
C**   NSTRT     NEND      BTL (m)     BBL (m)     BTR     BBR     BTC     BBC
        1       3         1.000       1.000
C**   NSTRT     NEND      DCTL (m)  DL (m)      DCTR     DR     DCTC     DC
       1        3         0.300     0.025
                          END-OF-GROUP
C**** LAYOUT AND PROPERTIES OF NONPRESTRESSED STEEL LAYERS.
C** I INSTYP(I,1) INSTYP(I,2) NSTRT NEND  ASL(m2)    DSL(m) DSR DSC
    1       1          1             1     3   0.00114     0.240

C**** ELEMENT 3:
C**** CONTROL INFORMATION.
C**     IE  NOD(IE,1) NOD(IE,2) NSEC NCL NPSL NSL INTG ILSTG DO(m) SRM(m)
        3    3          4         3     2    0   1    1    1   0.1625
```

```
C**    NC      DX
       2    0.5
C**    IWSC(IE,I), I=1,NSEC
       1   2   3
C**    PART 1:  NC     ICTYP(1,1)     ICTYP(1,2)
                1         4               0
C**   NSTRT     NEND     BTL (m)    BBL (m)     BTR     BBR     BTC     BBC
       1        3        1.000       1.000
C**   NSTRT     NEND     DCTL (m)  DL (m)      DCTR     DR     DCTC     DC
       1        3        0.000      0.300
C**    PART 2:  NC     ICTYP(1,1)     ICTYP(1,2)
                1         1               0
C**   NSTRT     NEND     BTL (m)    BBL (m)     BTR     BBR     BTC     BBC
       1        3        1.000       1.000
C**   NSTRT     NEND     DCTL (m)  DL (m)      DCTR     DR     DCTC     DC
       1        3        0.300      0.025
                    END-OF-GROUP
C**** LAYOUT AND PROPERTIES OF NONPRESTRESSED STEEL LAYERS.
C** I INSTYP(I,1) INSTYP(I,2) NSTRT NEND  ASL(m2)    DSL(m) DSR DSC
    1       1             1       1    3   0.00114     0.240

C**** ELEMENT 4:
C**** CONTROL INFORMATION.
C**    IE  NOD(IE,1) NOD(IE,2) NSEC NCL NPSL NSL INTG ILSTG DO(m) SRM(m)
       4    4          5       3     2    0   1     1    1  0.1625
C**    NC      DX
       2    0.5
C**    IWSC(IE,I), I=1,NSEC
     1   2   3
C**    PART 1:  NC     ICTYP(1,1)     ICTYP(1,2)
                1         5               0
C**   NSTRT     NEND     BTL (m)    BBL (m)     BTR     BBR     BTC     BBC
       1        3        1.000       1.000
C**   NSTRT     NEND     DCTL (m)  DL (m)      DCTR     DR     DCTC     DC
       1        3        0.000     0.300
C**    PART 2:  NC     ICTYP(1,1)     ICTYP(1,2)
                1         1               0
C**   NSTRT     NEND     BTL (m)    BBL (m)     BTR     BBR     BTC     BBC
       1        3        1.000       1.000
C**   NSTRT     NEND     DCTL (m)  DL (m)      DCTR     DR     DCTC     DC
       1        3       0.300      0.025
                    END-OF-GROUP
C**** LAYOUT AND PROPERTIES OF NONPRESTRESSED STEEL LAYERS.
C** I INSTYP(I,1) INSTYP(I,2) NSTRT NEND  ASL(m2)    DSL(m) DSR DSC
    1       1             1        1    3   0.00114     0.240

C**** ELEMENT 5:
C**** CONTROL INFORMATION.
