ABSTRACT

Lewis, David Dylan. DropHead Formwork System Implications in Flat Plate Concrete Floor Construction. (Under the direction of Dr. David W. Johnston.)

This study analyzed aspects of shoring and reshoring of multi-story concrete construction using retractable dropheads for the temporary support of the flat plate floor slabs. A comparison was made between this method and the traditional method using removed shores. The objective was to determine the ability of the slabs to support construction loads at the stage of very early age removal of the forming panels. The strength evaluations were conducted considering punching shear, flexural strength, and beam shear. The results were presented as a series of charts showing the structural capacities of a slab for various slab depths and concrete compressive strengths attained. An analysis is also presented of the distribution of the construction loads throughout a structure using both the traditional method and the method with the retractable dropheads. An additional analysis was also done considering the effects of having various percentages of slab activation due to partial rather than full release of the shores.

DROPHEAD FORMWORK SYSTEM IMPLICATIONS IN FLAT PLATE CONCRETE FLOOR CONSTRUCTION

by

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A Thesis submitted to the Graduate Faculty of
North Carolina State University in partial fulfillment of
the requirements of the degree of Master of Science

CIVIL ENGINEERING

Raleigh, North Carolina

July 2005

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BIOGRAPHY

David D. Lewis was born in Baton Rouge, Louisiana on December 26, 1978. He grew up in the Denham Springs suburb of Baton Rouge and graduated from Denham Springs High School in May of 1996. He enrolled in Louisiana State University in the fall of 1996 where he received his Bachelor of Science in Civil Engineering in December of 2002 from the Department of Civil and Environmental Engineering at Louisiana State University. He also received a minor in both structural engineering and construction management from Louisiana State University.

While attending Louisiana State University, he participated in a co-operative education experience that allowed him to work with an engineering firm for a semester. He also participated in an independent study program for senior class credit, which included conducting research on piping materials for the Louisiana Department of Transportation and Development. For summer work, he helped to conduct a survey on building codes that have been adopted across the state of Louisiana.

In January of 2003, he enrolled in the Construction Engineering and Management program in the Department of Civil, Construction, and Environmental Engineering at North Carolina State University. A continued interest in the construction process led to a research project with Dr. David W. Johnston involving shoring and reshoring, which resulted in the thesis presented here.

ACKNOWLEDGEMENTS

First and foremost, I would like to express thanks and gratitude for the direction and guidance by my Lord and Savior, Jesus Christ. I attribute all successes to Him, whether in my past, present, or future. I know that this stage in my life is one that the Lord had ordained for me and His grace upon my life is the only reason that I was able to accomplish something that would have been impossible without Him. Philippians 4:13

I would also like to express my love and appreciation to my wife, Jennie Marie Lewis, who has had to endure with me the process of completing this task. I know that times have been tough, both financially and emotionally and your continued support, whether verbal or non-verbal, has meant the world to me. Thank you, my darling.

I would like to express my deepest appreciation and gratitude to my advisor, Dr. David W. Johnston. His direction, guidance, and most of all, his patience with me in the development of this thesis is deserving of my utmost respect. Thank you.

Further gratitude is extended to the other members of the thesis committee, Dr. Leming and Dr. Kowalsky for their review of the literature and their constructive suggestions.

I offer a special note of thanks to Dr. William Rasdorf for his invaluable advice and encouragement along the way. Thank you for caring enough to reach out.

Lastly, to the many other friends and family who offered their time and encouragement, I extend a sincere Thank You and God Bless.

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1. INTRODUCTION

1.1 Background

When erecting any concrete structure, there must be formwork in order to shape and mold the concrete into its final form. In multi-story building construction, formwork underneath the floor being cast serves as the mold and vertical elements called shores support the forms from the previously constructed floors below.

Since the loads of the fresh concrete, formwork, placing personnel, and placing equipment for a floor are typically greater than the load capacity of the floor below, a system of shores and reshores is used to distribute the loads over several lower floors. In the traditional forming/shoring method, the formwork is removed when the concrete slab that is being supported has gained enough strength to carry its own weight, and reshores are inserted between floors to distribute added construction loads to several floors below.

Throughout the history of concrete construction, there have been occasional building failures during construction caused by inadequate formwork or insufficient shoring and reshoring. The cause has often been the premature removal of the concrete floor temporary support or inadequate analysis of the early age strength of the concrete floor system. Concrete has lower strength at early ages. This must be taken into consideration when designing the formwork system and requires the contractor to be knowledgeable about the strengths of the concrete at early ages. The principal properties of concern are the flexural and shear strength of the concrete floor system.

The strength of a concrete slab is dependent upon many things such as the proportions of the mixture, the water to cement ratio, the type of cement, and curing temperature. The two most critical factors that affect the early strength of a concrete slab are the age of the concrete and the curing temperature of the concrete. These two factors determine the rate of strength gain and the maturity of the concrete at a given time. The higher the temperature of the concrete, the faster a concrete section will gain maturity and therefore gain strength. Maturity increases with age, but at a higher rate with higher concrete temperatures and a lower rate with lower concrete temperatures.

New methods have been developed for the shoring and reshoring process, which improve the economy of the construction process by allowing removal and reuse of some forming elements at an earlier concrete age. An example is a forming system that allows the forming panels and panel support beams to be removed by lowering the head of the shore, while leaving the loaded shore in place as shown in Figure 1.1.

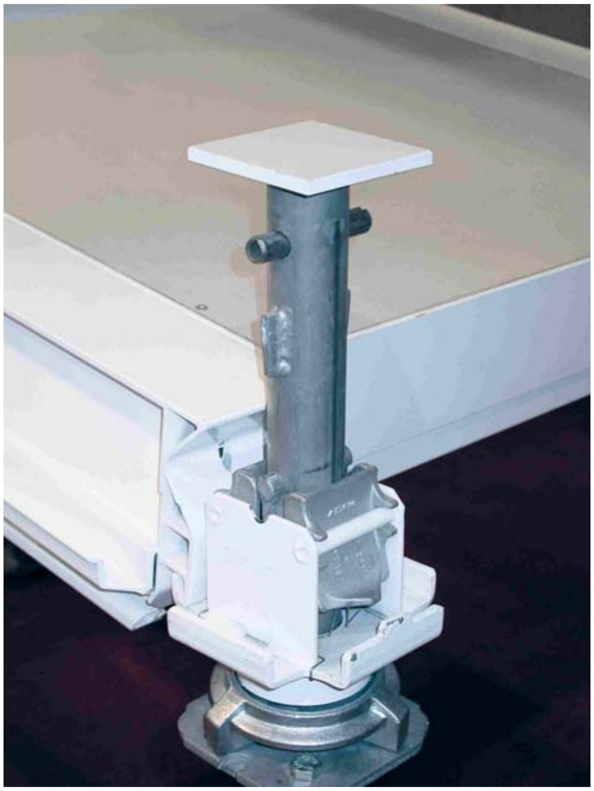


Figure 1.1 – Drophead (courtesy of Meva)

As in any industry, new and improved construction methods must be introduced in order to have a safer and more economical construction process. An example of this is the Drophead Shore Formwork System. This report introduces this new method and studies the effects it has upon the structural integrity of a slab during construction.

1.2 Problem Statement

Knowledge among structural designers of the shoring and reshoring process for construction of multi-story buildings is often limited to traditional methods, which involve complete removal of the shores and forming systems at certain stages of the process. New, more efficient and safe systems and methods have been introduced which allow removal of the forming elements while the shores remain in place. An example is the MevaDec Drophead Formwork System for slabs. Structural engineers and construction engineers need information and analytical procedures, which will help them to understand how such systems perform in relation to building code standards for design of concrete structures and construction safety standards and regulations for construction of buildings.

1.3 Research Objectives

The objectives of this research are the following:

1. Examine the construction loads on the slabs that span between shores and how those loads are applied during the construction process.
2. Evaluate the strength of the early age concrete floor systems during the construction process.

3. Compare traditional methods of shoring and reshoring for multistory concrete construction with the Drophead Shore method.
4. Investigate the early age strength of concrete slabs for resisting loads upon stripping of the formwork panels while shores remain in place.
5. Investigate the effects of having various percentages of slab activation due to partial rather than full release of the shores.

1.4 Research Significance

Both safety and economy are critical in concrete construction for multistory buildings. The Drophead Shore Formwork System for slab construction offers the possibility of increasing safety by keeping the shores in place while the forms can be stripped and reused more rapidly. This allows the concrete to mature for a longer period of time before it is required to support major loads. Economy results from both the increased safety and more rapid reuse of the formwork elements. The information developed also aids the engineer of record for the building structure to better understand, evaluate, and approve (as appropriate) the construction process designed by the construction engineer.

2. REVIEW OF LITERATURE

2.1 Introduction

In this chapter, a summary of literature reviewed concerning multistory concrete slab shoring and reshoring is presented. Information from industry standards regarding the recommended and required loadings is presented. Analytical methods for determining how these loads are distributed through, and resisted by, a multistory concrete structure will be discussed. Finally, literature discussing the rate at which a concrete slab matures or gains compressive strength and how this strength gain is related to punching shear, tensile strength, and bond strength will be introduced.

2.2 Construction Loads

Before undertaking an analysis, the loads on a particular structure during construction must be determined. The loads applied to a structure during construction are different from the loads encountered during the service of the building. There are several resources for reference in determining appropriate loads on buildings during construction.

OSHA[11], the Occupational Safety and Health Administration of the United States Department of Labor, provides rules and regulations in Section 1926 of the Code of Federal Regulations governing the safety standards for the construction industry. Subpart Q on Concrete and Masonry Construction Section 1926.703 (a), states that “Formwork shall be designed, fabricated, erected, supported, braced and maintained so that it will be capable of supporting, without failure, all vertical and lateral loads that may reasonably be anticipated to be applied to the formwork.” Removal of all formwork should only occur when the concrete has gained sufficient strength to support its own weight plus any loads that may be imposed upon the slab. Section 1926.703 (e) states that all forms and shores are not to be removed until the employer determines that the concrete has gained sufficient strength to support its weight and any superimposed loads. OSHA[11] does not necessarily list requirements for live load allowances, but it does expect the designer to take all precautions necessary to prepare for any possible live loads that might occur. Several non-mandatory references are listed in Appendix A to Subpart Q. Among those are ACI 347[6], ACI SP-4[7], and ANSI A10.9[12].

OSHA[11] lists guidelines in the non-mandatory appendices concerning scaffold specifications and intended loads for various applications. Although scaffolds are not the same as formwork or shores/reshores, they are similar systems. The scaffold guidelines (Table 2.1) provide possible live load levels that could be imposed on the structure.

Table 2.1 Minimum loading guidelines for scaffolds

Rated Load Capacity	Intended Load	Application
Light-Duty	25 psf	Uniformly over the entire span
Medium-Duty	50 psf	Uniformly over the entire span
Heavy-Duty	75 psf	Uniformly over the entire span
One-person	250 lbs	Concentrated at the center of the span
Two-person	250 lbs	Concentrated 18 inches to the left or right of the center of the span
Three-person	250 lbs	Combination of one-person and two-person application

ACI SP-4[7] is a manual for the design and construction of formwork for concrete. It provides guidelines for an engineer to determine the loadings on which to base a form design under normal conditions. Formwork for floor system construction can vary from 3 to 15 psf in weight. Concrete for the floor system usually has a density of 145 to 150 pcf including an allowance for normal reinforcement. Fortunately, these dead loads can be estimated fairly well and used directly for analysis purposes.

ACI 347-04[6] recommends that slab formwork be designed for a minimum live load of 50 psf occurring during placing of the concrete. This would provide for the weight of the workers, and any small equipment used. If motorized carts are used, the minimum live load should be 75 psf. Including a minimum dead load of 50 psf, the minimum combined dead and live load should be no less than 100 psf, and when motorized carts are used, it should be no less than 125 psf. ACI SP-4[7] contains guidelines for determining the distribution of those loads in the multistory structure under construction. An application of that analysis procedure is presented in Chapter 3 of this report.

ASCE 37[9] provides requirements for loads on structures during construction and lists the minimum concentrated personnel and equipment loads for design. A minimum concentrated load of 250 lbs (1.1 kN) should be used in strength calculations of individual structural members. If the actual loads exceed this minimum requirement, the actual loads should be used. For working surfaces or areas, it is traditional to use a uniformly distributed load for design purposes. ASCE 37[9] uses 4 basic classes: very light duty, light duty, medium duty, and heavy duty. Table 2.2 shows the minimum requirements for combined loads on working surfaces. It is important to note that these loads are the combination of personnel, equipment, and material in transit or staging.

It is also clearly stated how each operational class is defined. Very light duty is defined as sparsely populated personnel with very small amounts of construction materials. Light duty is defined as light frame construction or concrete transport and placement by hose. It also includes any concrete finishing with hand tools. Medium duty is defined as average construction and more specifically includes the transportation of concrete using buckets, chutes, or handcarts. It also includes any masonry or structural steel construction along with rebar placement. Although OSHA[11] does not give specific definitions of the duty classifications in Table 2.1, OSHA[11] and SEI/ASCE 37[9] are consistent in the requirements and they are also in general agreement with ACI 347-04[6].

Table 2.2 Classes of working surfaces for combined uniformly distributed live loads (ASCE 37-02)

Operational Class	Uniform Load	
	psf	kN/mm^2
Very Light Duty	20	0.96
Light Duty	25	1.20
Medium Duty	50	2.40
Heavy Duty	75	3.59

ANSI[12] specifies a minimum formwork dead load of 10 psf. ANSI[12] also incorporates many other of the recommendations of ACI 347[6] as requirements. If motorized carts are used, the live load allowance shall increase by an additional 25 psf, for a total vertical load of 125 psf. The total vertical load shall not be less than 100 psf.

2.3 Shoring Analysis Methods

Grundy & Kabaila[10] presented a method for determining the erection, or construction, loads for flat slabs. As mentioned earlier, these loads normally exceed the service loads of the individual building slabs. The process of shoring different numbers of floors and the effects on the construction load distribution are discussed. The analytical procedure is based on several assumptions. First, the shores in place are infinitely rigid in comparison to the slab vertical deflection. Second, it is assumed that the shores are so closely spaced that the vertical force from the shores can be considered a distributed load instead of a concentrated load. Finally, a typical construction cycle usually rises at a rate of one floor per week.

In their example analysis of the construction loads applied to slabs, 3 sets of shores were used. The analysis showed that peak or maximum applied slab loads occurred at a time in which all available shoring sets were being used to support the floors below. Continued analysis was conducted using varying amounts of shoring sets. It was concluded that although increasing the number of shored levels does bring about an increase in the maximum applied slab loads, it also delays the occurrence of the maximum slab loads allowing more time to develop greater strength. This seems to be in agreement with ACI 347[6].

ACI 347[6] also states that increasing the number of reshored floors decreases the maximum load applied to the slab. This is because the reshores allow for a greater distribution of the loads throughout the floors.

ACI SP-4[7] lists recommended assumptions for analysis of the shoring and reshoring process.

1. The shores and reshores are infinitely stiff relative to the slabs.
2. Any slabs that are interconnected by shores all deflect equally when a new load is added, and carry a share of the added load in proportion to their relative stiffnesses.
3. All of the slabs have equal stiffness, therefore, added loads are shared equally by the interconnected slabs
4. The ground level floor or any other base support is assumed to be rigid.

These assumptions are not perfectly true. However, actual field measurements have indicated that any error introduced from these assumptions is generally small and normally neglected.

2.4 Load Factors

After over 10 years of development, SEI/ASCE 37[9] was released in 2002 as a standard for determining construction loads and load factors. It provides a detailed description of possible load factors. Table 2.3 lists the load factors that are applicable to the research presented in the following chapters.

Table 2.3 Construction load factors

Load	Description	Load Factor (max)
D	Permanent Structure Dead Load	0.9 (when counteracting with wind or seismic loads) 1.4 (when combined with only construction or material load) 1.2 (for all other combinations)
C_D	Temporary Structure Dead Load	0.9 (when counteracting with wind or seismic loads) 1.4 (when combined with only construction or material load) 1.2 (for all other combinations)
C_{FML}	Fixed Material Dead Load	1.2
C_{VML}	Variable Material Dead Load	1.4
C_P	Personnel and Equipment Load	1.6

During the building construction process, the erected formwork and shoring are temporary structures and fresh concrete is a fixed material. When the concrete becomes part of the system supporting construction loads, it is permanent structure dead load.

For the shoring and reshoring process, the applicable loading combination is most typically:

$$U_C = 1.2D + 1.2C_D + 1.2C_{FML} + 1.4C_{VML} + 1.6C_P \qquad (2.1)$$

This is consistent with the provisions of ACI 318-05 for the permanent structure of U = 1.2D + 1.6L, a reduction from the previous requirement of U = 1.4D + 1.7L, where L is the permanent structure live load. The reduction in the factor of safety for the permanent structure has implications for construction process safety that benefited in the past from the extra structural capacity, which has sometimes covered an insufficient engineering of the shoring and reshoring operations. The loss of this extra capacity increases the importance of engineering the construction process.

2.5 Strength Evaluation

ACI 318-05[8] is a building code requirement for structural concrete members. This report will use the equations and limitations from ACI 318-05[8] for any area of structural analysis that is investigated. The particular areas of concern are punching shear, flexure, and beam shear. Table 2.4 lists the chapters and sections from which pertinent information will be taken.

Table 2.4 ACI 318-05 references

Description	Chapter	Section
Punching Shear (reinforced)	11	12
Punching Shear (plain concrete)	22	5
Flexure (reinforced)	9	1
Flexure (plain concrete)	22	5
Beam Shear	11	3

2.5.1 Tensile Strength

An estimate of the tensile strength at early ages is required for the analysis of cracking strength. Neville[3] indicates the tensile strength is often expressed by researchers as a function of the compressive strength.

$$f_t = k(f'_c)^n \quad (2.2)$$

Various researchers and practitioners have used a range of values for both coefficients k and n. The exponent coefficient n often ranges from ½ to ¾ and the coefficient k often ranges from 5 to 12 with an average value of 8.3. There are many factors controlling this relationship, which is unique for each set of raw materials. Neville[13] earlier suggested

$$f_t = 9.5(f'_c)^{0.5} \quad (2.3)$$

which has coefficients within the often used ranges.

2.5.2 Bond

Bond strength can also be an important factor due to the possibility of experiencing a loss of bond strength due to early loading of a slab. Although ACI 318-05[8] gives various requirements for bonding, it does not specifically mention limitations for bond strength during the curing stages of concrete.

Clark and Johnston[1] examined the effects on bond strength due to early loading. For all of the beam samples tested, the same beam size, concrete mixture, and reinforcement size were used. The 28-day concrete strength and reinforcement strength used were 4000 psi and 60 ksi respectively. Three samples were cast for each embedment length used. One was loaded to ultimate bond capacity after one day of curing. The second sample was loaded after one day and was maintained for the remainder of the curing process. The third sample had no early loading and was loaded to failure after the sample had gained full strength. The amount of movement was measured as slip at both the loaded end and the free end of the beam sample. It was found that early loading with a suitable factor of safety resulted in no detrimental effects on the 28-day ultimate concrete bond strength. Under slip criteria, early loading would result in a reduced critical bond stress. Also, slip from either the loaded end or the free end was greater. This was due to creep that took place after the early loading had occurred. This greater amount of slip would therefore result in larger permanent deformations. However, due to the autogenous healing and early wedging later slip increase with load was reduced and the total slip that occurred was comparable to the slip that resulted from loading only after 28-day strength had been obtained.

3. SHORING ANALYSIS

3.1 <u>Introduction</u>

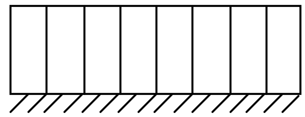
For multi-story concrete construction, freshly cast-in-place floors must be temporarily supported. This temporary system includes the formwork, shoring, and reshoring. Shores are the temporary vertical elements used to transfer the weight of a freshly cast floor to the previously cast floors below. The formwork system is usually made of wood, steel, and aluminum framing elements including panels, joists, beams, shores, and lateral bracing.