C**    IE  NOD(IE,1) NOD(IE,2) NSEC NCL NPSL NSL INTG ILSTG DO(m) SRM(m)
       5    5          6       3     2    0   1     1     1   0.1625
C**    NC      DX
       2    0.5
C**    IWSC(IE,I), I=1,NSEC
       1   2   3
C**    PART 1:  NC      ICTYP(1,1)     ICTYP(1,2)
                1         6               0
C**   NSTRT     NEND     BTL (m)    BBL (m)     BTR     BBR     BTC     BBC
```

```
      1        3        1.000        1.000
C**   NSTRT    NEND     DCTL (m)   DL (m)     DCTR    DR    DCTC    DC
      1        3        0.000      0.300
C**     PART 2:  NC     ICTYP(1,1)    ICTYP(1,2)
                 1         1             0
C**   NSTRT    NEND     BTL (m)    BBL (m)    BTR     BBR   BTC     BBC
      1        3        1.000        1.000
C**   NSTRT    NEND     DCTL (m)   DL (m)     DCTR    DR    DCTC    DC
     1        3         0.300      0.025
                    END-OF-GROUP
C**** LAYOUT AND PROPERTIES OF NONPRESTRESSED STEEL LAYERS.
C** I INSTYP(I,1) INSTYP(I,2) NSTRT NEND  ASL(m2)   DSL(m) DSR DSC
    1      1           1        1    3   0.00114    0.240

C**** ELEMENT 6:
C**** CONTROL INFORMATION.
C**    IE  NOD(IE,1) NOD(IE,2) NSEC NCL NPSL NSL INTG ILSTG DO(m) SRM(m)
       6     6          7        3    2    0    1   1    1   0.1625
C**    NC     DX
       2    0.5
C**    IWSC(IE,I), I=1,NSEC
       1   2   3
C**    PART 1:  NC     ICTYP(1,1)    ICTYP(1,2)
                1         7             0
C**   NSTRT    NEND     BTL (m)    BBL (m)    BTR     BBR   BTC     BBC
      1        3        1.000        1.000
C**   NSTRT    NEND     DCTL (m)   DL (m)     DCTR    DR    DCTC    DC
      1        3        0.000      0.300
C**    PART 2:  NC     ICTYP(1,1)    ICTYP(1,2)
                1         1             0
C**   NSTRT    NEND     BTL (m)    BBL (m)    BTR     BBR   BTC     BBC
      1        3        1.000        1.000
C**   NSTRT    NEND     DCTL (m)   DL (m)     DCTR    DR    DCTC    DC
      1        3        0.300        0.025
                    END-OF-GROUP
C**** LAYOUT AND PROPERTIES OF NONPRESTRESSED STEEL LAYERS.
C** I INSTYP(I,1) INSTYP(I,2) NSTRT NEND  ASL(m2)   DSL(m) DSR DSC
    1      1           1         1    3   0.00114    0.240

C**** ELEMENT 7:
C**** CONTROL INFORMATION.
C**    IE  NOD(IE,1) NOD(IE,2) NSEC NCL NPSL NSL INTG ILSTG DO(m) SRM(m)
       7     7          8        3    2    0    1   1    1   0.1625
C**    NC     DX
       2    0.5
C**    IWSC(IE,I), I=1,NSEC
       1   2  3
C**    PART 1:  NC     ICTYP(1,1)    ICTYP(1,2)
                1         8             0
C**   NSTRT    NEND     BTL (m)    BBL (m)    BTR     BBR   BTC     BBC
      1        3        1.000        1.000
C**   NSTRT    NEND     DCTL (m)   DL (m)     DCTR    DR    DCTC    DC
      1        3        0.000        0.300
C**    PART 2:  NC     ICTYP(1,1)    ICTYP(1,2)
                1         1             0
C**   NSTRT    NEND     BTL (m)    BBL (m)    BTR     BBR   BTC     BBC
      1        3        1.000        1.000
```

```
C**  NSTRT    NEND     DCTL (m)  DL (m)     DCTR     DR    DCTC     DC
       1       3       0.300     0.025
                     END-OF-GROUP
C**** LAYOUT AND PROPERTIES OF NONPRESTRESSED STEEL LAYERS.
C** I INSTYP(I,1) INSTYP(I,2) NSTRT NEND  ASL(m2)    DSL(m) DSR DSC
    1      1            1       1     3     0.00114     0.240

C**** ELEMENT 8:
C**** CONTROL INFORMATION.
C**   IE  NOD(IE,1) NOD(IE,2) NSEC NCL NPSL NSL INTG ILSTG DO(m) SRM(m)
      8     8          9        3    2    0   1    1    1   0.1625
C**   NC     DX
      2    0.5
C**   IWSC(IE,I), I=1,NSEC
      1   2  3
C**   PART 1:  NC    ICTYP(1,1)    ICTYP(1,2)
               1        9              0
C**  NSTRT    NEND     BTL (m)     BBL (m)     BTR     BBR     BTC     BBC
       1       3       1.000       1.000
C**  NSTRT    NEND     DCTL (m)  DL (m)     DCTR     DR    DCTC     DC
       1       3       0.000     0.300
C**   PART 2:  NC    ICTYP(1,1)    ICTYP(1,2)
               1        1              0
C**  NSTRT    NEND     BTL (m)     BBL (m)     BTR     BBR     BTC     BBC
       1       3       1.000       1.000
C**  NSTRT    NEND     DCTL (m)  DL (m)     DCTR     DR    DCTC     DC
       1       3       0.300     0.025
                     END-OF-GROUP
C**** LAYOUT AND PROPERTIES OF NONPRESTRESSED STEEL LAYERS.
C** I INSTYP(I,1) INSTYP(I,2) NSTRT NEND  ASL(m2)    DSL(m) DSR DSC
    1      1            1       1     3     0.00114     0.240


C**** ELEMENT 9:
C**** CONTROL INFORMATION.
C**   IE  NOD(IE,1) NOD(IE,2) NSEC NCL NPSL NSL INTG ILSTG DO(m) SRM(m)
      9     9         10        3    2    0   1    1    1   0.1625
C**   NC     DX
      2    0.5
C**   IWSC(IE,I), I=1,NSEC
      1   2   3
C**   PART 1:  NC    ICTYP(1,1)    ICTYP(1,2)
               1       10              0
C**  NSTRT    NEND     BTL (m)     BBL (m)     BTR     BBR     BTC     BBC
       1       3       1.000       1.000
C**  NSTRT    NEND     DCTL (m)  DL (m)     DCTR     DR    DCTC     DC
       1       3       0.000     0.300
C**   PART 2:  NC    ICTYP(1,1)    ICTYP(1,2)
               1        1              0
C**  NSTRT    NEND     BTL (m)     BBL (m)     BTR     BBR     BTC     BBC
       1       3       1.000       1.000
C**  NSTRT    NEND     DCTL (m)  DL (m)     DCTR     DR    DCTC     DC
       1       3       0.300     0.025
                     END-OF-GROUP
C**** LAYOUT AND PROPERTIES OF NONPRESTRESSED STEEL LAYERS.
C** I INSTYP(I,1) INSTYP(I,2) NSTRT NEND  ASL(m2)    DSL(m) DSR DSC
    1      1            1       1     3     0.00114     0.240
```

```
C**** ELEMENT 10:
C**** CONTROL INFORMATION.
C**    IE  NOD(IE,1)  NOD(IE,2)  NSEC NCL NPSL NSL INTG ILSTG DO(m)  SRM(m)
       10     10         11        3    2    0    1    1    1   0.1625
C**    NC      DX
       2     0.5
C**    IWSC(IE,I), I=1,NSEC
       1   2   3
C**    PART 1:  NC     ICTYP(1,1)     ICTYP(1,2)
                1          11              0
C**   NSTRT     NEND      BTL (m)     BBL (m)     BTR     BBR     BTC     BBC
       1        3        1.000       1.000
C**   NSTRT     NEND      DCTL (m)   DL (m)      DCTR     DR     DCTC     DC
        1        3        0.000      0.300
C**    PART 2:  NC     ICTYP(1,1)     ICTYP(1,2)
                1          1              0
C**   NSTRT     NEND      BTL (m)     BBL (m)     BTR     BBR     BTC     BBC
        1        3        1.000       1.000
C**   NSTRT     NEND      DCTL (m)   DL (m)      DCTR     DR     DCTC     DC
        1        3        0.300      0.025
                         END-OF-GROUP
C**** LAYOUT AND PROPERTIES OF NONPRESTRESSED STEEL LAYERS.