During construction of concrete buildings, it is necessary to share the construction loads of workers, materials, forms, and freshly placed and early age concrete over several previously cast floors. The loads at the upper level during construction are typically greater than the single floor design strength of the completed structure and the strength is less due to the early age of the concrete. Distributing the loads over several lower floors can achieve a state that is safe for the workers and avoids damage to the structure. A typical traditional construction cycle would often include one set of shores and two sets of reshores. However, this depends upon the rate of strength gain of the concrete used, the loadings for which the structure was designed, and the length of time available for the project.

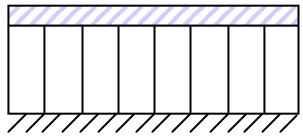
3.1.1 <u>Description of Traditional Method</u>

There are four main steps in a typical construction cycle for a multistory concrete building.

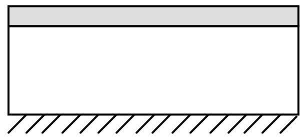
1. The installation of the formwork system including the shores



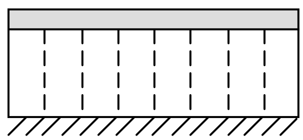
2. The casting of the floor slab



3. The removal of the formwork and shoring system



4. The placement of the reshoring system (this step may include the removal of a set of reshores from a lower level).



During construction, the early age concrete floors have typically not yet reached their design strength. It is also typical that the loads imposed upon the last previously cast concrete floor will exceed the service loads for which the floor was designed. Therefore, shores and reshores must be used to distribute the imposed loads over several floors below. Before any shores or formwork can be removed, the slab has to have at least gained enough strength to support its own self-weight and any loads imposed upon it. Once the shoring system has been removed, the concrete floor can be considered to be an activated slab. For continued support, a set of reshores is placed under that particular slab. The reshores should only be installed snug tight. They are not jacked upward so that their starting force is known to be zero, whereas if jacked, there would not be a way to know the force carried by the shores and relieved from the slab.

3.1.2 <u>Description of the New Method</u>

A new shoring system used is one that involves steel shores with a retractable head known as a drophead, aluminum beams, and aluminum framed panels with an integrated plastic forming face. The top of the shore has a plate that is in direct contact with the placed concrete as shown in Figure 3.1. The shore head to which the beams are attached can be dropped allowing the beams and panels to be removed while the shore remains in place.

Figure 3.1 Drophead shore with removable beams and panels (courtesy of Meva)

Early stripping allows earlier re-use of the panels and beams, which can reduce the material needed on site. The mechanization of the system reduces labor force requirements. It is important to emphasize that the steel shores used in the support of the slab do not need to be removed during the early stripping. Once the dropheads are released, the panels and beams can be removed while leaving the slab supported by the steel shores. Essentially, this system allows removal of the panels used to form the concrete without causing the slab to support its own weight, except for the short span between the shores. This in turn affects the speed at which new floors can be constructed.

One of the forming arrangements that can be used is noted as the Drophead-beam-panel-method. In this method, the ready-made panels are inserted between the rows of primary beams, which are supported by the adjustable shores attached to the removable drophead. An example of the arrangement of the panels and beams in this approach is shown in Figure 3.2.

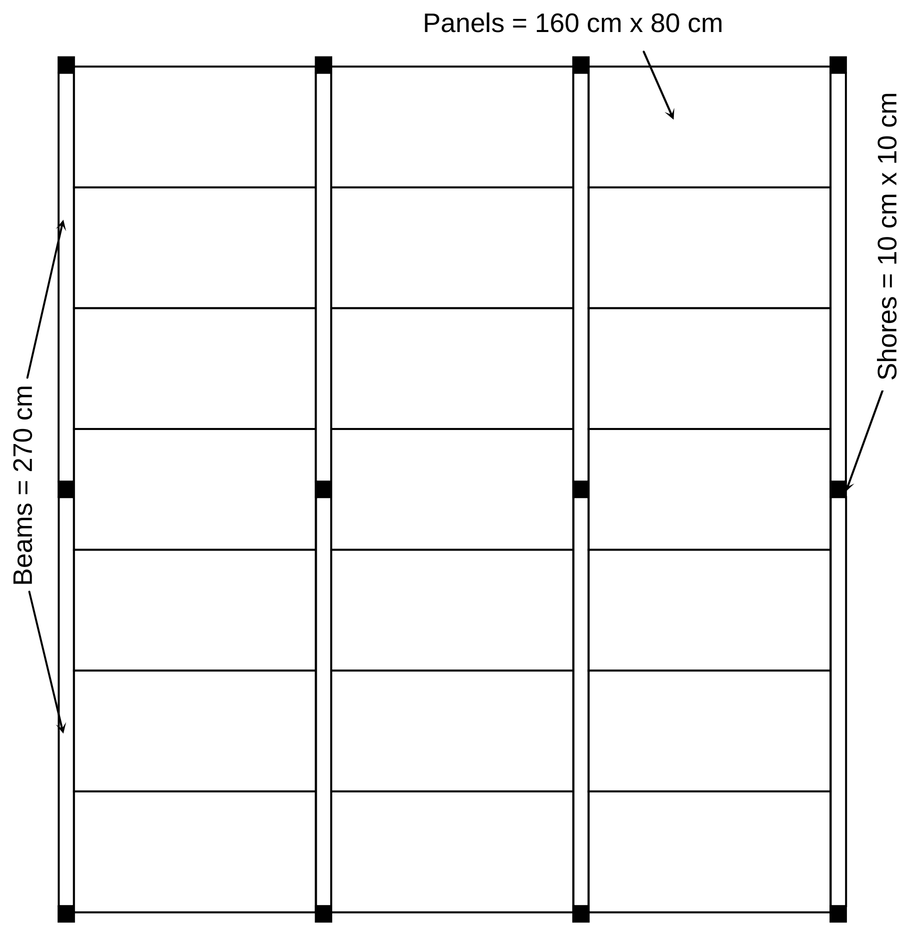


Figure 3.2 Drophead-beam-panel method

With this method, the layout should begin in a corner and it is best, in general, to assemble the rows of beams in a perpendicular direction to the longest wall. The beams range in length from 80 cm (2.6 ft) to 270 cm (8.85 ft). The specific layout of the beams and panels can be determined on site, but are best determined through pre-planning. Also, the secondary beams can be used as supplemental supports to help customize the beam/panel layout around columns and similar irregular areas.

3.2 Construction Load Analysis

3.2.1 Assumptions

During the construction of a building, any load that is imposed upon the partially completed structure would be considered a construction load. As previously mentioned, these construction loads can exceed the service loads for which the individual floors were designed. In addition, during construction the age of the concrete floors below is such that they have often not attained their full 28-day strength. Therefore, an analysis must be made of the loads imposed upon the structure during the construction process. Some of the possible construction loads include vertical loads imposed by the structure self weight, temporary support systems (forms), personnel, equipment, and construction materials.

Once an estimate of the loads imposed upon the structure is determined, an analysis of the temporary construction/shoring system can be made. This analysis is the key to understanding what is actually happening during the construction process. Using the steps mentioned previously and transferring all of the loads estimated into terms of the dead weight of the concrete slab, one can follow the loads through the formwork system. As mentioned in Chapter 2, ACI SP-4[7] lists the following assumptions that are the basis of this analysis.

1. The shores and reshores are infinitely stiff relative to the slabs.
2. Any slabs that are interconnected by shores all deflect equally when a new load is added and carry a share of the added load in proportion to their relative stiffness.
3. All of the slabs have equal stiffness; therefore, the interconnected slabs share added loads equally.
4. The ground level floor or any other base support is assumed to be rigid.

As with all assumptions made for an analysis, they are not exact. However, actual field measurements have indicated that any error introduced from these assumptions is generally small and normally neglected.

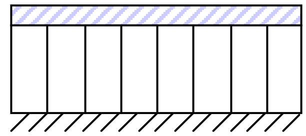
3.2.2 Traditional Analysis

Four loads are typically considered in the analysis of the shoring process. They include the slab weight, the form and shore weight, the construction live load, and the weight of the reshores, all expressed as uniform distributed loads. Each of these loads is expressed as a proportion of the dead weight of the concrete slab, D. For the example where the slab weighs 100 psf, forms and shores weigh 10 psf, reshores weigh 5 psf, and the construction live load is 50 psf, the loads expressed as a proportion of D are:

Slab Weight	1.000
Form/shore Weight	0.100
Construction Live Load	0.500
Reshore Weight	0.050

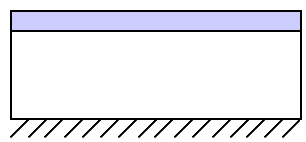
Using the assumptions of ACI SP-4[7], the loads are transferred, as they are applied, through the shores, reshores, and the slabs.

Slab #	Load Carried by Slab			Load Carried by Shore
	Begin	Change	End	
1	0.000	0.000	0.000	
				1.600



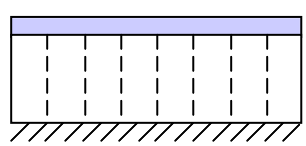
Initially, the shoring system must support all of the loads that are applied from the placement of the first slab. The loads that have been applied in the above figure include the weight of the slab, the form weight, and the construction live load. The construction live load is removed immediately following the placement of the slab and the load carried by the shore drops to 1.100D. Once the slab gains sufficient strength, the shores can be removed.

Slab #	Load Carried by Slab			Load Carried by Shore
	Begin	Change	End	
1	0.000	1.000	1.000	
				0.000



As shown in the above figure, during the process of removing the shores, the slab is now activated and begins to support its own self-weight. The weight that was previously supported by the shores, minus their own self-weight, is now transferred to any connected slabs. In this case, there is only one slab to transfer the loads to. Obviously, since the shores have been removed, there is not any load being supported by the shores. In the figure below, it is shown that when the reshores are added, there is no additional weight transferred to the slab. The reshores are snuggly fit to the bottom of the slab as only to minimize deflection of the slab. Therefore, the only weight carried by the shore is its own self-weight.

Slab #	Load Carried by Slab			Load Carried by Shore
	Begin	Change	End	
1	1.000	0.000	1.000	
				0.050



This analysis continues as higher floors are constructed. A repetitive pattern will be seen to develop as the analysis progresses to the higher floors. A complete traditional analysis for a structure can be found in Appendix 8.1.

A disadvantage of the traditional approach is that the forms and shores must be completely removed from a given floor before the reshores are placed. Minimizing sets of forms and achieving rapid reuse is important since the formwork costs often represent 40 to 60% of the structure cost. The construction schedule is often controlled by the time required for the slab to become self-supporting before the forms can be stripped.

3.2.3 Analysis with New Method

For the new method with drophead shores, the same assumptions are applied and the loads are transferred through the temporary structure with the same principles. The difference is that the Drophead-beam-panel method has slightly different steps and applied load stages. Stripping the beams and panels while the shores stay in place, the stresses in the slab are initially limited to those induced by the short span between the shores. While work proceeds on form erection and reinforcement placement for the floor above, the previously cast slab can gain additional strength before activation. At the appropriate time, the shore heads are dropped, transferring load to the slab while the shores remain in position providing a safety net for the activated floor as shown in Figure 3.3.

The floor activation usually takes place after a higher floor has been formed and sometimes reinforcement placed but before the placement of that concrete slab. Following slab activation, the shore heads are re-extended and snugged for distribution of loads in subsequent construction steps. Because of the different components, setting up and tearing down of this formwork system varies. This does not necessarily change the analysis; it only changes the order in which certain loads are transferred throughout the formwork system and interconnected slabs.

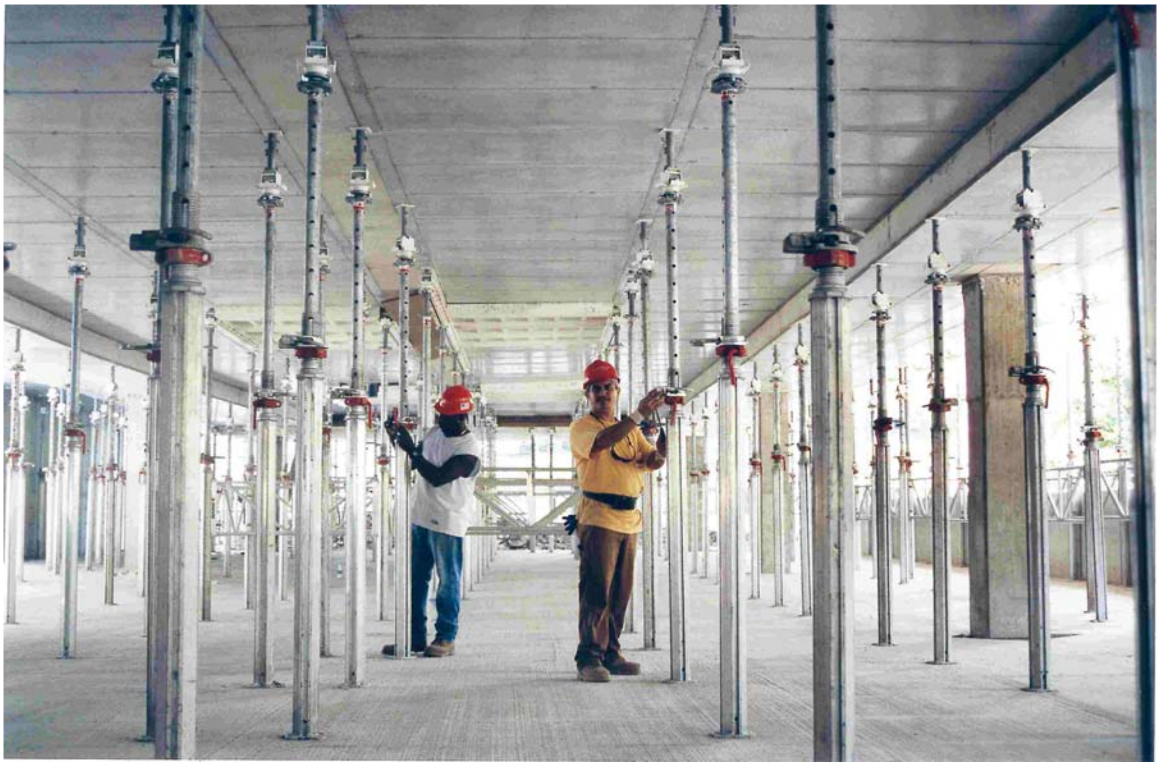
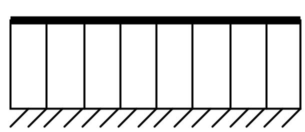


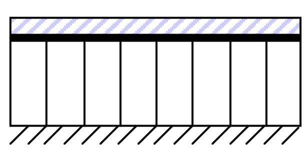
Figure 3.3 – Activation of floor slab (courtesy of Meva)

Below is a compilation of the steps involved in this new method.

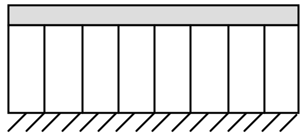
1. Setting up the formwork system



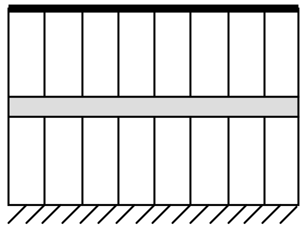
2. Casting the floor slab



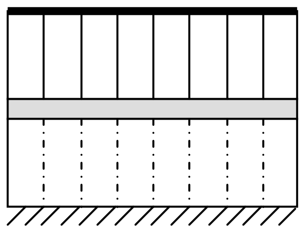
3. Removing the aluminum beams and panels



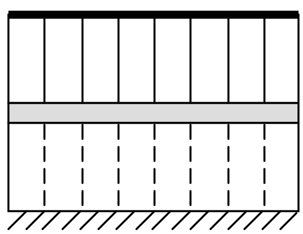
4. Setting up the formwork system on the next floor



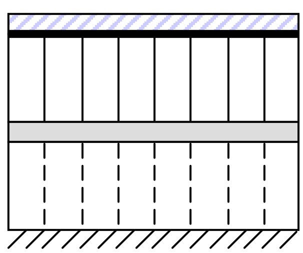
5. Activating the first slab (lowering shores)



6. Snug up the shores to act as reshores



7. Casting the next floor slab

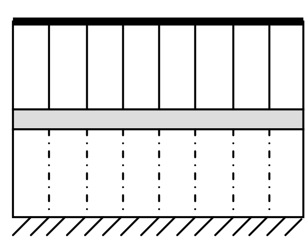


The same set of assumptions is used in this shoring analysis. This analysis is also case specific to having one set of shores and the number of sets of reshores selected. In this case, the shoring and reshoring are the same material. Shores become reshores after a slab has been activated. An example analysis for this method for a structure can also be found in Appendix 8.1.

3.2.4 Partial Activation

Removing a set of shores completely all at the same time will result in the full load from above being carried in the slab being “activated;” however, this does not always occur. Sometimes the shores are released only a few at a time, or just one at a time. In this process, an individual shore may be released fully, and resnugged, but, when a second adjacent shore is released, load again accumulates in the first shore. Thus, all shores supporting a slab do not end up fully released unless the release cycle is repeated several times. As a result, the slab may not be fully activated. An example of a slab being partially activated at 80% of the initial shore load is shown below. In this particular example, the loads that are being applied are the placed reinforcement at 0.100D, forming panels and shore at 0.100D, the weight of the slab at 1.000D, and the shores below that are released to become reshores at 0.050D. Notice that the shores continue to carry some load after the slab is activated at 0.80(0.100D + 0.100D + 1.000D).

Slab #	Load Carried by Slab			Load Carried by Shore
	Begin	Change	End	
2	0.000	0.000	0.000	
				0.200
1	0.000	0.960	0.960	
				0.290



Analyses were performed considering different percentages of “activation”. The different cases that were examined are shown in Table 3.1. It was found that Cases 2 and 3 seemed to apply the most load to the interconnected slabs, but the most critical was Case 2. Therefore, Figures 3.4, 3.5, and 3.6 are representations of Case 2. Each slab under consideration is referred to as slab “i”.

Table 3.1 Activation percentage analysis cases

Case	Description	Load at top slab
1	When shores, forms, and reinforcement are being erected and placed	C_D = 0.100D (reinforcement) C_{FML} = 0.050D (forms) C_{FML} = 0.050D (shores) C_P = 0.200D (forming live load)
2	When the slab is being placed including construction live load of 0.500D	C_D = 1.000D (slab) C_{FML} = 0.050D (forms) C_{FML} = 0.050D (shores) C_P = 0.500D (placing live load)
3	When a slab is being activated	C_D = 0.100D (reinforcement) C_{FML} = 0.050D (forms) C_{FML} = 0.050D (shores)
4	When the lowest level of shores is completely removed	N/A - No load on top slab

Since there were several interconnected slabs, the loads supported by every slab were determined. Figure 3.4 illustrates results from one of the critical cases in which a slab is being placed and all three sets of shores/reshores are in place. In Figure 3.4, floor “i” is always 3 levels below the level being placed and is the lowest slab supporting shores. Considering the step when floor 10 is being placed, Figure 3.4 shows the loads that are being carried by floor 7 depending upon the percent that the slab was activated initially. The load applied to the lowest interconnected slab ranges from 1.45D at level 2 under 100% activation to 2.1D at level 3 under 40% activation.

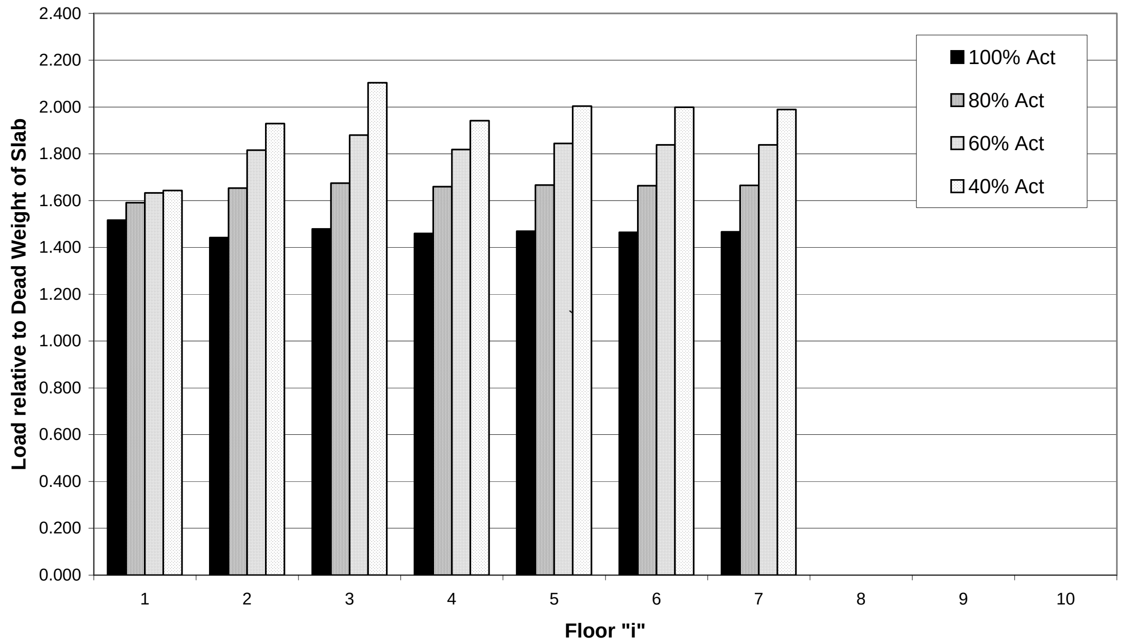


Figure 3.4 Loads carried by floor "i" when floor "i + 3" is placed

Clearly, the more that a slab is activated, the less load that is applied to the lower most interconnected slab. For example, if the slab was initially activated 100%, then the load being carried by floor 7 is 1.467D. If the slab was initially activated 40%, then the load now carried by floor 7 is 1.990D.

Figure 3.5 shows the load in slab "i" when slab "i + 2" is being placed. For example, when slab 10 is placed, the load in slab 8 is 1.566D if the initial slab activation is 100% and 1.570D if the initial activation is 40%.

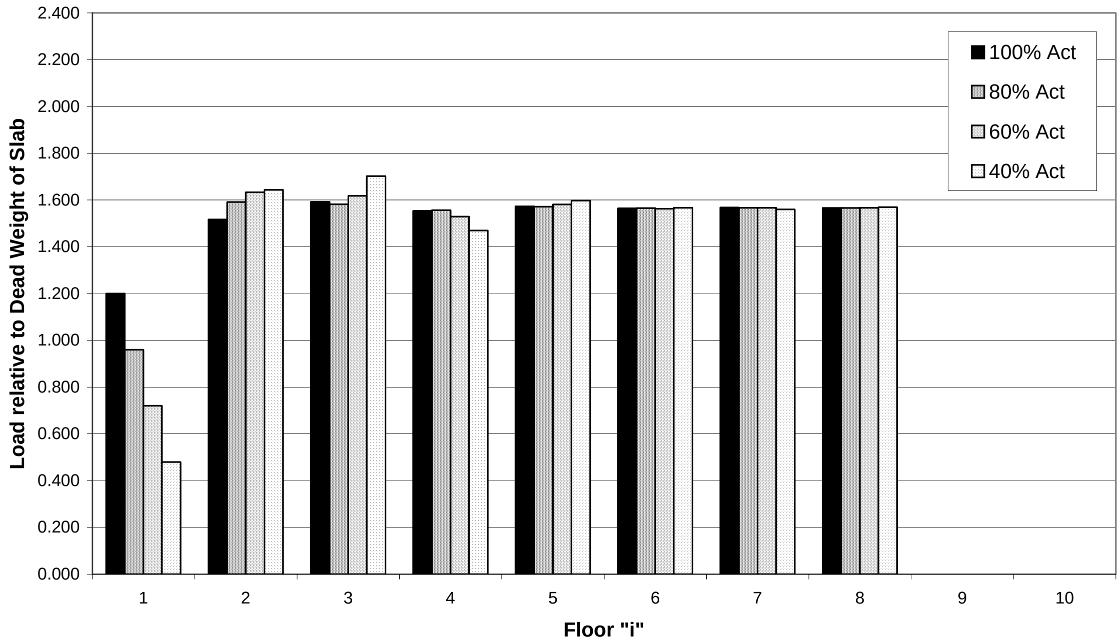


Figure 3.5 Loads carried by floor “i” when floor “i + 2” is placed

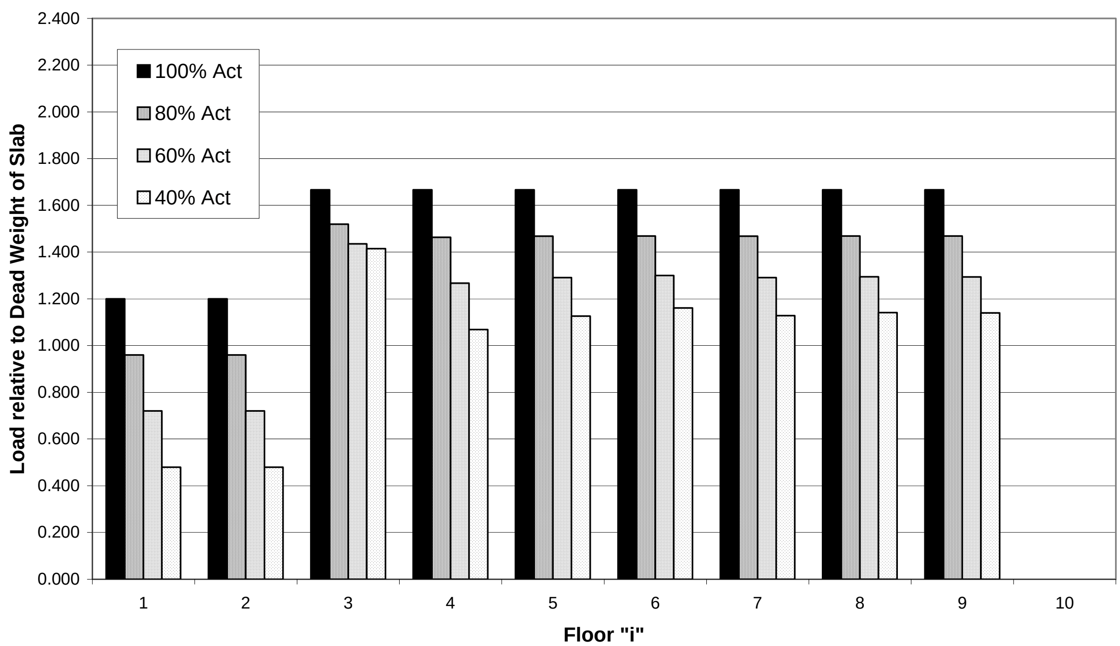


Figure 3.6 Loads carried by floor “i” when floor “i + 1” is placed

No matter what percentage of activation initially occurs when the shores are released, the floor 2 loads below the floor being cast eventually carries a load that is approximately 1.6D.

Figure 3.6 shows the load in slab "i" when slab "i + 1" is being placed. The lower the percentage of activation that takes place, the less the load that will be carried by the uppermost previously cast slab. Using the same example of when slab 10 is placed, the load in slab 9 is 1.667D if the initial slab activation is 100% and 1.140D if the initial activation is 40%.

Table 3.2 confirms that although the way the loads are distributed varies, the total amount of load that is distributed throughout the concrete floors remains the same regardless of the initial activation percentage as would be expected.

Table 3.2 Load carried by lower floors when floor 10 is placed

Floor	Activation	
	100%	40%
9	1.667	1.140
8	1.566	1.570
7	1.467	1.990
Total	4.700	4.700

Figures 3.4, 3.5, and 3.6 each indicate a peak load carried by the 3rd floor when the floor 1, 2, or 3 levels above is being placed. This peak may temporarily control the stripping schedule since more time would be needed to attain adequate strength.

4. STRENGTH ANALYSIS AND RESULTS

4.1 Strength Criteria

Approaches for evaluating early strength have varied in practice. For several editions, ACI SP-4[7] has contained analysis examples based on assuming the structure floor strength at an early age compared to the 28-day strength is directly proportional to the proportion of concrete compressive strength developed. Unfortunately, the examples do not result in workable solutions and methods for accelerating strength gain are suggested. ACI 318[8] assumes that the strength evaluation procedures therein are not a function of age and are applicable over a wide range of concrete strengths. CIRIA Report 136[4] presents an excellent summary of many of the issues involved plus discussions of the methods available for improved evaluation of in-place strength, such as maturity, penetration, pullout, and break-off tests.

The actual rate of concrete compressive strength gain varies with the concrete temperature and characteristics of the mixture proportions and ingredients. Figure 4.1 presents an analysis of components of structure strength gain versus time based on an assumed concrete compressive strength gain relationship. The relationship assumed for the concrete compressive strength gain is based on the following equation form:

$$f'_{ct} = [t / (a + bt)]f'_{c} \qquad (4.1)$$

where f'_{c} = compressive strength at 28 days, psi;

f'_{ct} = compressive strength at t, psi;

t = concrete age, days;

a,b = coefficients based on boundary conditions

For the case of f’ct / f’c = 1.0 at 28 days and f’ct / f’c = 0.7 at 7 days, the equation becomes:

$$f'_{ct} = [t / (4 + 0.857t)]f'_{c} \qquad (4.2)$$

This assumption is for example purposes only. Other concrete may gain strength at different rates or behave differently depending on concrete temperature, cement type, etc.

The strengths were estimated using the methods of ACI 318[8]. As noted previously by others and summarized in CIRIA Report 136[4], the flexural strength gain is very rapid. For reinforcing amounts seen in flat plates, typically < 1%, the flexural strength gain even at very early ages (Figure 4.1) is higher than for bond, beam shear, and punching shear. Bond and shear behavior at early ages has been more controversial and still needs more attention for the benefit of construction. Some recommendations in the CIRIA Report[4] suggest that the strength gain for bond and shear versus the 28-day strength should be conservatively assumed proportional to the concrete compressive strength gain or that standard shear strength prediction methods should not apply below a concrete strength of 10 MPa (1450 psi). Other investigations suggest the early strength is predictable (Neville[3]) and that early loading to normal limitations from the predictable strengths has no detrimental effect on performance of the permanent structure (Shah[1], Clark and Johnston[2]). In ACI 318[8], both bond and shear are expressed as a function of the square root of concrete compressive strength as illustrated in Figure 4.1. Some practitioners have suggested a compromise with other root relationships such as $(f_{c\text{-time}} / f_{c\text{-}28})^{0.7}$ or applied a lower limit of 8 MPa (1150 psi), for the minimum compressive strength, based on experience.

The analysis presented in this report will assume the provisions of ACI 318 are applicable for low strength early age concrete; however, an advisable lower limit should be considered at compressive strengths of 8 to 10 N/mm^2 (1150 to 1450 psi).

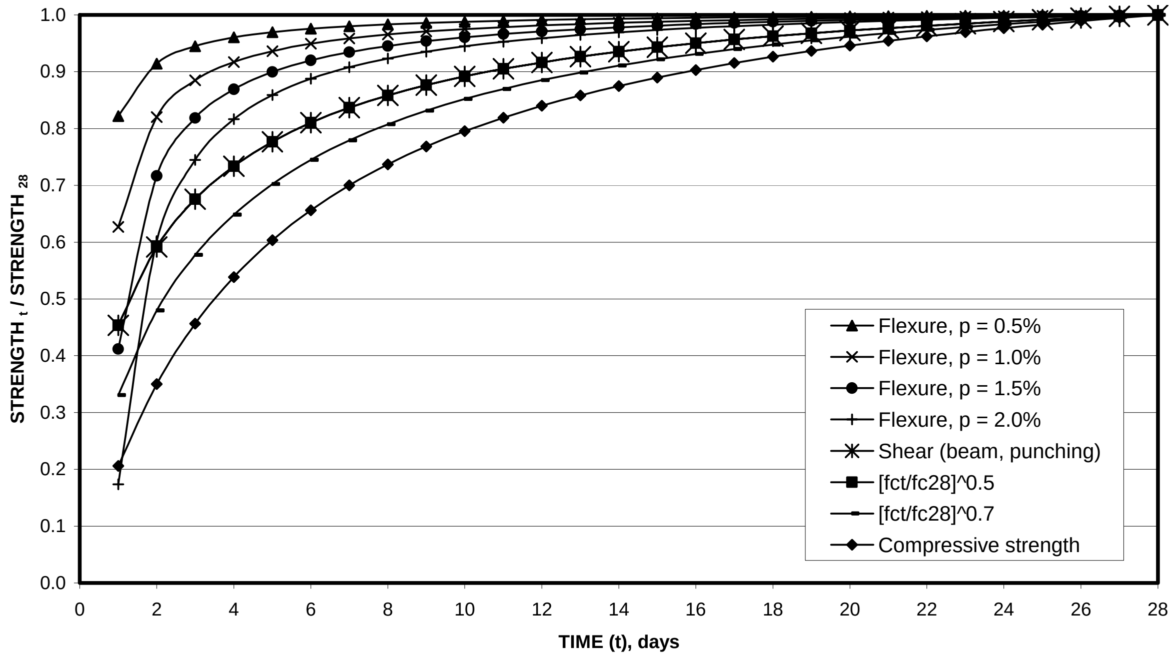


Figure 4.1 - Alternatives for development of structural strength based on ACI 318 and an example assumed concrete compressive strength gain

4.2 Slab Analysis Approaches

When the reinforcement for concrete floor slabs is designed, it is designed for the loads that will be imposed during the service life of the building. It is not designed specifically for loads encountered during construction. For flat plates, which do not have any beams between columns, the slab is designed by subdividing it into a series of column strips and middle strips (Figure 4.2) with widths which depend on the spacing of the columns or supports. ACI 318-05[8] defines a column strip as a design strip with a width on each side of a column centerline equal to 0.25 l_1 or 0.25 l_2 whichever is less. The terms l_1 and l_2 are defined as the length of span in the same direction or transverse to the direction that moments are being determined, measured center to center of supports. A middle strip is the design strip bounded by two column strips. For design purposes, these column strips are considered to be continuous beams that span across the columns of the structure. The amount of reinforcement required in each direction is a function of the moments at the critical sections. There is only positive moment across most of the span and negative moment near and over the columns. The middle strips have positive moment over much of their span, but negative moment in part of the support region provided by the column strips. Because of this, there will be differing amounts of reinforcement throughout the slab. Positive moment requires bottom-reinforcing steel in that direction and negative moment requires top-reinforcing steel in that direction. Thus, the reinforcement pattern can vary in the different locations. Figures 4.3b and 4.3c show an example of the possible variation in reinforcement.

The shores supporting the slab also make up their own temporary “bays” as shown in Figure 4.3a. However, the shores have a smaller spacing and the reinforcement pattern in the “bay” between shores is thus often highly variable in its suitability for the moments and shears generated by early construction loads when the form panels are stripped and the slab must span between the supporting shores. Due to the short spans involved, the column strips between shores can be referred to as “shore strips”. Figure 4.2 helps to illustrate the column or “shore” strip.

It is not practical to investigate every possible reinforcement level and arrangement. Any shore bay that has reinforcement beyond the needs for the short span would not pose a problem structurally. Therefore, this study will investigate the case of code-required minimum levels of reinforcement and the case of plain concrete in which there is no reinforcement.

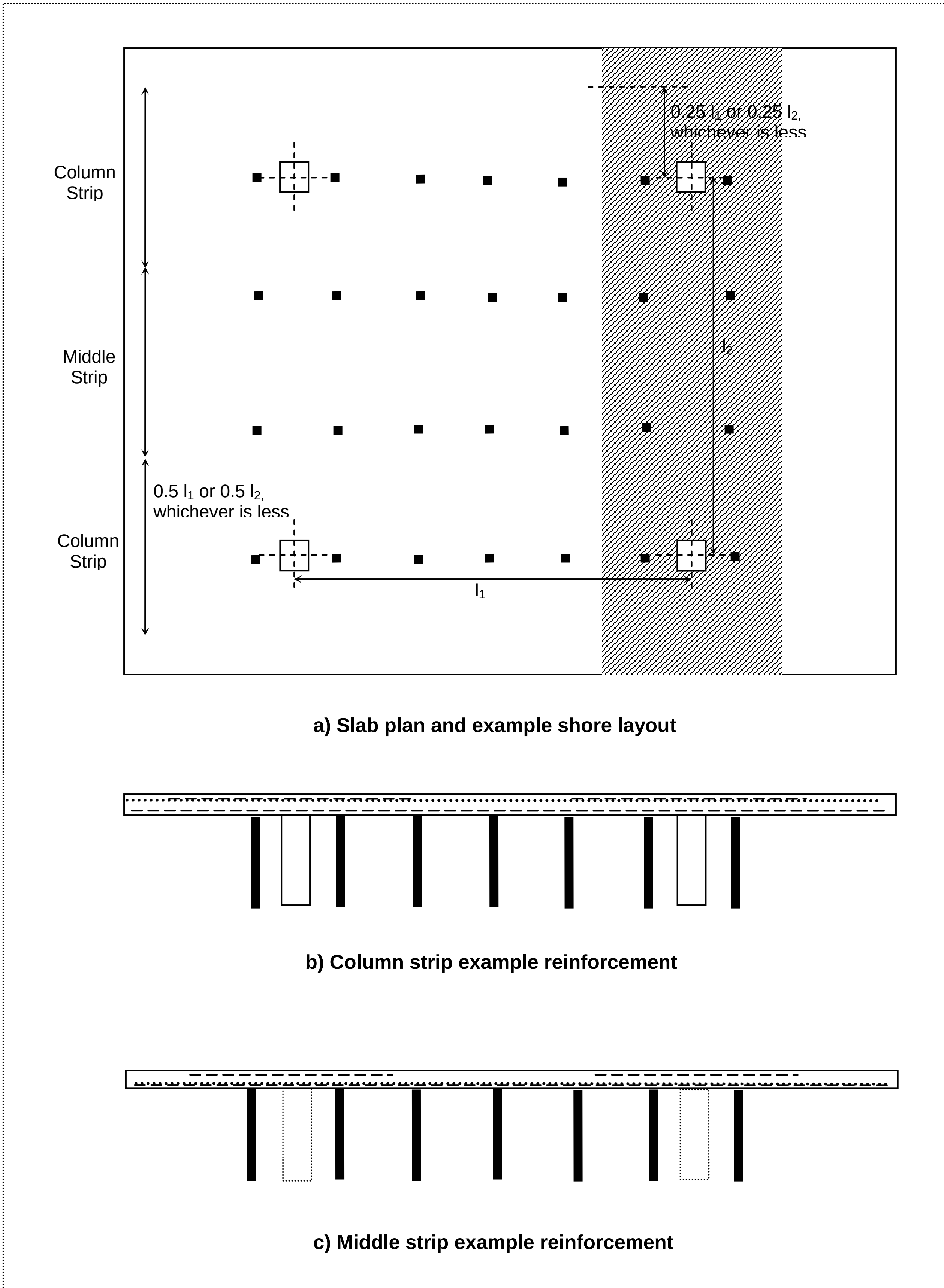


Figure 4.2 Typical floor plan and shore layout

Below is a list of the areas of analysis that were examined.

- Punching Shear at the Shore (without shear reinforcement)
    - Reinforced for Flexure
    - Plain Concrete (temperature steel only on compression face)
- Flexural Strength in spans between shores
    - Reinforced on tension face
    - Plain Concrete (temperature steel only on compression face)
    - Crack Development
- Beam Shear (without shear reinforcement)

The objective of the analysis is to determine the total uniform distributed load that can be carried by the early age slab in spanning between shores after stripping the beams and panels but before activation of the slab. A sample of results for the primary controlling cases in presented in this chapter. The complete set of results is provided in Appendix 8.2.

4.3 Punching Shear

There are many different levels of load sensitivity for each structural member. Concrete slabs are one of the most sensitive structural members during the construction process due to shear dependency. One of the more likely failure modes could be punching shear. Punching shear is load transferred through a slab to an interior column, or shore, and is sometimes referred to as two way shear. This can occur in a column bearing on a footing, or in our case, a flat slab supported by a column or shore. Figure 4.3 illustrates a slab supported by a column or a shore.