C** I INSTYP(I,1) INSTYP(I,2) NSTRT NEND  ASL(m2)    DSL(m) DSR DSC
    1      1            1       1    3    0.00114     0.240

C**** ELEMENT 11:
C**** CONTROL INFORMATION.
C**    IE  NOD(IE,1)  NOD(IE,2)  NSEC NCL NPSL NSL INTG ILSTG DO(m)  SRM(m)
       11     11         12        3    2    0    1    1    1   0.1625
C**    NC      DX
       2     0.5
C**    IWSC(IE,I), I=1,NSEC
     1   2   3
C**    PART 1:  NC     ICTYP(1,1)     ICTYP(1,2)
                1          12              0
C**   NSTRT     NEND      BTL (m)     BBL (m)     BTR     BBR     BTC     BBC
        1        3        1.000       1.000
C**   NSTRT     NEND      DCTL (m)   DL (m)      DCTR     DR     DCTC     DC
        1        3        0.000      0.300
C**    PART 2:  NC     ICTYP(1,1)     ICTYP(1,2)
                1          1              0
C**   NSTRT     NEND      BTL (m)     BBL (m)     BTR     BBR     BTC     BBC
        1        3        1.000       1.000
C**   NSTRT     NEND      DCTL (m)   DL (m)      DCTR     DR     DCTC     DC
        1        3        0.300      0.025
                         END-OF-GROUP
C**** LAYOUT AND PROPERTIES OF NONPRESTRESSED STEEL LAYERS.
C** I INSTYP(I,1) INSTYP(I,2) NSTRT NEND  ASL(m2)    DSL(m) DSR DSC
    1      1            1       1    3      0.00114     0.240
           END-OF-GROUP


C**** SET 6: TIME-DEPENDENT PARAMETERS.
C**** FOR TIME INTERVAL NO. 1:
C**** CONTROL PARAMETERS.
  IBC = 1  ILOAD = 1 ITPND = 0 ITMP = 0 IFORM = 0 JOUT = 0
```

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C**** BOUNDARY CONDITIONS.
C** NBC     NBD(1)    NBD(2)    NBD(3)      BD(1)     BD(2)     BD(3)
     1       0         0         1
     2       1         2         1          0.0      3.02E08    0.0
     3       1         2         1          0.0      3.02E08    0.0
     4       1         2         1          0.0      3.02E08    0.0
     5       1         2         1          0.0      3.02E08    0.0
     6       1         2         1          0.0      3.02E08    0.0
     7       1         2         1          0.0      3.02E08    0.0
     8       1         2         1          0.0      3.02E08    0.0
     9       1         2         1          0.0      3.02E08    0.0
    10       1         2         1          0.0 -    3.02E08    0.0
    11       1         2         1          0.0      3.02E08    0.0
          END-OF-GROUP
C**** TIME-DDEPENDENT MATERIAL PROPERTIES
C**    FOR CONCRETE:
C**    I       EC(T0)  (N/m2)
       1        200E9
C**    I       EC(T0)  (N/m2)
      2        1E-6
C**** APPLIED LOADS:
DISTRIBUTED LOADS:
C** IELEM  IDIR  DIST1    DIST2(m)    QL(N/m)   QR     QC
      1      0     0.00      0.30       6.87E3       1E-6
          END-OF-GROUP

C**** FOR TIME INTERVAL NO. 2:
C**** CONTROL PARAMETERS.
  IBC = 0  ILOAD = 1 ITPND = 0 ITMP = 0 IFORM = 0 JOUT = 0
C**** TIME-DEPENDENT MATERIAL PROPERTIES.