The basic equation for shear from Chapter 11 of ACI 318-05[8] is:

$$V_u < \phi V_n \tag{4.3}$$

where V_n is the nominal shear resistance of the slab and V_u is the factored shear force due to the loads. The strength reduction factor for shear, ϕ, is equal to 0.85. Shear forces exerted on the slab can be resisted by the punching shear strength of the concrete, V_c, and any shear

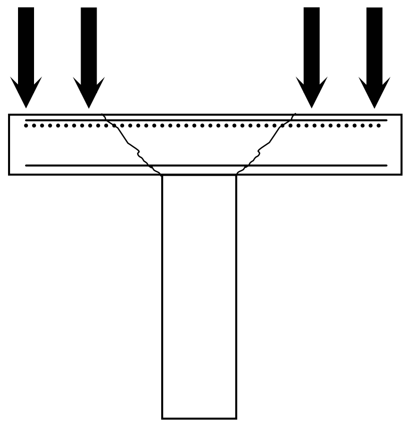


Figure 4.3 Slab supported by column or shore

reinforcement, V_s, that is in the slab. For this analysis, it is assumed that there is no shear reinforcement within the slab section; thus, V_s is zero and equation 4.3 reduces to:

$$V_u < \phi V_c \tag{4.4}$$

where V_c is the shear strength obtained from the concrete section alone. According to Shah[2], for beams that don't have any shear reinforcement, this equation is also conservative for concrete at early ages. Two situations will be considered as shown in Figure 4.4: the situation where there is flexural reinforcement at the top, and the situation where there is no top flexural reinforcement.

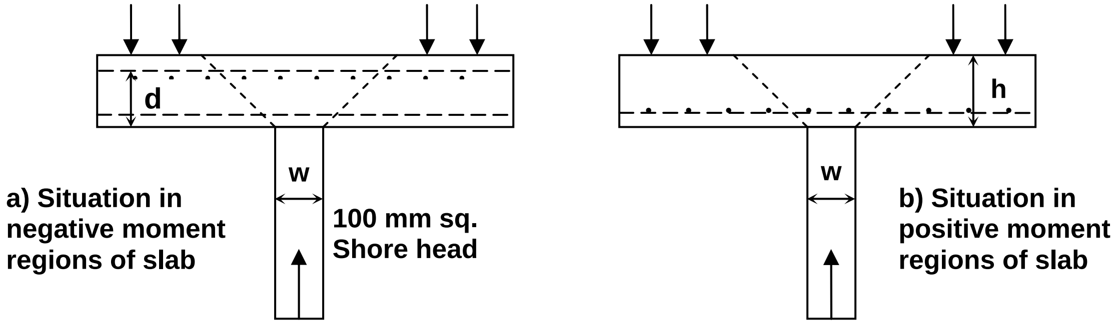


Figure 4.4 Example variations in shore punching shear resistance conditions

4.3.1 Reinforced Slab Punching Shear

The provisions in ACI 318-05 Chapter 11 for nominal two-way shear resistance of a slab for a square reaction area can be reduced to the following equation:

$$\phi V_n = \phi 4(f'^{\,0.5}_c)[4(w + d)d] \quad (4.5)$$

Figure 4.5 defines the dimensional variables in this equation.

The variables for punching shear strength are the size of the square shore head, “w”, the concrete strength, f'_c, and the depth of the slab, d. A range of slab depths and concrete strengths were used to evaluate the effects these variables have on the nominal punching shear load capacity. In all examples, the length of the “shore strip” was center-to-center spacing of the shores with the beams used to support the Meva shoring panels. The longest Meva beam with a length of 2.7 m (Beam 270) was used for the examples in this chapter. Results for other lengths are presented in Appendix 8.2. The size of the shore bearing plate is 10 cm x 10 cm (3.9 in. x 3.9 in.) and w = 10 cm (3.9 in.). Thus, the typical layout is as shown in Figure 4.6

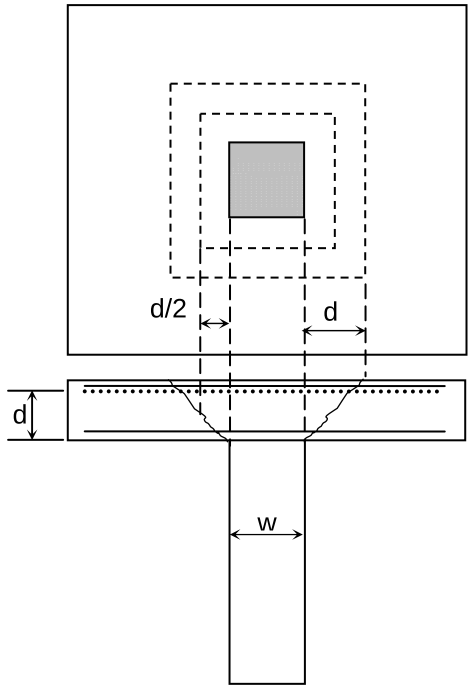


Figure 4.5 Punching shear diagram

For example, consider a slab thickness of 7.5 in. This slab is to contain at least a minimum amount of reinforcement on the top face with a clear cover of approximately 0.75 in. The depth, d, will be the slab thickness less the clear cover and the diameter of the reinforcement bar. The diameter of the bar could range from 0.375 in to about 1 in. Therefore, it will be assumed for analysis purposes to have a depth “d” of 6.00 in. Substituting these values into equation 4.5 and for a concrete compressive strength of 1500 psi, the punching shear capacity of the slab section is

$$\phi V_n = 0.85[4(1500^{0.5})[4(3.9 + 6)(6)]] = 31{,}404 \text{ lbs}$$

Figure 4.6 illustrates the tributary area in which the factored load is assumed applied. The critical section for punching shear is at the perimeter located at d/2 from the face of the shore. Load within the perimeter can normally be neglected. However, a more conservative approach is assumed by taking the load on the entire tributary area as the load that must be supported.

$$\text{Tributary Area} = (1.7\text{ m})(2.8\text{ m}) = 4.76\text{ m}^2 = 51.24\text{ ft}^2$$

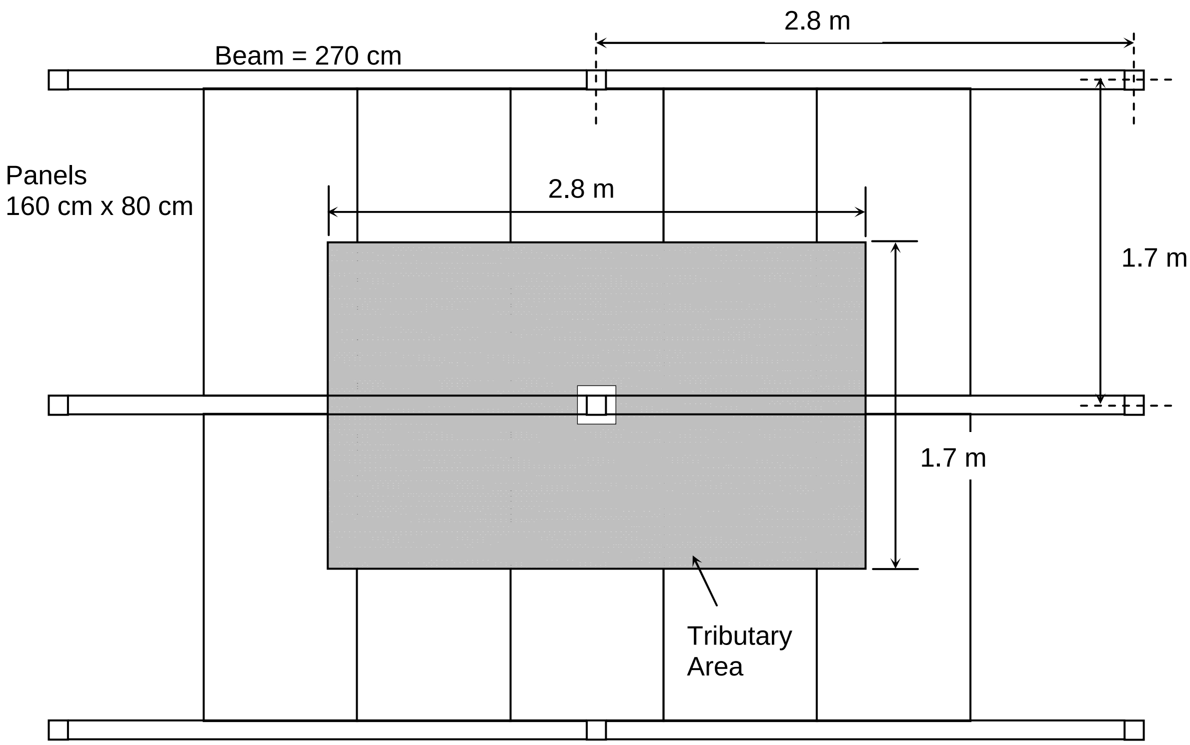


Figure 4.6 Plan view of shoring system

Using this area, the ultimate punching shear capacity distributed load of 613 lbs/ft^2 is determined by dividing the punching shear strength by the tributary area.

Figure 4.8 presents the ultimate load capacity of slabs based on punching shear for a range of concrete strengths developed and various slab depths, d. The punching shear capacity has been divided by the tributary area to determine the ultimate load capacity.

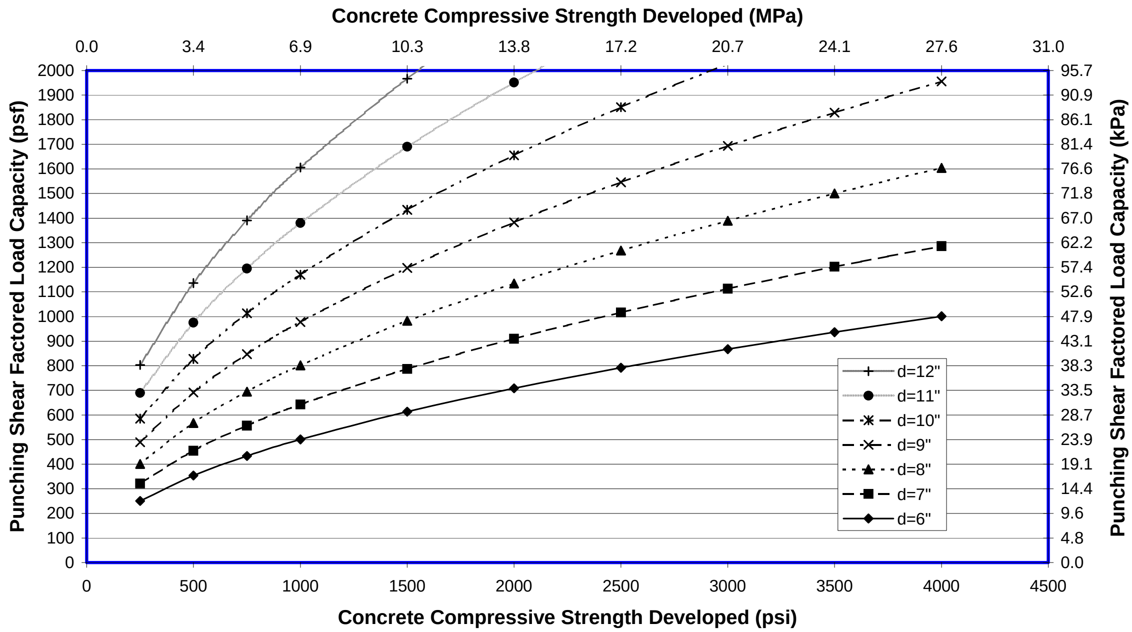


Figure 4.7 - Reinforced slab shore punching shear design strength load capacity (100x100 mm shore, 2.70 m primary beam, 1.6 m panel)

4.3.2 Plain Concrete Slab Punching Shear

There is also the possibility that the slab reinforcement is located only on the face of the slab in contact with the shore since the shore may be located in an area that is a positive moment region for the overall floor design. Therefore, it is necessary for such cases, to consider the punching shear strength of the plain concrete section. ACI 318-05 Chapter 22 provides methods for calculating the punching shear strength of plain concrete. For the case of the square shore of dimension w, and a slab thickness of h as shown in Figure 4.5b, the ultimate shear capacity is:

$$\phi V_n = \phi[4/3 + 8/(3\beta)](f'_c{}^{0.5})[4(w + h)h] \qquad (4.6)$$

but not greater than $\phi 2.66(f'_c{}^{0.5})[4(w + h)h]$ (4.7) β is defined as the ratio of the long side to the short side of the concentrated load or reaction area. In this example, the shore head is square, thus, β is equal to 1.0. The full thickness of the slab, h, is used in determining the ultimate shear resistance. After comparing these two equations, it becomes clear that equation 4.7 controls when β = 1.0. Figure 4.9 was developed from equation 4.7.

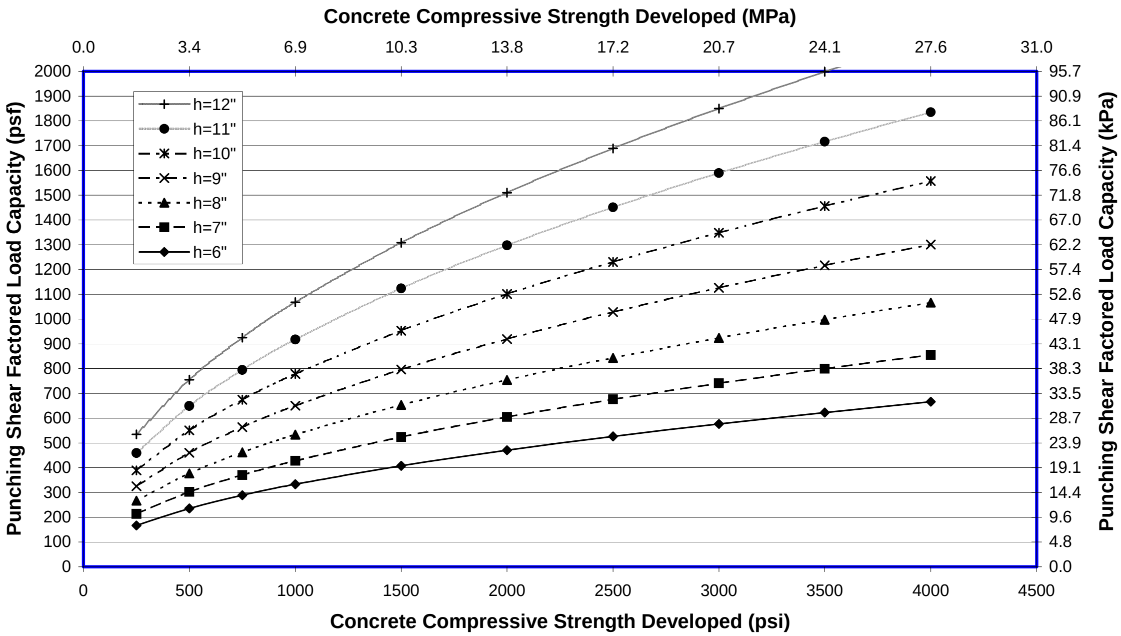


Figure 4.8 - Plain slab shore punching shear design strength load capacity
(100x100 mm shore, 2.70 m primary beam, 1.6 m panel)

Using Figure 4.9, the punching shear capacity can be determined for plain concrete slab situations. To help understanding of how this figure was established, an example follows. Using the same size slab and compressive strength as in the previous example, with h = 7.5 inches, the punching shear capacity was calculated as:

$$\phi V_n = 0.85[4/3 + 8/[(3)(1.0)]](1500^{0.5})[4(3.9 + 7.5)7.5] = 45{,}181 \text{ lbs}$$

$$\phi V_n = 0.85(2.66)(1500^{0.5})[4(3.9 + 7.5)7.5] = 30{,}046 \text{ lbs}$$

It is clear that the second equation controls. Therefore, using the same tributary area as in the previous example, the ultimate punching shear capacity is approximately 586 lbs/ft^2.

4.4 Flexural Capacity

Flexure of the early age slab spanning between shores after stripping must also be considered. A series of analyses considering one way and two way action along with moments generated in both directions of the slab section have been developed. The equations used for the nominal strength of a concrete section and the factored loads can be found in ACI 318-05, Chapter 9.

$$\phi M_n = \phi[A_s f_y(d - a/2)] \qquad (4.8)$$

where $a = A_s f_y / 0.85 f'_c b$

$$M_u = wl^2/8 \quad \text{simple span (positive moment)} \qquad (4.9)$$

$$M_u = wl^2/10 \quad \text{continuous span (negative moment)} \qquad (4.10)$$

4.4.1 Reinforced Flexural Capacity

The analysis presents a variety of concrete slab depths in comparison to the maximum capacity of that concrete section. Again, the pattern of reinforcement becomes important as indicated in Figure 4.3. Generally, ACI 318-05[8] requires a minimal amount of bottom reinforcement in middle and column strips to be continuous. For simplicity, the flexural analyses are performed assuming that the steel present corresponds to the minimum required and, thus, the resulting charts can be used safely with a variety of actual designs which will have equal or greater reinforcement. Top steel is not always present. Thus, dual charts are provided considering both continuous spans and simple spans, with the latter assuming flexural cracking may develop over the shores as indicated in Figure 4.9.

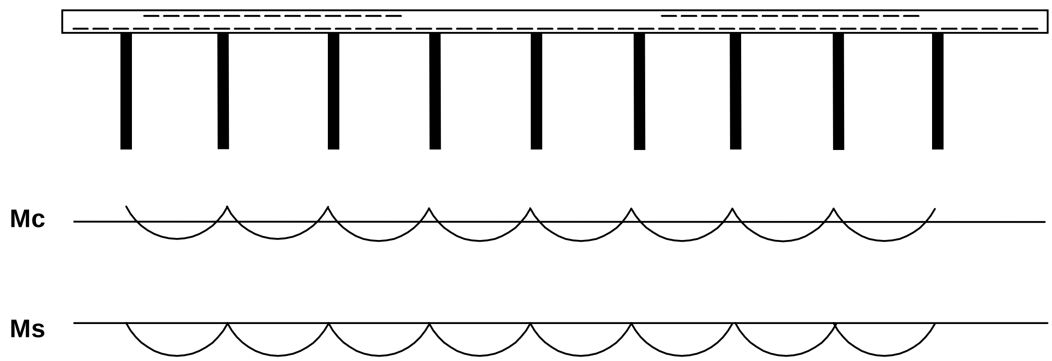


Figure 4.9 Moment resistance with and without negative moment capacity

ACI 318-05[8] gives the following equations for minimum area of steel.

$$A_{s,min} = 3[(f'_c{}^{0.5})/(f_y)]bd \tag{4.11}$$

but not less than

$$200 b_w d/f_y \tag{4.12}$$

Examining these two equations, equation 4.12 controls for concrete strengths less than 4500 psi. A 28 day strength of 4000 psi or higher is typically used in modern construction, so equation 4.12 provides the conservative lower bound for reinforcement that should be available.

Because flexure occurs in both directions of the two-way slab simultaneously, an adjustment is made to the one-way moment capacities calculated. ACI 318-05[8] states that for factored moments in column strips, the column strip shall be proportioned to resist portions in percent of interior negative moment as shown in Table 4.1

The variable α_1 is the ratio of flexural stiffness of a beam section to the flexural stiffness of a width of slab. Since there are no beams, α_1 is zero. Therefore, the column or "shore" strip shall be proportioned to resist 75% of the factored moment.

Table 4.1 Factored moments in column strips

l_2/l_1	0.5	1.0	2.0
$\alpha_1 l_2/l_1 = 0$	75%	75%	75%
$\alpha_1 l_2/l_1 > 1.0$	90%	75%	45%

This will increase load capacities for a two-way slab system. In Figures 4.11 and 4.12 respectively, one-way and two-way flexural capacity in the direction parallel to the beam are illustrated. These capacities were calculated based on the assumption of a reinforced, simply supported beam.

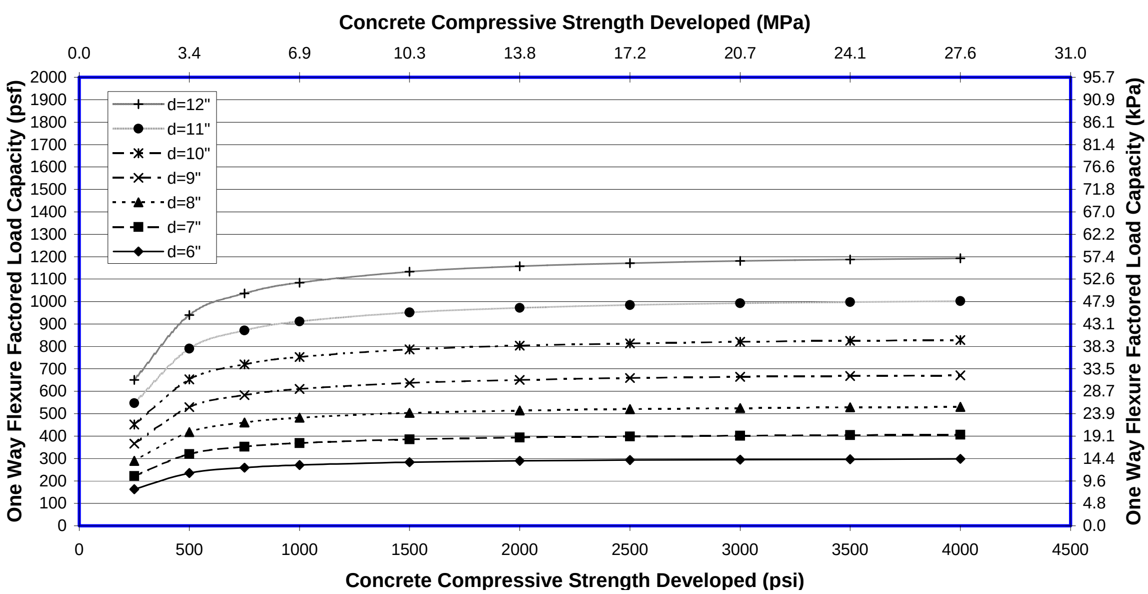


Figure 4.10 - Reinforced slab shore one way flexural factored load capacity (100x100 mm shore, 2.70 m primary beam, 1.6 m panel)

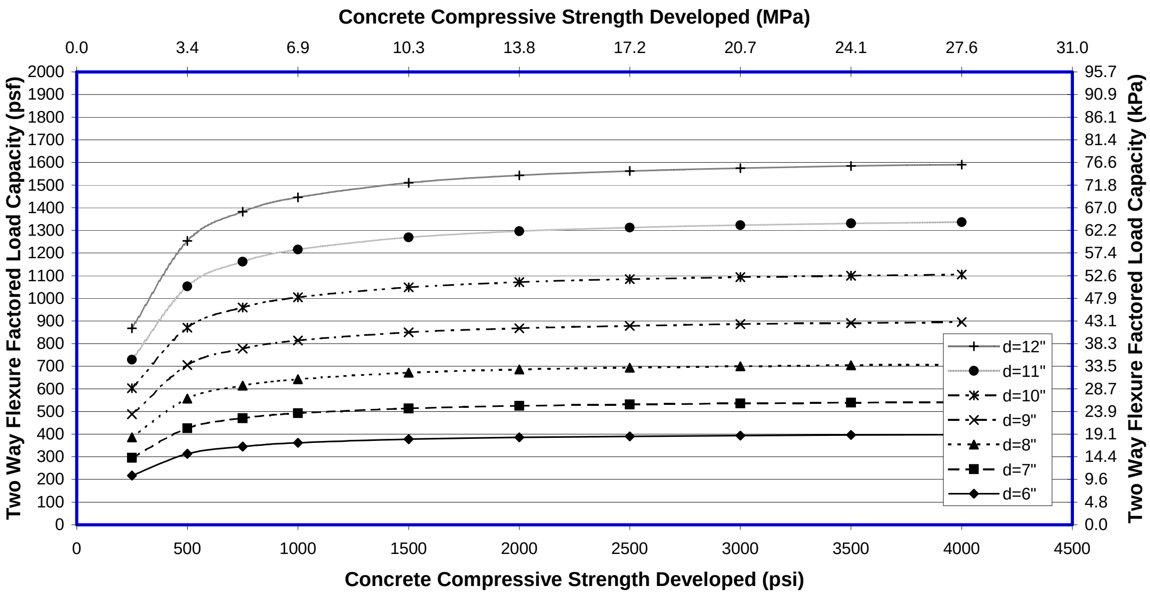


Figure 4.11 - Reinforced slab shore two way flexural factored load capacity (100x100 mm shore, 2.70 m primary beam, 1.6 m panel)

Both of these figures illustrate the moments calculated in the direction parallel to the column or "shore" strip. Moments were also calculated in the direction perpendicular to the column strips. Depending on the span between supports, the parallel or perpendicular direction may control. Figure 4.13 illustrates the shore strip that was used in the flexural analysis.

Using equation 4.11, the minimum area of steel for the column strip can be calculated.

$$A_{s,min} = 200*33.4*6/60,000 = 0.669 \text{ in}^2$$

Using this information, "a" can now be calculated as:

$$a = (0.669)(60,000) / (0.85)(1500)(33.4) = 0.9411 \text{ in.}$$

This allows the calculation of the flexural capacity as:

$$\phi M_n = 0.9[0.669(60,000)(6-0.9411/2)] / 12 = 16,646 \text{ ft-lbs}$$

Using equation 4.9, the calculated moment capacity can be converted into a uniform line load. Then dividing by the total tributary width of 5.57 ft (1.7 m) between shores, a maximum factored distributed load, q_u, that can be supported can be calculated as:

$$q_u = (16{,}646/0.75)(8) / (9.1)^2(5.57) = 378 \text{ lbs/ft}^2$$

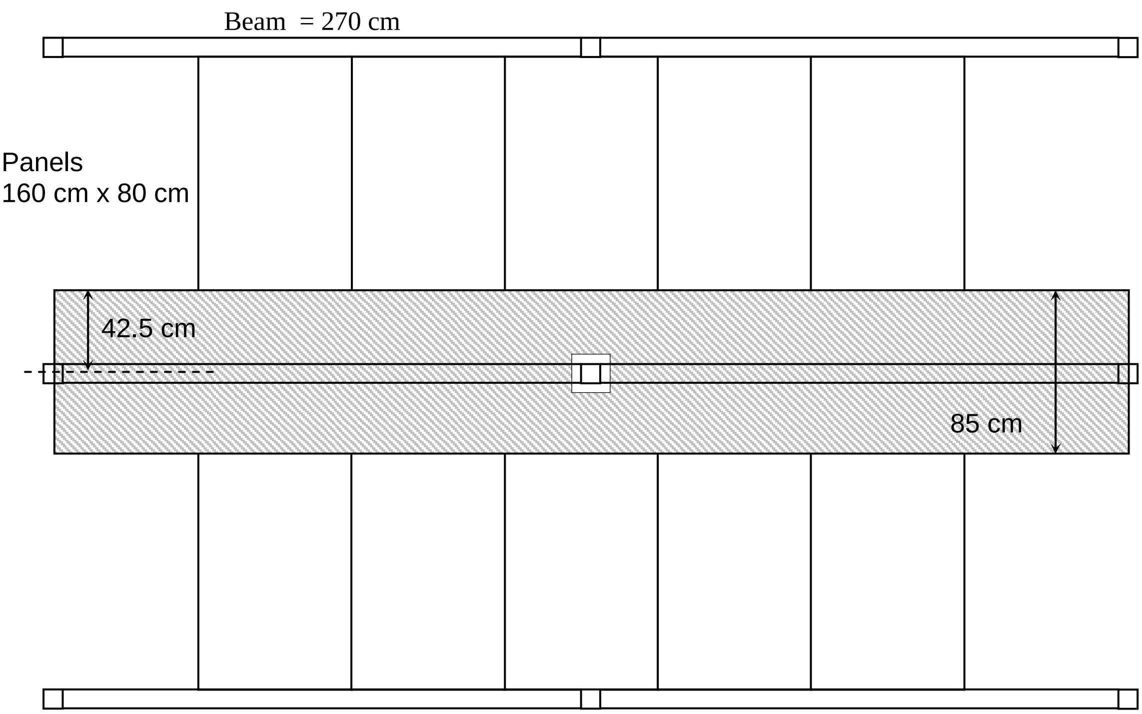


Figure 4.12 Plan view of shore strip

4.4.2 Plain Concrete Flexural Capacity

The possibility of the "shore strip" being in an area of the permanent structure where the slab has negative moment only and thus no bottom steel must be investigated. For this situation, the construction strength is based on the slab cracking strength. The analysis performed assumes that any reinforcement in the concrete section does not contribute to the flexural strength. The only reinforcement in this area would be for temperature and shrinkage and load resistance is provided only by the strength of the plain concrete. ACI 318[8] Chapter 22 limits the flexural capacity of plain concrete as shown in equation 4.13.

$$\phi M_n = \phi 5(f'_c{}^{0.5})S \qquad (4.13)$$

where "S" is the elastic section modulus of the cross section. The section modulus of a rectangular cross section is defined as $bh^2/6$. The width of the section is the width of the "shore" strip. The height is the full thickness of the slab. For the "shore" strip,

$$S = (33.4)(7.5)^2 / 6 = 313 \text{ in}^3$$

The cracking strength can now be calculated using equation 4.13 as:

$$\phi M_n = 0.9[5(1500^{0.5})(313)] / 12 = 4{,}545 \text{ ft-lbs}$$

Using equation 4.10, the calculated moment capacity can be converted into a uniform line load. Then dividing by the total tributary width of 5.57 ft (1.7 m) between shores, a maximum factored distributed load, q_u, that can be supported can be calculated as:

$$q_u = (4545)(10) / (9.1)^2(5.57) = 99 \text{ lbs/ft}^2 \qquad \text{for one way action}$$

$$q_u = (4545/0.75)(10) / (9.1)^2(5.57) = 132 \text{ lbs/ft}^2 \qquad \text{for two way action}$$

Figures 4.13 and 4.14 provide the ultimate flexural load capacity for cracking strength for one-way and two-way flexure action respectively in a parallel direction to the beams.

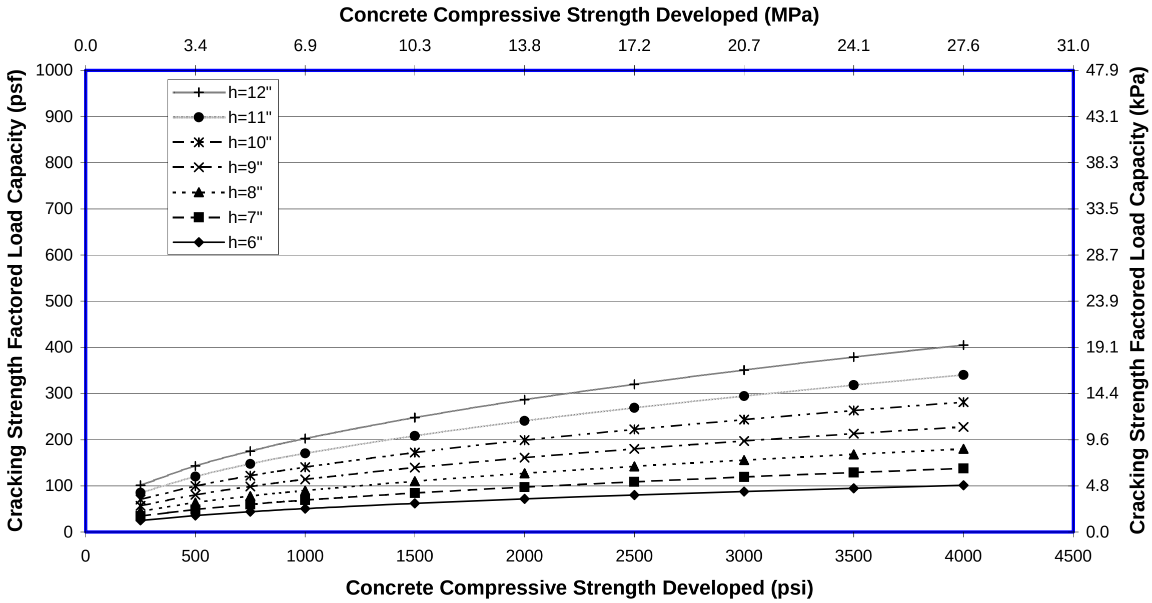


Figure 4.13 - Plain slab shore one way cracking strength factored load capacity (100x100 mm shore, 2.70 m primary beam, 1.6 m panel)

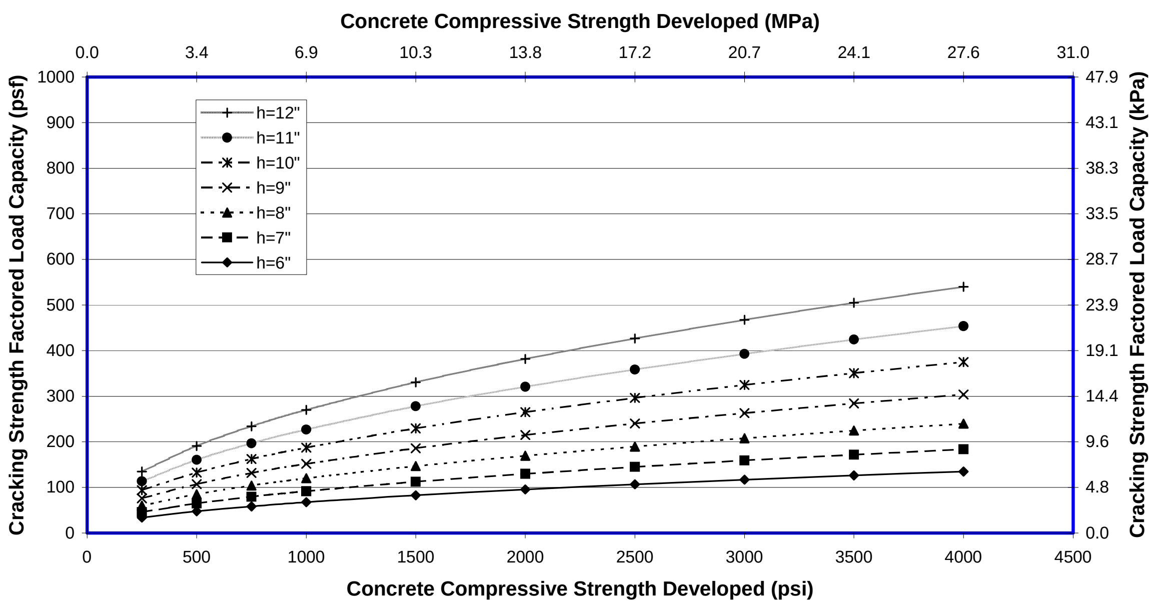


Figure 4.14 - Plain slab shore two way cracking strength factored load capacity (100x100 mm shore, 2.70 m primary beam, 1.6 m panel)

4.4.3 Cracking Moment in Reinforced Section

In some structures there may be a concern to avoid visible cracking in the concrete slab at the time of stripping the form panels. Therefore, an analysis of slab crack development due to early loading is required. The concern is not preventing collapse since the structure is reinforced, rather it is predicting the working loads that would cause cracks to occur. Analysis is based upon the same assumptions as for the plain concrete slab cracking strength except that the cracking stress is based on a value estimating the actual tensile strength of concrete, rather than a reduced value. Neville[13] has suggested an estimated of tensile strength as $9.5(f'^{0.5}_c)$. Thus, the flexural cracking strength will be determined as:

$$\phi M_{cr} = \phi 9.5(f'^{0.5}_c)S \tag{4.14}$$

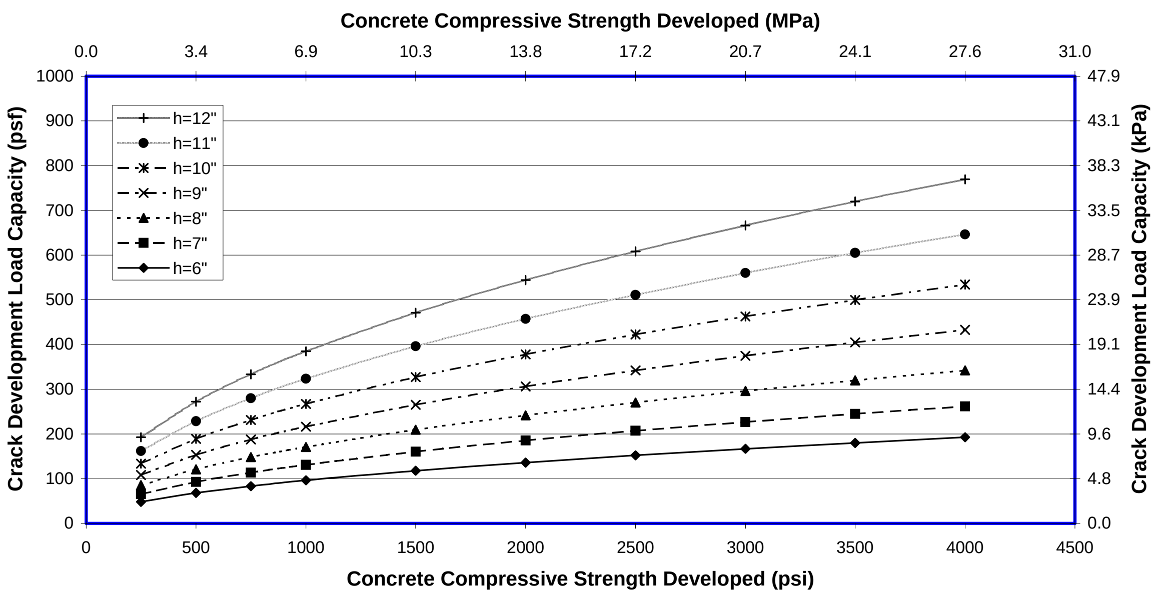


Figure 4.15 - Plain slab shore one way crack development load capacity (100x100 mm shore, 2.70 m primary beam, 1.6 m panel)

Figures 4.15 and 4.16 show the maximum loads at which cracks will occur in a concrete section. The values in these figures include the weight of the slab itself. The self weight of the slab would have to be deducted in order to determine the maximum superimposed load. For example, consider the case in which a two-way slab had a thickness of 7.5 in. and the slab had developed a concrete compressive strength of 1500 psi. From Figure 4.16, the load capacity at which cracks would begin to occur is about 250 psf. Subtracting the self-weight of the slab (90.6 psf) would indicate crack development would occur at an applied load of about 190 psf.

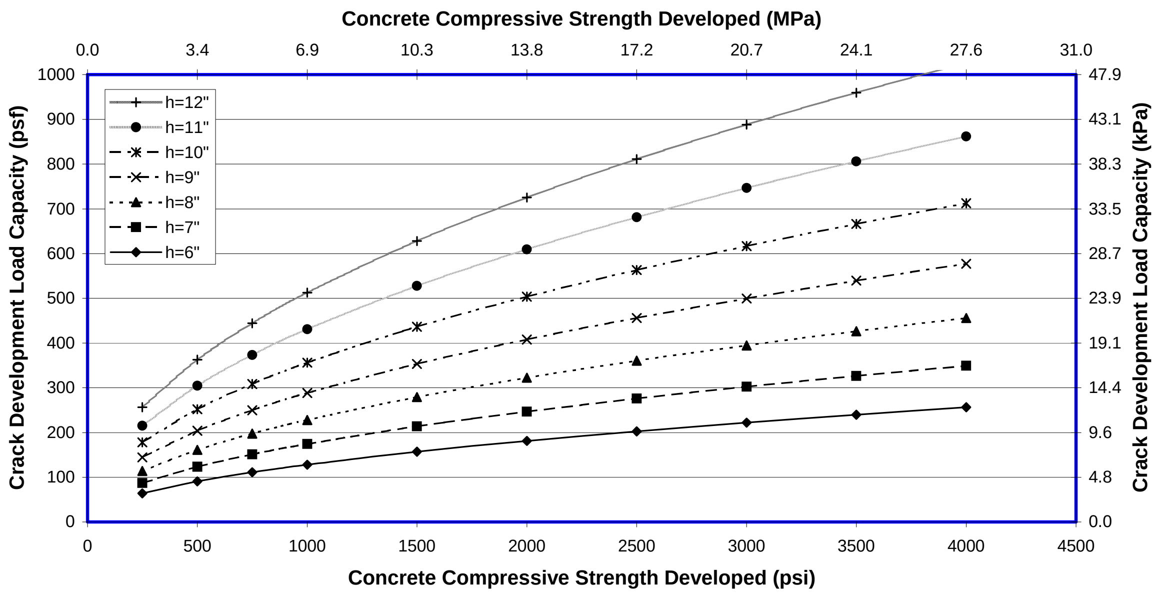


Figure 4.16 - Plain slab shore two way crack development load capacity
(100x100 mm shore, 2.70 m primary beam, 1.6 m panel)

4.5 Beam Shear

ACI 318[8] defines beam shear capacity as ϕV_n, which is dependent upon the compressive strength of the concrete section along with any added strength from shear reinforcement. With the assumption that there is no shear reinforcement within the slab, ϕV_n becomes ϕV_c. ACI 318-05[8] defines the parameters for beam shear capacity as:

$$\phi V_c = \phi 2(f'^{0.5}_c)bd \qquad (4.15)$$

The variables "b" and "d" are the width of the "shore" strip and the depth of the slab, respectively. Using equation 4.15, the beam shear capacity can be calculated as:

$$\phi V_c = 0.85(2)(1500)^{0.5}(33.4)(6) = 13{,}220 \text{ lbs}$$

The maximum shear force to be resisted is taken as the shear at the center of the support. A more precise required shear would be that at d/2 away from the support. However, the former is more conservative and, as shown in Figure 4.17, it is unnecessary to calculate shear at any other point along a beam. Figure 4.18 provides the capacity based on shear strength for various slab depths and concrete compressive strengths.