C**    FOR CONCRETE:
C**    I       EC(T0)  (N/m2)
       1        200E9
C**    I       EC(T0)  (N/m2)
       2        1.55E9
C**    I       EC(T0)  (N/m2)
       3        1E-6
C**** APPLIED LOADS:
DISTRIBUTED LOADS:
C** IELEM  IDIR  DIST1    DIST2(m)   QL(N/m)   QR     QC
     1      0     0.00       0.30     13.74E3   13.74E3
     2      0     0.00       0.60     13.74E3   1.00E-6
          END-OF-GROUP

C**** FOR TIME INTERVAL NO. 3:
C**** CONTROL PARAMETERS.
  IBC = 0  ILOAD = 1 ITPND = 0 ITMP = 0 IFORM = 0 JOUT = 0
C**** TIME-DEPENDENT MATERIAL PROPERTIES.
C**    FOR CONCRETE:
C**    I       EC(T0)  (N/m2)
       1        200E9
C**    I       EC(T0)  (N/m2)
       2        2.19E9
C**    I       EC(T0)  (N/m2)
       3       1.55E9
C**    I       EC(T0)  (N/m2)
       4        1.00E-6
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C**** APPLIED LOADS:
DISTRIBUTED LOADS:
C** IELEM   IDIR  DIST1    DIST2(m)  QL(N/m)       QR       QC
    1        0     0.00     0.30     13.74E3   13.74E3
    2        0     0.00     0.60     13.74E3   13.74E3
    3        0     0.00     0.60     13.74E3   1.00E-6
         END-OF-GROUP
C**** FOR TIME INTERVAL NO. 4:
C**** CONTROL PARAMETERS.
  IBC = 0  ILOAD = 1 ITPND = 0 ITMP = 0 IFORM = 0 JOUT = 0
C**** TIME-DEPENDENT MATERIAL PROPERTIES.
C**    FOR CONCRETE:
C**    I      EC(T0) (N/m2)
       1       200E9
C**    I      EC(T0) (N/m2)
       2       2.67E9
C**    I      EC(T0) (N/m2)
       3       2.19E9
C**    I      EC(T0) (N/m2)
       4      1.55E9
C**    I      EC(T0) (N/m2)
       5       1.00E-6
C**** APPLIED LOADS:
DISTRIBUTED LOADS:
C** IELEM   IDIR  DIST1    DIST2(m)  QL(N/m)       QR       QC
    1        0     0.00     0.30     13.74E3   13.74E3
    2        0     0.00     0.60     13.74E3   13.74E3
    3        0     0.00     0.60     13.74E3   13.74E3
    4        0     0.00     0.60     13.74E3   1.00E-6
         END-OF-GROUP

C**** FOR TIME INTERVAL NO. 5:
C**** CONTROL PARAMETERS.
  IBC = 0  ILOAD = 1 ITPND = 0 ITMP = 0 IFORM = 0 JOUT = 0
C**** TIME-DEPENDENT MATERIAL PROPERTIES.
C**    FOR CONCRETE:
C**    I      EC(T0) (N/m2)
       1       200E9
C**    I      EC(T0) (N/m2)
       2       3.08E9
C**    I      EC(T0) (N/m2)
       3       2.67E9
C**    I      EC(T0) (N/m2)
       4       2.19E9
C**    I      EC(T0) (N/m2)
       5      1.55E9
C**    I      EC(T0) (N/m2)
       6       1.00E-6
C**** APPLIED LOADS:
DISTRIBUTED LOADS:
C** IELEM   IDIR  DIST1    DIST2(m)  QL(N/m)       QR       QC
    1        0     0.00     0.14     10.53E3   10.53E3
    1        0     0.14     0.30     13.74E3   13.74E3
    2        0     0.00     0.60     13.74E3   13.74E3
    3        0     0.00     0.60     13.74E3   13.74E3
    4        0     0.00     0.60     13.74E3   13.74E3
    5        0     0.00     0.60     13.74E3   1.00E-6
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          END-OF-GROUP

C**** FOR TIME INTERVAL NO. 6:
C**** CONTROL PARAMETERS.