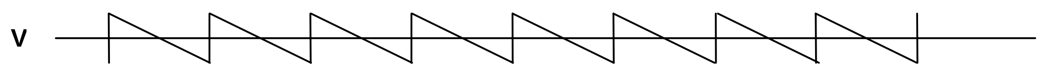


Figure 4.17 – Shear force diagram

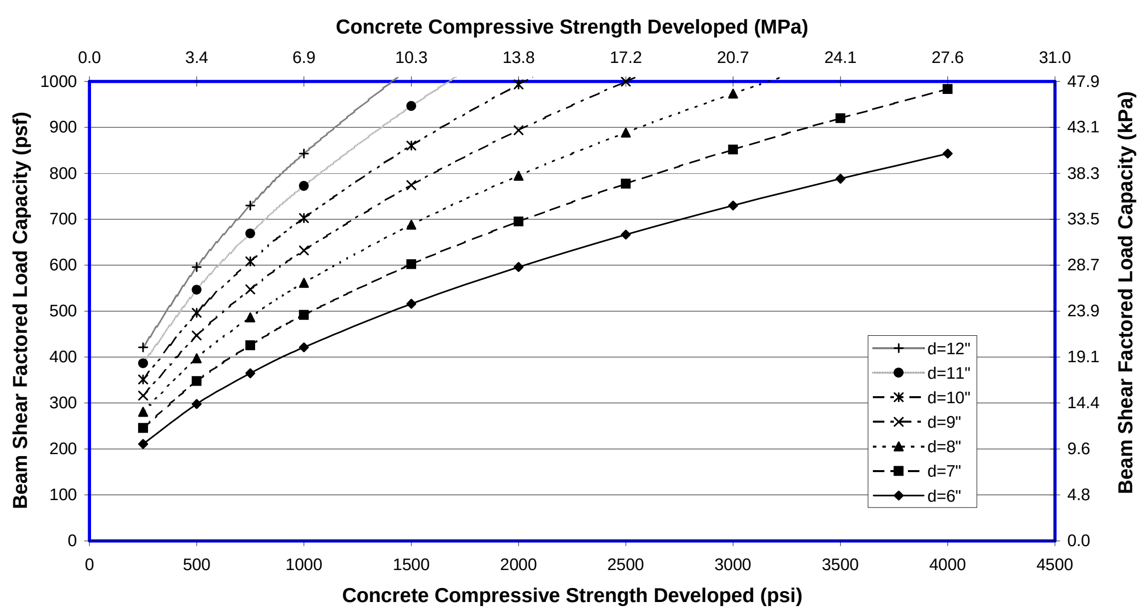


Figure 4.18 - Reinforced slab shore beam shear load capacity
(100x100 mm shore, 2.70 m primary beam, 1.6 m panel)

5. EXAMPLE USE OF ANALYSIS RESULTS

5.1 Introduction

The purpose of this chapter is to compare the results from the strength analysis to the actual factored loads that are required for a specific case. For the examples, the slabs will have a thickness, h, of 7.5" and have developed a concrete compressive strength of 1500 psi at the time of load application.

5.2 Construction Dead Load

Dead loads applied include the elements of the forming system and the dead weight of the slab. It can be reasonably assumed that the unit weight of the concrete is 145 pcf. This density of concrete includes normal reinforcement that may be in the slab. For the 7.5" slab, the dead load is:

$$D = (145 \text{ lbs/ft}^3)(7.5 \text{ in})(1 \text{ ft}/12 \text{ in}) = 90.6 \text{ lbs/ft}^2 \quad (5.1)$$

5.3 Construction Live Load

The construction live load consists of workers and equipment and is defined as C_P by SEI/ASCE 37[9]. ASCE 37[9] requires uniform load of 20 psf for a very light working live load. This is considered to be appropriate for areas of very light duty, which includes sparsely populated areas with personnel, hand tools, and very small amounts of construction materials. This assumption was used for live load when shores are being removed or formwork is being erected. For this example, the case is that in which the forms are being erected. During erection of forms, the number of personnel is relatively small and not concentrated and few tools are involved; thus the uniform load associated with very light duty is applicable. Very light duty corresponds to a personnel load, C_P, of 20 lb/ft^2.

5.4 Construction Material Load

The next area of concern is material load applied during the setup of the formwork, shores, beams, and panels. The material load could consist of a transport angle fully loaded with forming panels or a full rack of primary beams or secondary beams as shown in Figure 5.1. Only one can occupy a given area at a given time. Table 5.1 lists the different parts of a transport angle.

Table 5.1 Fully loaded transport angle weight

Item	Weight (kg)	No.	Total Weight (kg)	Total Weight (lbs)
Folding angle	17.0	2	34.0	75.0
Rigid angle	12.9	2	25.8	56.9
Swivel wheels	1.2	4	4.8	10.6
MD-panels	24.0	12	288.0	634.9
Total			352.6	777.35

The floor area occupied by a transport angle is approximately 16.5 ft^2. With a total weight of 777 lbs, this results in a distributed load of 47.2 lbs/ft^2. For the piling rack, two options must be considered. It could be full with 20 primary beams or 50 secondary beams, which weigh 23 kg and 9 kg respectively. The weight of the piling rack itself is 155 kg. Table 5.2 shows that the 20 primary beams would create a heavier load of 1356 lbs and therefore controls the calculation.

Figure 5.1 – Fully loaded transport angle and piling rack (courtesy of Meva)

Table 5.2 Fully loaded piling rack

Item	Weight (kg)	No.	Total Weight	
			kg	lbs
Piling Rack	155	1	155	341.7
Primary Beam	23	20	460	1014.1
Total			615	1355.8
Piling Rack	155	1	155	341.7
Secondary Beam	9	50	450	1014.1
Total			605	1333.8

The floor area occupied by the piling rack is approximately 21.5 ft^2. This results in a distributed load of about 63 lbs/ft^2. Comparing the full transport angle and the full piling rack, the piling rack loaded with 20 primary beams is the controlling material load.

5.5 Factored Load Combinations

The calculated loads can now be factored and combined in accordance with SEI/ASCE 37[9] as discussed in Chapter 2. Since the racks have a fixed configuration and a maximum loading arrangement, they can be considered to be a fixed material load, C_{FML}. In other cases in which the construction material may be stored more loosely, it may be more appropriate to consider it as variable material load.

Using these load factors, a required ultimate load is obtained for comparison to the capacities of punching shear, flexural strength, and beam shear. For the loads involved , Equation 2.1 reduces to:

$$U_C = 1.2D + 1.6C_P + 1.2C_{FML} \tag{5.2}$$

The personnel load and the material load of the rack cannot occupy the same area at the same time. Also, it must be understood that the 63 lbs/ft^2 is a construction load that is applied only over the area occupied by the piling rack. In order for this load to be used in the same equation as the dead and personnel load, it must be converted to a load that is applied over the full tributary area. A conservative approximation would be to apply the personnel load, which has the higher load factor, over the entire area and to reduce the material load by:

$C_P(1.6/1.2) = 20(1.6/1.2) = 26.7$ psf

This results in an adjusted variable material load of approximately (63 – 26.7) or 36.3 lbs/ft^2. Using the area of the piling rack, an adjusted concentrated load for the fully loaded piling rack is given by (36.3 lbs/ft^2)(21.5 ft^2) = 781 lbs. Dividing this concentrated load by the tributary area for the shore of 51.25 ft^2, gives a new material load of 15.3 lbs/ft^2. From equation 5.2 the required factored load capacity needed is:

$$U_C = 1.2(90.6) + 1.6(20) + 1.2(15.3) = 159 \text{ lbs/ft}^2$$

5.6 Comparison to Strength Available

The required load capacity of 159 lbs/ft^2 can now be compared to the appropriate figures of strength available in Chapter 4. Consider the case mentioned earlier in which the slab thickness is 7.5” and the amount of compressive concrete strength developed is 1500 psi. The comparison will consider the needed punching shear, flexure, and beam shear capacities.

Figure 4.8 shows that for this particular size slab, there is an ultimate punching shear capacity of approximately 600 lbs/ft^2 when the slab has reached a compressive strength of 1500 psi, which clearly exceeds the anticipated load. Figure 4.8 assumes there is minimum reinforcing steel. In a case in which there is not any reinforcement, Figure 4.9 can be used to estimate the ultimate shear load capacity. A 7.5 in. thick slab will have a punching shear capacity of about 575 lbs/ft^2 when the slab has reached a compressive strength of 1500 psi, so punching shear is not a controlling factor.

Figure 4.18 is used to evaluate beam shear. Assuming the same slab depth and concrete compressive strength, the ultimate beam shear capacity is approximately 530 lbs/ft^2. This is enough capacity for the loads being applied.

Referring to Figure 4.12, the flexural capacity in the parallel direction to the beam for two-way action can be estimated. For the same case as before, the ultimate load capacity based on flexure is approximately 380 lbs/ft^2. In the direction perpendicular to the beams, the ultimate load capacity is about 625 lbs/ft^2. Again, this is only for the case in which there is a minimum amount of steel reinforcement.

In the case in which there would be plain concrete, Figure 4.14 is used to estimate the flexural capacity. Similarly, the ultimate flexural capacities are approximately 140 lbs/ft^2 and 240 lbs/ft^2 respectively in the parallel and perpendicular directions of the beams. For these particular situations, there is insufficient flexural capacity for moments in the parallel direction of the beams. In this situation, the contractor would have to wait until the slab had gained more compressive strength before panels could be removed from the shoring system.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

The following observations and conclusions are based on the research conducted in this study.

- Analysis spreadsheets can be and were developed for the shoring and reshoring of multistory concrete structures including the cases of the traditional method and the case of the drophead shore method. The spreadsheets include options to include load factors and varying levels of slab activation.
- Advances in the mechanization of forming and shoring systems for multi-story concrete building offer the possibility of reducing formwork materials required on site and providing additional safety.
- Earlier stripping of form panels can be accomplished without activation of the floor since the shores initially remain in place; however, more careful evaluation of the floor is needed due to its low strength and the variety of reinforcing patterns with respect to the supporting shores.
- Charts for simplified evaluation of strength available have been developed based on a set of strength assumptions. Similar charts may be developed for other limiting parameters.
- It is conservative to consider the case with the longest span between the shores. If this case proves to be structurally sufficient, then it would also be sufficient for situations with shorter spans.

- Due to the short spans, deflection of the early age slab between shores at the most recent level cast is very small. Deflections are sometimes a concern as floors are activated to span between columns and at lower levels where shores accumulate greater loads.
- An advantage of the drophead shore system is that the time required for slab activation is brief and there is less time available for creep to occur before the shores are snugged.
- Another advantage is the presence of the shores as a safety net upon activation of the floor. In traditional methods, the striking of shores and forms from under a slab results in the complete removal of support. Although the drophead shores are lowered to provide a gap and assure complete activation, the shores remain in place and could support the slab if problems developed during activation.

6.2 Recommendations for Future Research

Considering that formwork is 40 to 60% of the cost of concrete construction, there is an unfortunately limited level of fundamental research emphasis on the construction phase. Several recommendations for future research were identified during this study as follows:

- Measurement of shore forces before and after slab activation considering various shore release patterns.
- Structural strength development on slabs during the very early stages of concrete strength development.
- Slab capacity for the case of slab loading in which the shores below and above a slab are not aligned or are at different spacings.

- Evaluation of the applicability of ACI 318 methods of structural strength determination for early age very low strength concrete.

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8. APPENDIX

8.1 Shoring Analysis

This appendix provides the complete analysis for the shoring process. The cases presented here include the traditional method and the new method using the process described by Meva. The traditional analysis has one set of shores and two sets of reshores. The Meva analysis has three sets of reshores as suggested by Meva.

The shoring analysis spreadsheet is set up in two sections. The first section is the inputs. The loads for each particular step are imputed along with the load factors associated with them. The example shows loads that are inputted into the cells as a ratio of the dead weight of the slab. Actual loads could be input as an alternative. The second section is the output showing loads that are carried by each particular slab as well as the sets of shores/reshores. All of the cells within the output section include equations that relate back to the inputs.

An example of a key cell formula is included in step 2-C, activating floor #2 by removing the shores supporting that slab. This formula consists of taking the load that was in the shores and subtracting the self-weight of the shores and transferring that particular load from the shores into the slab. Equation 8.1 helps to illustrate step 2-C.

$$1.100 - 0.100 = 1.000 \tag{8.1}$$

Therefore, the activated slab is now responsible for supporting its own weight. Another example of a key cell formula is when a set of reshores is completely removed from the structure as in step 3-D. Here, the reshores from the first floor are completely removed and the load that was carried by those reshores must now be distributed among the connected slabs. There are only two slabs that are connected by shores or reshores. This formula consists of taking the load that was in the reshores supporting the first slab and subtracting the self-weight of the reshores and divide that load among the two connecting slabs, slab 1 and slab 2. Equation 8.2 helps to illustrate step 3-C.

$$(0.100 - 0.050) / 2 = 0.025 \tag{8.2}$$

This same principle is applied when load is added to the top of a structure. The load must be distributed between the floors that are connected below. For finding a load that is carried by a shore or reshore, simply take the load from the nearest shore above and add the weight of the slab and any loads that may be applied to that floor, then subtract out the load that the slab is carrying itself. This will give you the load that is carried by the shore/reshore. For example, in step 2-A the load being carried by the shores supporting slab #2 is 1.7 times the dead weight of the slab. Adding the weight of slab #1, the self-weight of the shores below slab #1, and subtracting the weight being carried by slab #1 gives you the load being carried by the reshores below slab #1. Equation 8.3 helps to illustrate this calculation.

$$(1.7000 + 1.000 + 0.050) - 1.00 = 1.750 \tag{8.3}$$

Although the traditional and Meva shoring processes are somewhat different, these same cell formulas are used in both of the analysis that are presented here. This is due to the fact that the same assumptions are made concerning the shoring process, regardless of how the steps are divided.

SHORING AND RESHORING ANALYSIS - SP4 METHOD

INPUT

Form sets	1	Reshore sets	2

MEMBER	L.F.	LOAD	
slab weight	1.0	1.000	1.000
Form/shore weight	1.0	0.1	0.100
Constr. Live Load	1.0	0.6	0.600
Reshore weight	1.0	0.05	0.050

OUTPUT

Step #	Slab #	Load carried by slab beginning	during	end	Load carried by shore end	Floor	
1 - A	1	0.000	0.000	0.000		1	*Place 1st floor slab*
					1.700		
1 - B	1	0.000	0.000	0.000		1	*Remove live load*
					1.100		
1 - C	1	0.000	1.000	1.000		1	*Remove shore*
					0.000		
1 - D	1	1.000	0.000	1.000		1	*Place 1st floor reshores*
					0.050		
2 - A	2	0.000	0.000	0.000		2	*Place 2nd floor slab*
					1.700		
	1	1.000	0.000	1.000		1	
					1.750		
2 - B	2	0.000	0.000	0.000		2	*Remove live load*
					1.100		
	1	1.000	0.000	1.000		1	
					1.150		
2 - C	2	0.000	1.000	1.000		2	*Remove shore*
					0.000		
	1	1.000	0.000	1.000		1	
					0.050		
2 - D	2	1.000	0.000	1.000		2	*Place 2nd floor reshores*
					0.050		
	1	1.000	0.000	1.000		1	
					0.100		

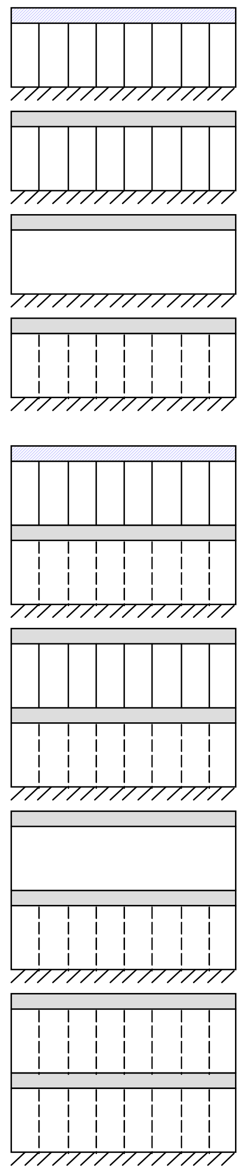


Figure 8.1 – Traditional method for shoring and reshoring analysis

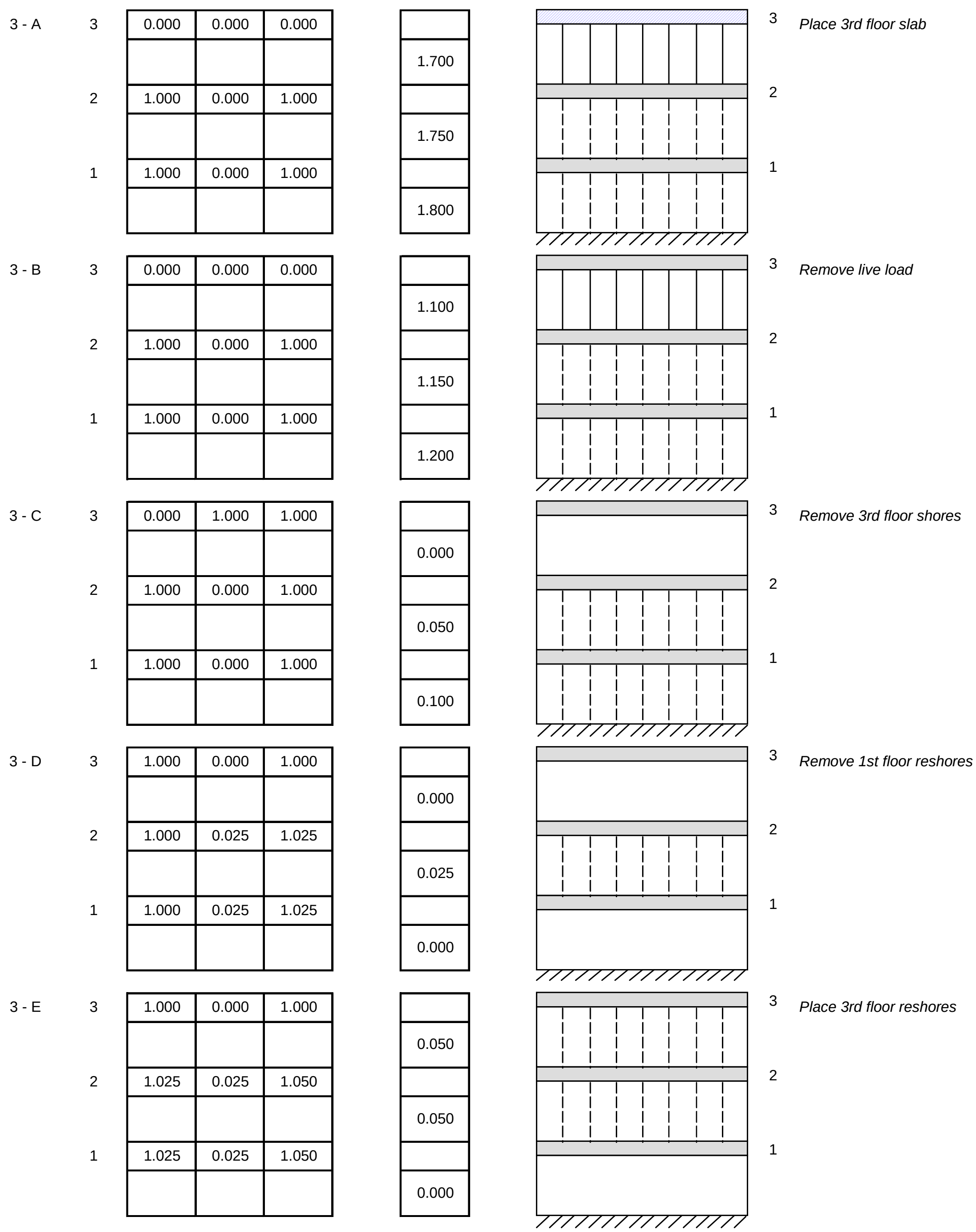
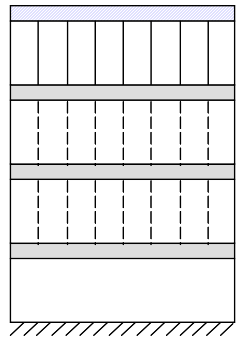