  IBC = 0  ILOAD = 1 ITPND = 0 ITMP = 0 IFORM = 0 JOUT = 0
C**** TIME-DEPENDENT MATERIAL PROPERTIES.
C**   FOR CONCRETE:
C**   I      EC(T0) (N/m2)
      1       200E9
C**   I      EC(T0) (N/m2)
      2       3.44E9
C**   I      EC(T0) (N/m2)
      3       3.08E9
C**   I      EC(T0) (N/m2)
      4       2.67E9
C**   I      EC(T0) (N/m2)
      5       2.19E9
C**   I      EC(T0) (N/m2)
      6      1.55E9
C**   I      EC(T0) (N/m2)
      7       1.00E-6
C**** APPLIED LOADS:
DISTRIBUTED LOADS:
C** IELEM  IDIR  DIST1   DIST2(m)  QL(N/m)       QR     QC
    1       0     0.14    0.30     1.00E-6   03.66E3
    2       0     0.00    0.44     03.66E3   13.74E3
    2       0     0.44    0.60     13.74E3   13.74E3
    3       0     0.00    0.60     13.74E3   13.74E3
    4       0     0.00    0.60     13.74E3   13.74E3
    5       0     0.00    0.60     13.74E3   13.74E3
    6       0     0.00    0.60     13.74E3   1.00E-6
          END-OF-GROUP

C**** FOR TIME INTERVAL NO. 7:
C**** CONTROL PARAMETERS.
  IBC = 0  ILOAD = 1 ITPND = 0 ITMP = 0 IFORM = 0 JOUT = 0
C**** TIME-DEPENDENT MATERIAL PROPERTIES.
C**   FOR CONCRETE:
C**   I      EC(T0) (N/m2)
      1       200E9
C**   I      EC(T0) (N/m2)
      2       3.76E9
C**   I      EC(T0) (N/m2)
      3       3.44E9
C**   I      EC(T0) (N/m2)
      4       3.08E9
C**   I      EC(T0) (N/m2)
      5       2.67E9
C**   I      EC(T0) (N/m2)
      6       2.19E9
C**   I      EC(T0) (N/m2)
      7      1.55E9
C**   I      EC(T0) (N/m2)
      8       1.00E-6
C**** APPLIED LOADS:
DISTRIBUTED LOADS:
C** IELEM  IDIR  DIST1   DIST2(m)  QL(N/m)       QR     QC
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```
    2      0     0.44     0.60     1.00E-6    03.66E3
    3      0     0.00     0.44     03.66E3    13.74E3
    3      0     0.44     0.60     13.74E3    13.74E3
    4      0     0.00     0.60     13.74E3    13.74E3
    5      0     0.00     0.60     13.74E3    13.74E3
    6      0     0.00     0.60     13.74E3    13.74E3
    7      0     0.00     0.60     13.74E3    1.00E-6
         END-OF-GROUP

C**** FOR TIME INTERVAL NO. 8:
C**** CONTROL PARAMETERS.
  IBC = 0  ILOAD = 1 ITPND = 0 ITMP = 0 IFORM = 0 JOUT = 0
C**** TIME-DEPENDENT MATERIAL PROPERTIES.
C**   FOR CONCRETE:
C**   I      EC(TO) (N/m2)
      1       200E9
C**   I      EC(TO) (N/m2)
      2       4.06E9
C**   I      EC(TO) (N/m2)
      3       3.76E9
C**   I      EC(TO) (N/m2)
      4       3.44E9
C**   I      EC(TO) (N/m2)
      5       3.08E9
C**   I      EC(TO) (N/m2)
      6       2.67E9
C**   I      EC(TO) (N/m2)
      7       2.19E9
C**   I      EC(TO) (N/m2)
      8      1.55E9
C**   I      EC(TO) (N/m2)
      9       1.00E-6
C**** APPLIED LOADS:
DISTRIBUTED LOADS:
C** IELEM  IDIR  DIST1    DIST2(m)  QL(N/m)      QR      QC
    3      0     0.44     0.60     1.00E-6    03.66E3
    4      0     0.00     0.44     03.66E3    13.74E3
    4      0     0.44     0.60     13.74E3    13.74E3
    5      0     0.00     0.60     13.74E3    13.74E3
    6      0     0.00     0.60     13.74E3    13.74E3
    7      0     0.00     0.60     13.74E3    13.74E3
    8      0     0.00     0.60     13.74E3    1.00E-6
         END-OF-GROUP

C**** FOR TIME INTERVAL NO. 9:
C**** CONTROL PARAMETERS.
  IBC = 0  ILOAD = 1 ITPND = 0 ITMP = 0 IFORM = 0 JOUT = 0

C**** TIME-DEPENDENT MATERIAL PROPERTIES.
C**   FOR CONCRETE:
C**   I      EC(TO) (N/m2)
      1       200E9
C**   I      EC(TO) (N/m2)
      2       4.33E9
C**   I      EC(TO) (N/m2)
      3       4.06E9
C**   I      EC(TO) (N/m2)
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       4        3.76E9
C**    I       EC(T0)  (N/m2)
       5        3.44E9
C**    I       EC(T0)  (N/m2)
       6        3.08E9
C**    I       EC(T0)  (N/m2)
       7        2.67E9
C**    I       EC(T0)  (N/m2)
       8        2.19E9
C**    I       EC(T0)  (N/m2)
       9       1.55E9
C**    I       EC(T0)  (N/m2)
      10        1.00E-6
C**** APPLIED LOADS:
DISTRIBUTED LOADS:
C** IELEM   IDIR   DIST1    DIST2(m)  QL(N/m)       QR       QC
    4       0      0.44     0.60     1.00E-6   03.66E3
    5       0      0.00     0.44     03.66E3   13.74E3
    5       0      0.44     0.60     13.74E3   13.74E3
    6       0      0.00     0.60     13.74E3   13.74E3
    7       0      0.00     0.60     13.74E3   13.74E3
    8       0      0.00     0.60     13.74E3   13.74E3
    9       0      0.00     0.60     13.74E3   1.00E-6
          END-OF-GROUP

C**** FOR TIME INTERVAL NO. 10:
C**** CONTROL PARAMETERS.
  IBC = 0   ILOAD = 1 ITPND = 0 ITMP = 0 IFORM = 0 JOUT = 0
C**** TIME-DEPENDENT MATERIAL PROPERTIES.
C**    FOR CONCRETE:
C**    I       EC(T0)  (N/m2)
       1        200E9
C**    I       EC(T0)  (N/m2)
       2        4.59E9
C**    I       EC(T0)  (N/m2)
       3        4.33E9
C**    I       EC(T0)  (N/m2)
       4        4.06E9
C**    I       EC(T0)  (N/m2)
       5        3.76E9
C**    I       EC(T0)  (N/m2)
       6        3.44E9
C**    I       EC(T0)  (N/m2)
       7        3.08E9
C**    I       EC(T0)  (N/m2)
       8        2.67E9
C**    I       EC(T0)  (N/m2)
       9        2.19E9
C**    I       EC(T0)  (N/m2)
      10       1.55E9
C**    I       EC(T0)  (N/m2)
      11        1.00E-6
C**** APPLIED LOADS:
DISTRIBUTED LOADS:
C** IELEM   IDIR   DIST1    DIST2(m)  QL(N/m)       QR       QC
    5       0      0.44     0.60     1.00E-6   03.66E3
    6       0      0.00     0.44     03.66E3   13.74E3
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    6        0      0.44     0.60    13.74E3    13.74E3
    7        0      0.00     0.60    13.74E3    13.74E3
    8        0      0.00     0.60    13.74E3    13.74E3
    9        0      0.00     0.60    13.74E3    13.74E3
   10        0      0.00     0.60    13.74E3    1.00E-6
           END-OF-GROUP

C**** FOR TIME INTERVAL NO. 11:
C**** CONTROL PARAMETERS.
  IBC = 0  ILOAD = 1 ITPND = 0 ITMP = 0 IFORM = 0 JOUT = 0
C**** TIME-DEPENDENT MATERIAL PROPERTIES.