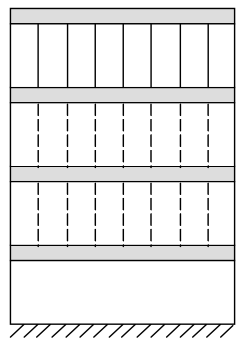
Figure 8.1 (cont.) – Traditional method for shoring and reshoring analysis

4 - A	4	0.000	0.000	0.000	
					1.700
	3	1.000	0.567	1.567	
					1.183
	2	1.050	0.567	1.617	
					0.617
	1	1.050	0.567	1.617	
					0.000



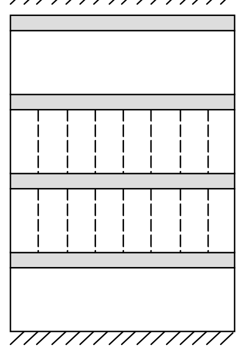
Place 4th floor slab

4 - B	4	0.000	0.000	0.000	
					1.100
	3	1.567	-0.200	1.367	
					0.783
	2	1.617	-0.200	1.417	
					0.417
	1	1.617	-0.200	1.417	
					0.000



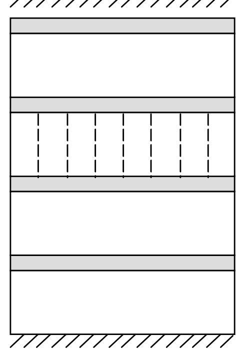
Remove live load

4 - C	4	0.000	1.000	1.000	
					0.000
	3	1.367	-0.367	1.000	
					0.050
	2	1.417	-0.367	1.050	
					0.050
	1	1.417	-0.367	1.050	
					0.000



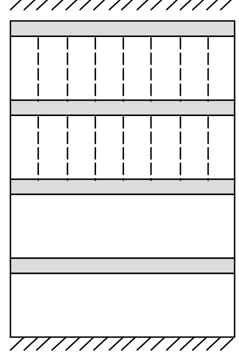
Remove 4th floor shores

4 - D	4	1.000	0.000	1.000	
					0.000
	3	1.000	0.000	1.000	
					0.050
	2	1.050	0.000	1.050	
					0.000
	1	1.050	-0.050	1.000	
					0.000



Remove 2nd floor reshores

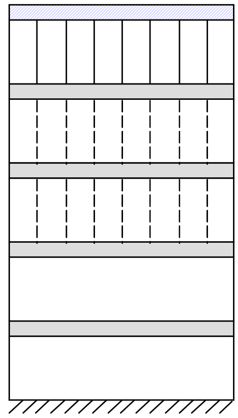
4 - E	4	1.000	0.000	1.000	
					0.050
	3	1.000	0.025	1.025	
					0.075
	2	1.050	0.025	1.075	
					0.000
	1	1.000	0.000	1.000	
					0.000



Place 4th floor reshores

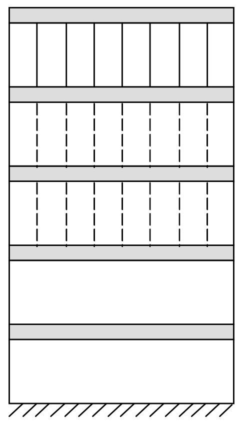
Figure 8.1 (cont.) – Traditional method for shoring and reshoring analysis

5 - A	5	0.000	0.000	0.000	
					1.700
	4	1.000	0.567	1.567	
					1.183
	3	1.025	0.567	1.592	
					0.642
	2	1.075	0.567	1.642	
					0.000
	1	1.000	0.000	1.000	
					0.000



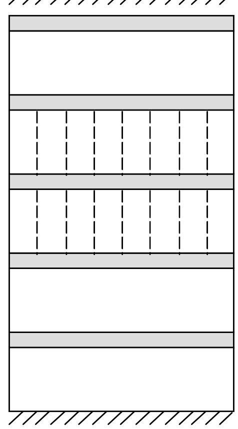
Place 5th floor slab

5 - B	5	0.000	0.000	0.000	
					1.100
	4	1.567	-0.200	1.367	
					0.783
	3	1.592	-0.200	1.392	
					0.442
	2	1.642	-0.200	1.442	
					0.000
	1	1.000	0.000	1.000	
					0.000



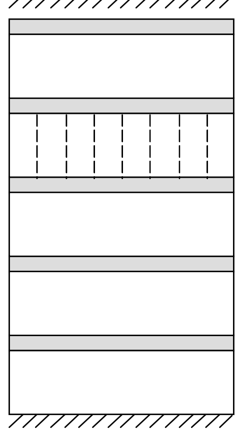
Remove live load

5 - C	5	0.000	1.000	1.000	
					0.000
	4	1.367	-0.367	1.000	
					0.050
	3	1.392	-0.367	1.025	
					0.075
	2	1.442	-0.367	1.075	
					0.000
	1	1.000	0.000	1.000	
					0.000



Remove 5th floor shores

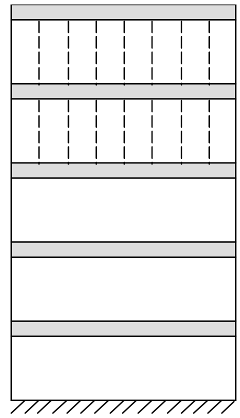
5 - D	5	1.000	0.000	1.000	
					0.000
	4	1.000	0.013	1.013	
					0.037
	3	1.025	0.013	1.038	
					0.000
	2	1.075	-0.075	1.000	
					0.000
	1	1.000	0.000	1.000	
					0.000



Remove 3rd floor reshores

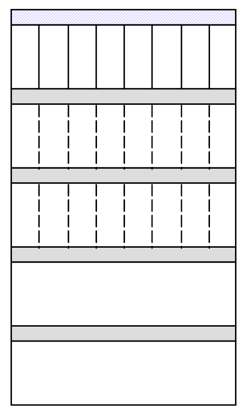
Figure 8.1 (cont.) – Traditional method for shoring and reshoring analysis

Stage	Floor				Shore
5 - E	5	1.000	0.000	1.000	
					0.050
	4	1.013	0.025	1.038	
					0.062
	3	1.038	0.025	1.063	
					0.000
	2	1.000	0.000	1.000	
					0.000
	1	1.000	0.000	1.000	
					0.000



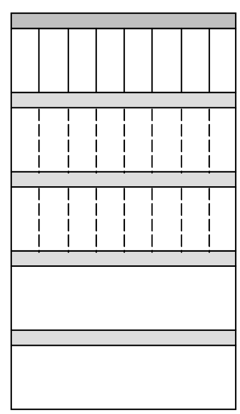
Place 5th floor reshores

Stage	Floor				Shore
6 - A	6	0.000	0.000	0.000	
					1.700
	5	1.000	0.567	1.567	
					1.183
	4	1.038	0.567	1.604	
					0.629
	3	1.063	0.567	1.629	
					0.000
	2	1.000	0.000	1.000	
					0.000



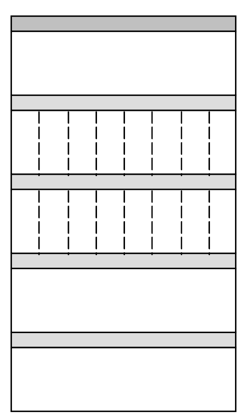
Place 6th floor slab

Stage	Floor				Shore
6 - B	6	0.000	0.000	0.000	
					1.100
	5	1.567	-0.200	1.367	
					0.783
	4	1.604	-0.200	1.404	
					0.429
	3	1.629	-0.200	1.429	
					0.000
	2	1.000	0.000	1.000	
					0.000



Remove live load

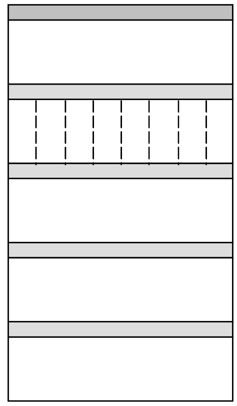
Stage	Floor				Shore
6 - C	6	0.000	1.000	1.000	
					0.000
	5	1.367	-0.367	1.000	
					0.050
	4	1.404	-0.367	1.038	
					0.062
	3	1.429	-0.367	1.063	
					0.000
	2	1.000	0.000	1.000	
					0.000



Remove 6th floor shores

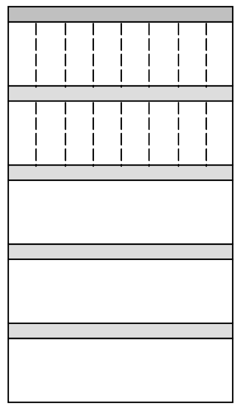
Figure 8.1 (cont.) – Traditional method for shoring and reshoring analysis

Stage	Level				Shore
6 - D	6	1.000	0.000	1.000	
					0.000
	5	1.000	0.006	1.006	
					0.044
	4	1.038	0.006	1.044	
					0.000
	3	1.063	-0.062	1.000	
					0.000
	2	1.000	0.000	1.000	
					0.000



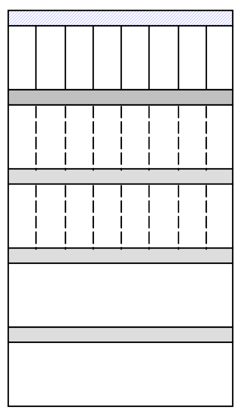
Remove 4th floor reshores

Stage	Level				Shore
6 - E	6	1.000	0.000	1.000	
					0.050
	5	1.006	0.025	1.031	
					0.069
	4	1.044	0.025	1.069	
					0.000
	3	1.000	0.000	1.000	
					0.000
	2	1.000	0.000	1.000	
					0.000



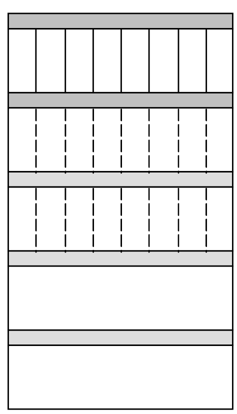
Place 6th floor reshores

Stage	Level				Shore
7 - A	7	0.000	0.000	0.000	
					1.700
	6	1.000	0.567	1.567	
					1.183
	5	1.031	0.567	1.598	
					0.635
	4	1.069	0.567	1.635	
					0.000
	3	1.000	0.000	1.000	
					0.000



Place 7th floor slab

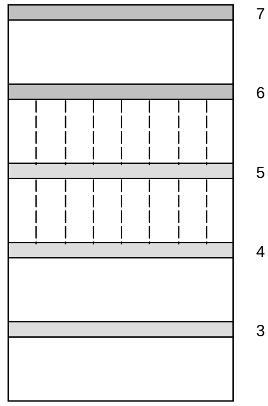
Stage	Level				Shore
7 - B	7	0.000	0.000	0.000	
					1.100
	6	1.567	-0.200	1.367	
					0.783
	5	1.598	-0.200	1.398	
					0.435
	4	1.635	-0.200	1.435	
					0.000
	3	1.000	0.000	1.000	
					0.000



Remove live load

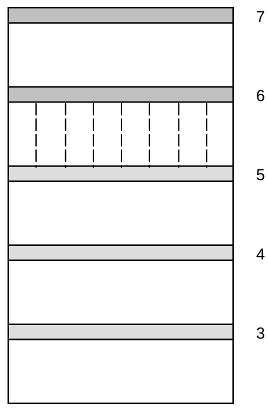
Figure 8.1 (cont.) – Traditional method for shoring and reshoring analysis

7 - C	7	0.000	1.000	1.000	
					0.000
	6	1.367	-0.367	1.000	
					0.050
	5	1.398	-0.367	1.031	
					0.069
	4	1.435	-0.367	1.069	
					0.000
	3	1.000	0.000	1.000	
					0.000



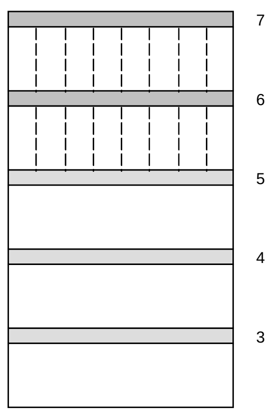
Remove 7th floor shores

7 - D	7	1.000	0.000	1.000	
					0.000
	6	1.000	0.009	1.009	
					0.041
	5	1.031	0.009	1.041	
					0.000
	4	1.069	-0.069	1.000	
					0.000
	3	1.000	0.000	1.000	
					0.000



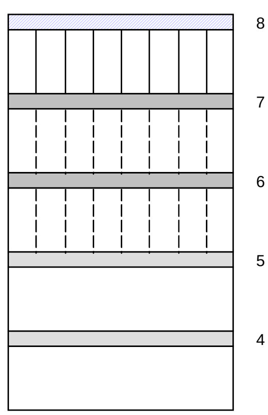
Remove 5th floor reshores

7 - E	7	1.000	0.000	1.000	
					0.050
	6	1.009	0.025	1.034	
					0.066
	5	1.041	0.025	1.066	
					0.000
	4	1.000	0.000	1.000	
					0.000
	3	1.000	0.000	1.000	
					0.000



Place 7th floor reshores

8 - A	8	0.000	0.000	0.000	
					1.700
	7	1.000	0.567	1.567	
					1.183
	6	1.034	0.567	1.601	
					0.632
	5	1.066	0.567	1.632	
					0.000
	4	1.000	0.000	1.000	
					0.000



Place 8th floor slab

Figure 8.1 (cont.) – Traditional method for shoring and reshoring analysis

SHORING AND RESHORING ANALYSIS - SP4 METHOD - MEVA

INPUTS

Panel Sets	1
Shore sets	1
Reshore sets	3

MEMBER	L.F.	LOAD	
slab weight	1.0	1.000	1.000
reinforcing/misc	1.0	0.100	0.100
Panel Weight	1.0	0.050	0.050
Shore weight	1.0	0.050	0.050
Placing Live Load	1.0	0.500	0.500
Forming Live Load	1.0	0.200	0.200
Reshore weight	1.0	0.050	0.050

Activation %	100%

Step #	Slab #	Load carried by slab beginning	Load carried by slab during	Load carried by slab end	Load carried by shores end		
1 - A	1	0.000	0.000	0.000	0.400	1	*Set up 1st floor panels/shores/reinf'*
1 - B	1	0.000	0.000	0.000	0.200	1	*Remove Forming Live Load*
1 - C	1	0.000	0.000	0.000	1.600	1	*Place 1st floor slab*
1 - D	1	0.000	0.000	0.000	1.100	1	*Remove 1st floor live load*
1 - E	1	0.000	0.000	0.000	1.050	1	*Remove 1st floor panels*
2 - A	2	0.000	0.000	0.000	0.400	2	*Set up 2nd floor panels/shores/reinf'*
	1	0.000	0.000	0.000	1.450	1	
2 - B	2	0.000	0.000	0.000	0.200	2	*Remove Forming Live Load*
	1	0.000	0.000	0.000	1.250	1	
2 - C	2	0.000	0.000	0.000	0.200		*Activate 1st floor - Lower Drop Heads*
	1	0.000	1.200	1.200	0.050		
2 - D	2	0.000	0.000	0.000	1.600	2	*Place 2nd floor slab*
	1	1.200	0.000	1.200	1.450	1	

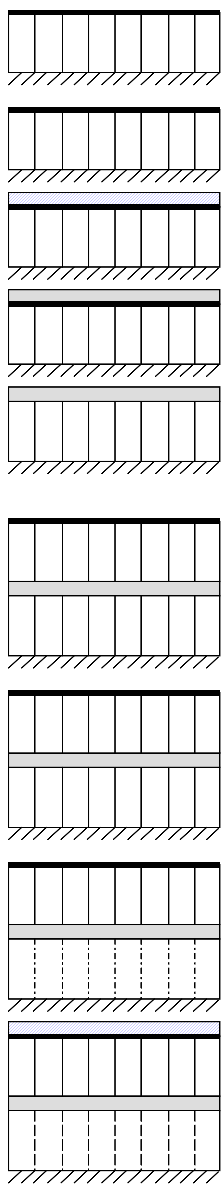


Figure 8.2 – Meva method for shoring and reshoring analysis

Stage	Floor				Shore	Description
2 - E	2	0.000	0.000	0.000		*Remove 2nd floor live load*
					1.100	
	1	1.200	0.000	1.200		
					0.950	
2 - F	2	0.000	0.000	0.000		*Remove 2nd floor panels*
					1.050	
	1	1.200	0.000	1.200		
					0.900	
3 - A	3	0.000	0.000	0.000		*Set up 3rd floor panels/shores/reinf'*
					0.400	
	2	0.000	0.000	0.000		
					1.450	
	1	1.200	0.000	1.200		
					1.300	
3 - B	3	0.000	0.000	0.000		*Remove Forming Live Load*
					0.200	
	2	0.000	0.000	0.000		
					1.250	
	1	1.200	0.000	1.200		
					1.100	
3 - C	3	0.000	0.000	0.000		*Activate 2nd floor - Lower Drop Heads*
					0.200	
	2	0.000	1.200	1.200		
					0.050	
	1	1.200	0.000	1.200		
					-0.100	
3 - D	3	0.000	0.000	0.000		*Place 3rd floor slab*
					1.600	
	2	1.200	0.000	1.200		
					1.450	
	1	1.200	0.000	1.200		
					1.300	
3 - E	3	0.000	0.000	0.000		*Remove 3rd floor live load*
					1.100	
	2	1.200	0.000	1.200		
					0.950	
	1	1.200	0.000	1.200		
					0.800	

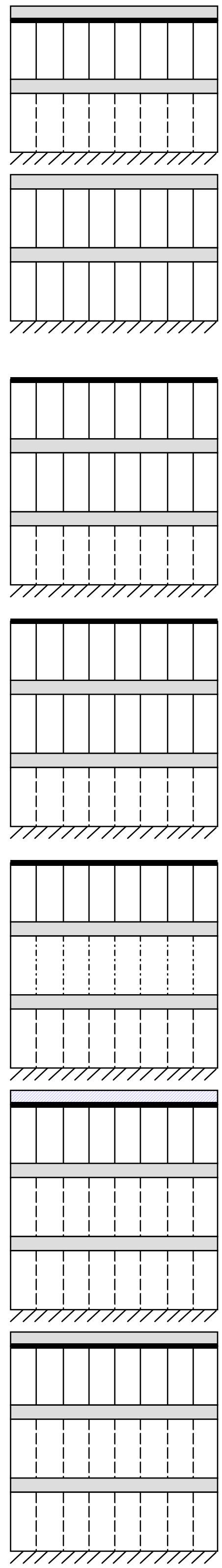
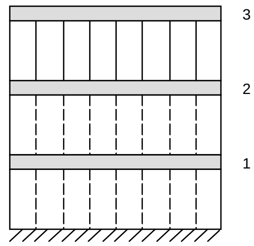