C**   FOR CONCRETE:
C**     I       EC(T0) (N/m2)
        1        200E9
C**     I       EC(T0) (N/m2)
        2        4.71E9
C**     I       EC(T0) (N/m2)
        3        4.46E9
C**     I       EC(T0) (N/m2)
        4        4.20E9
C**     I       EC(T0) (N/m2)
        5        3.91E9
C**     I       EC(T0) (N/m2)
        6        3.61E9
C**     I       EC(T0) (N/m2)
        7        3.27E9
C**     I       EC(T0) (N/m2)
        8        2.89E9
C**     I       EC(T0) (N/m2)
        9        2.44E9
C**     I       EC(T0) (N/m2)
       10        1.89E9
C**     I       EC(T0) (N/m2)
       11       1.1E9
C**     I       EC(T0) (N/m2)
       12        1.00E-6
C**** APPLIED LOADS:
DISTRIBUTED LOADS:
C** IELEM   IDIR   DIST1    DIST2(m)  QL(N/m)       QR      QC
    6        0      0.44     0.60    1.00E-6    03.66E3
    7        0      0.00     0.14    03.66E3    06.87E3
    7        0      0.14     0.60    06.87E3    06.87E3
    8        0      0.00     0.60    06.87E3    06.87E3
    9        0      0.00     0.60    06.87E3    06.87E3
   10        0      0.00     0.60    06.87E3    06.87E3
   11        0      0.00     0.30    06.87E3    1.00E-6

           END-OF-GROUP
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GLOSSARY

This glossary provides some basic definition for formwork terms used in this thesis. Definitions are taken from ***Formwork for Concrete*** (Hurd, 1995).

BACKSHORE
: Shore placed snugly under a concrete slab or structural member after the original formwork and shores have been removed from a small area without allowing the slab or member to deflect or support its own weight or existing construction loads from above.

BULKHEAD
: A partition in the forms blocking fresh concrete from a section of the forms or closing the end of a form, such as at a construction joint.

FALSEWORK
: The temporary structure erected to support work in the process of construction. In discussion of concrete construction, the term may be used much the same as formwork to include shores or vertical posts, forms for beams or slabs, and lateral bracing

FORM
: A temporary structure or mold for the support of concrete while it is setting and gaining sufficient strength to be self supporting; sometimes used interchangeably with formwork, but also used in a more restricted sense to indicate supporting members in direct contact with the freshly placed concrete.

FORMWORK
: The total system of support for freshly placed concrete including the mold or sheathing which contacts the concrete as well as all supporting members, hardware, and necessary bracing.

JOIST
: A horizontal structural member supporting decks from sheathing; usually rests on stringers or ledgers.

PRESHORES
: Added shores placed snugly under selected panels of a deck forming system before any primary (original) shores are removed. Preshores and the panels they support remain in place until the remainder of the bay has stripped and backshored, a small area at a time.

RESHORES, RESHORING
: Shores placed snugly under a concrete slab or other structural member after the original forms and shores have been removed from a large area, thus requiring a

new slab or structural member to support its own weight and existing construction loads applied prior to installation of reshores.

SHEATHING
: The material forming the contact faces of forms; also called lagging, sheeting.

SHORE
: Temporary vertical or inclined support for formwork and fresh concrete or for recently built structures which have not developed full design strength. Also called prop, tom, post, strut.

SHORING
: System of vertical or inclined supports for forms; may be wood or metal posts, scaffold-type frames, or various patented members.

STRINGER
: Horizontal structural member usually (in slab forming) supporting joists and resting on vertical supports.

STUD
: Vertical supporting member to which sheathing is attached.

TIE
: A concrete form tie is a tensile unit adapted to holding concrete forms secure against the lateral pressure of unhardened concrete, with or without provision for spacing the forms a definite distance apart, and with or without provision for removal of metal to a specified distance back from the finished concrete surface.

WALE
: Long horizontal member (usually double) used to hold studs in position; also called waler, ranger.