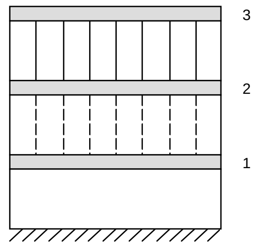
Figure 8.2 (cont.) – Meva method for shoring and reshoring analysis

3 - F	3	0.000	0.000	0.000	
					1.050
	2	1.200	0.000	1.200	
					0.900
	1	1.200	0.000	1.200	
					0.750



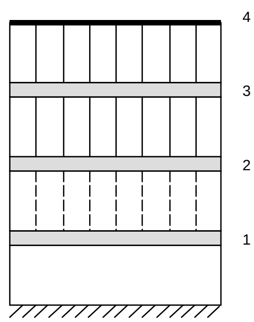
Remove 3rd floor panels

3 - G	3	0.000	0.233	0.233	
					0.817
	2	1.200	0.233	1.433	
					0.433
	1	1.200	0.233	1.433	
					0.000



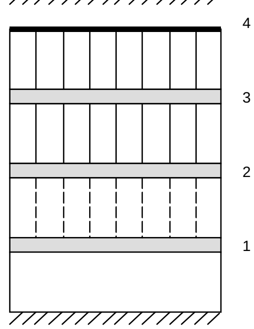
Remove 1st floor reshores

4 - A	4	0.000	0.000	0.000	
					0.400
	3	0.233	0.133	0.367	
					1.083
	2	1.433	0.133	1.567	
					0.567
	1	1.433	0.133	1.567	
					0.000



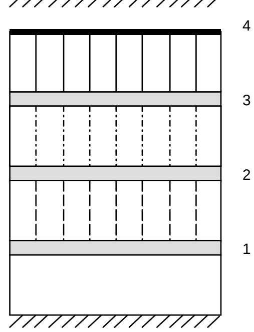
Set up 4th floor panels/shores/reinf'

4 - B	4	0.000	0.000	0.000	
					0.200
	3	0.367	-0.067	0.300	
					0.950
	2	1.567	-0.067	1.500	
					0.500
	1	1.567	-0.067	1.500	
					0.000



Remove Forming Live Load

4 - C	4	0.000	0.000	0.000	
					0.200
	3	0.300	0.900	1.200	
					0.050
	2	1.500	-0.450	1.050	
					0.050
	1	1.500	-0.450	1.050	
					0.000

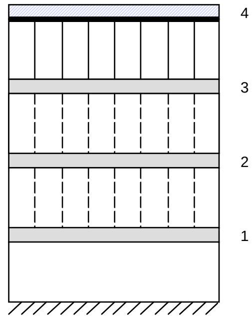


Activate 3rd floor - Lower Drop Heads

Figure 8.2 (cont.) – Meva method for shoring and reshoring analysis

4 - D

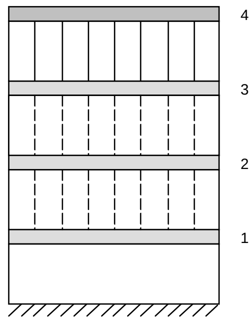
4	0.000	0.000	0.000	
				1.600
3	1.200	0.467	1.667	
				0.983
2	1.050	0.467	1.517	
				0.517
1	1.050	0.467	1.517	
				0.000



Place 4th floor slab

4 - E

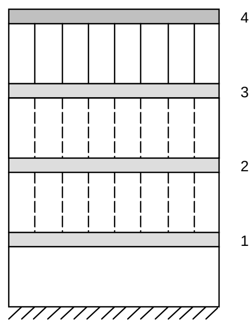
4	0.000	0.000	0.000	
				1.100
3	1.667	-0.167	1.500	
				0.650
2	1.517	-0.167	1.350	
				0.350
1	1.517	-0.167	1.350	
				0.000



Remove 4th floor live load

4 - F

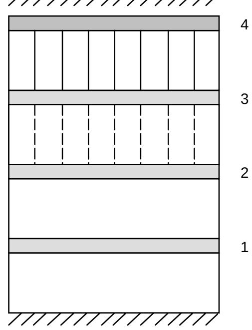
4	0.000	0.000	0.000	
				1.050
3	1.500	-0.017	1.483	
				0.617
2	1.350	-0.017	1.333	
				0.333
1	1.350	-0.017	1.333	
				0.000



Remove 4th floor panels

4 - G

4	0.000	0.094	0.094	
				0.956
3	1.483	0.094	1.578	
				0.428
2	1.333	0.094	1.428	
				0.000
1	1.333	-0.333	1.000	
				0.000

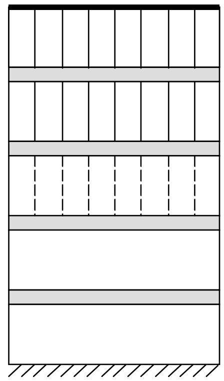


Remove 2nd floor reshores

Figure 8.2 (cont.) – Meva method for shoring and reshoring analysis

5 - A

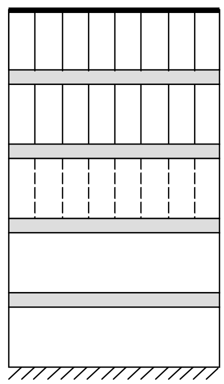
Level				Shore
5	0.000	0.000	0.000	
				0.400
4	0.094	0.133	0.228	
				1.222
3	1.578	0.133	1.711	
				0.561
2	1.428	0.133	1.561	
				0.000
1	1.000	0.000	1.000	
				0.000



Set up 5th floor panels/shores/reinf'

5 - B

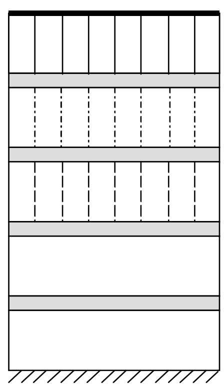
Level				Shore
5	0.000	0.000	0.000	
				0.200
4	0.228	-0.067	0.161	
				1.089
3	1.711	-0.067	1.644	
				0.494
2	1.561	-0.067	1.494	
				0.000
1	1.000	0.000	1.000	
				0.000



Remove Forming Live Load

5 - C

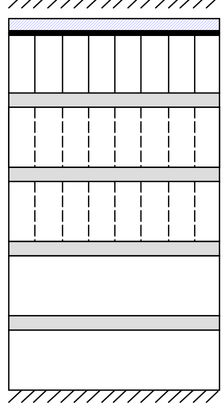
Level				Shore
5	0.000	0.000	0.000	
				0.200
4	0.161	1.039	1.200	
				0.050
3	1.644	-0.519	1.125	
				-0.025
2	1.494	-0.519	0.975	
				0.000
1	1.000	0.000	1.000	
				0.000



Activate 4th floor - Lower Drop Heads

5 - D

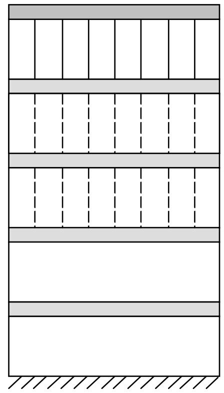
Level				Shore
5	0.000	0.000	0.000	
				1.600
4	1.200	0.467	1.667	
				0.983
3	1.125	0.467	1.592	
				0.442
2	0.975	0.467	1.442	
				0.000
1	1.000	0.000	1.000	
				0.000



Place 5th floor slab

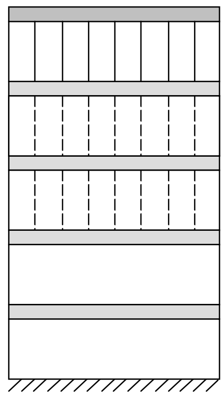
Figure 8.2 (cont.) – Meva method for shoring and reshoring analysis

Stage	Floor				Shores
5 - E	5	0.000	0.000	0.000	
					1.100
	4	1.667	-0.167	1.500	
					0.650
	3	1.592	-0.167	1.425	
					0.275
	2	1.442	-0.167	1.275	
					0.000
	1	1.000	0.000	1.000	
					0.000



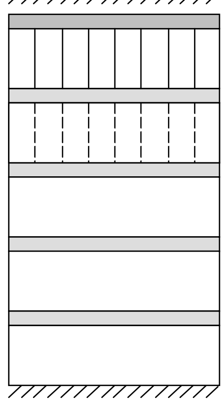
Remove 5th floor live load

Stage	Floor				Shores
5 - F	5	0.000	0.000	0.000	
					1.050
	4	1.500	-0.017	1.483	
					0.617
	3	1.425	-0.017	1.408	
					0.258
	2	1.275	-0.017	1.258	
					0.000
	1	1.000	0.000	1.000	
					0.000



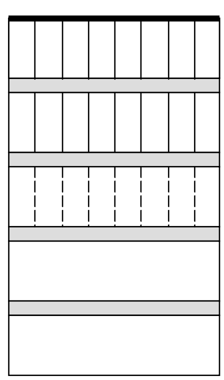
Remove 5th floor panels

Stage	Floor				Shores
5 - G	5	0.000	0.069	0.069	
					0.981
	4	1.483	0.069	1.553	
					0.478
	3	1.408	0.069	1.478	
					0.000
	2	1.258	-0.258	1.000	
					0.000
	1	1.000	0.000	1.000	
					0.000



Remove 3rd floor reshores

Stage	Floor				Shores
6 - A	6	0.000	0.000	0.000	
					0.400
	5	0.069	0.133	0.203	
					1.247
	4	1.553	0.133	1.686	
					0.611
	3	1.478	0.133	1.611	
					0.000
	2	1.000	0.000	1.000	
					0.000

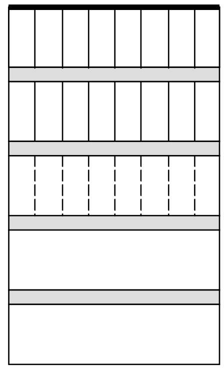


Set up 6th floor panels/shores/reinf'

Figure 8.2 (cont.) – Meva method for shoring and reshoring analysis

6 - B

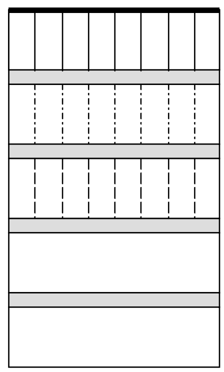
Floor				Shore load
6	0.000	0.000	0.000	
				0.200
5	0.203	-0.067	0.136	
				1.114
4	1.686	-0.067	1.619	
				0.544
3	1.611	-0.067	1.544	
				0.000
2	1.000	0.000	1.000	
				0.000



Remove Forming Live Load

6 - C

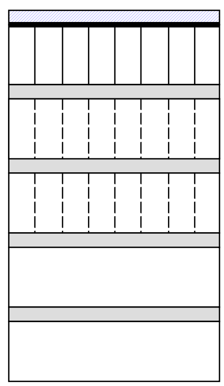
Floor				Shore load
6	0.000	0.000	0.000	
				0.200
5	0.136	1.064	1.200	
				0.050
4	1.619	-0.532	1.088	
				0.013
3	1.544	-0.532	1.013	
				0.000
2	1.000	0.000	1.000	
				0.000



Activate 5th floor - Lower Drop Heads

6 - D

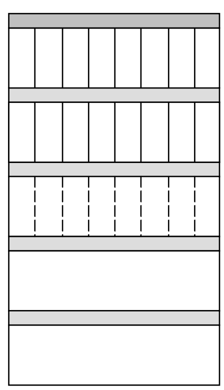
Floor				Shore load
6	0.000	0.000	0.000	
				1.600
5	1.200	0.467	1.667	
				0.983
4	1.088	0.467	1.554	
				0.479
3	1.013	0.467	1.479	
				0.000
2	1.000	0.000	1.000	
				0.000



Place 6th floor slab

6 - E

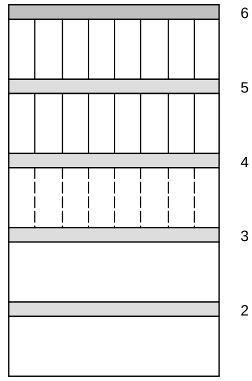
Floor				Shore load
6	0.000	0.000	0.000	
				1.100
5	1.667	-0.167	1.500	
				0.650
4	1.554	-0.167	1.388	
				0.313
3	1.479	-0.167	1.313	
				0.000
2	1.000	0.000	1.000	
				0.000



Remove 6th floor live load

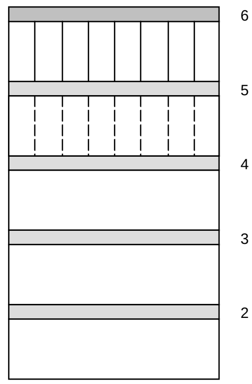
Figure 8.2 (cont.) – Meva method for shoring and reshoring analysis

Stage	Floor				Shore load
6 - F	6	0.000	0.000	0.000	
					1.050
	5	1.500	-0.017	1.483	
					0.617
	4	1.388	-0.017	1.371	
					0.296
	3	1.313	-0.017	1.296	
					0.000
	2	1.000	0.000	1.000	
					0.000



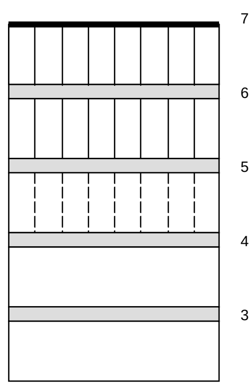
Remove 6th floor panels

Stage	Floor				Shore load
6 - G	6	0.000	0.082	0.082	
					0.968
	5	1.483	0.082	1.565	
					0.453
	4	1.371	0.082	1.453	
					0.000
	3	1.296	-0.296	1.000	
					0.000
	2	1.000	0.000	1.000	
					0.000



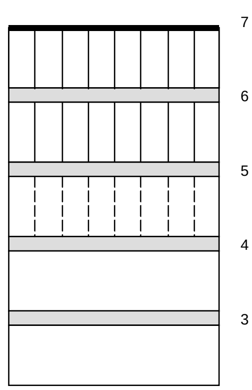
Remove 4th floor reshores

Stage	Floor				Shore load
7 - A	7	0.000	0.000	0.000	
					0.400
	6	0.082	0.133	0.215	
					1.235
	5	1.565	0.133	1.699	
					0.586
	4	1.453	0.133	1.586	
					0.000
	3	1.000	0.000	1.000	
					0.000



Set up 7th floor panels/shores/reinf'

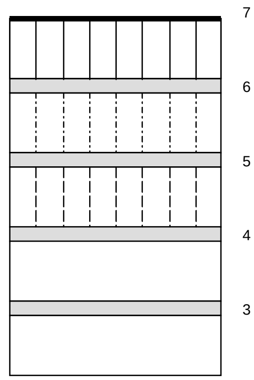
Stage	Floor				Shore load
7 - B	7	0.000	0.000	0.000	
					0.200
	6	0.215	-0.067	0.149	
					1.101
	5	1.699	-0.067	1.632	
					0.519
	4	1.586	-0.067	1.519	
					0.000
	3	1.000	0.000	1.000	
					0.000



Remove Forming Live Load

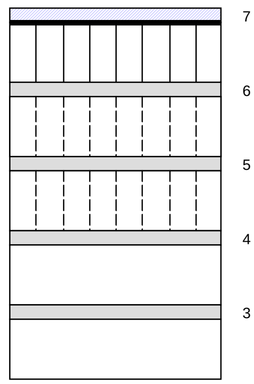
Figure 8.2 (cont.) – Meva method for shoring and reshoring analysis

Stage	Floor				Shore load
7 - C	7	0.000	0.000	0.000	
					0.200
	6	0.149	1.051	1.200	
					0.050
	5	1.632	-0.526	1.106	
					-0.006
	4	1.519	-0.526	0.994	
					0.000
	3	1.000	0.000	1.000	
					0.000



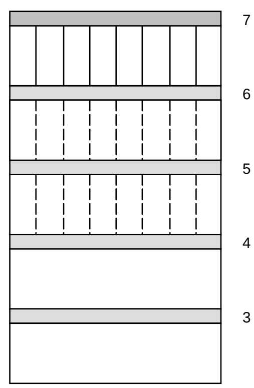
Activate 6th floor - Lower Drop Heads

Stage	Floor				Shore load
7 - D	7	0.000	0.000	0.000	
					1.600
	6	1.200	0.467	1.667	
					0.983
	5	1.106	0.467	1.573	
					0.460
	4	0.994	0.467	1.460	
					0.000
	3	1.000	0.000	1.000	
					0.000



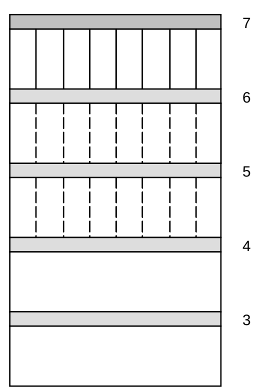
Place 7th floor slab

Stage	Floor				Shore load
7 - E	7	0.000	0.000	0.000	
					1.100
	6	1.667	-0.167	1.500	
					0.650
	5	1.573	-0.167	1.406	
					0.294
	4	1.460	-0.167	1.294	
					0.000
	3	1.000	0.000	1.000	
					0.000



Remove Placing Live Load

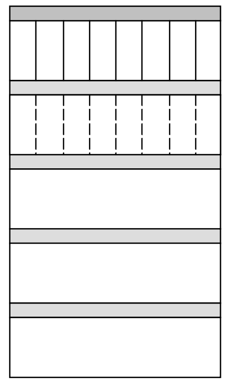
Stage	Floor				Shore load
7 - F	7	0.000	0.000	0.000	
					1.050
	6	1.500	-0.017	1.483	
					0.617
	5	1.406	-0.017	1.390	
					0.277
	4	1.294	-0.017	1.277	
					0.000
	3	1.000	0.000	1.000	
					0.000



Remove 7th floor panels

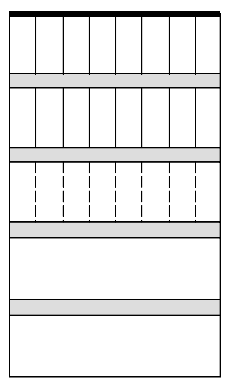
Figure 8.2 (cont.) – Meva method for shoring and reshoring analysis

Stage	Floor				Shores
7 - G	7	0.000	0.076	0.076	
					0.974
	6	1.483	0.076	1.559	
					0.465
	5	1.390	0.076	1.465	
					0.000
	4	1.277	-0.277	1.000	
					0.000
	3	1.000	0.000	1.000	
					0.000



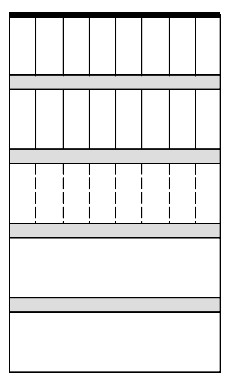
Remove 5th floor reshores

Stage	Floor				Shores
8 - A	8	0.000	0.000	0.000	
					0.400
	7	0.076	0.133	0.209	
					1.241
	6	1.559	0.133	1.692	
					0.599
	5	1.465	0.133	1.599	
					0.000
	4	1.000	0.000	1.000	
					0.000



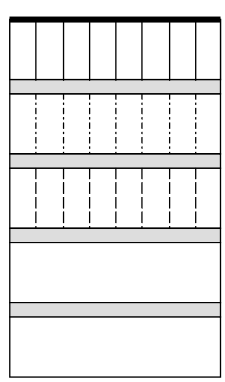
Set up 8th floor panels/shores/reinf'

Stage	Floor				Shores
8 - B	8	0.000	0.000	0.000	
					0.200
	7	0.209	-0.067	0.142	
					1.108
	6	1.692	-0.067	1.626	
					0.532
	5	1.599	-0.067	1.532	
					0.000
	4	1.000	0.000	1.000	
					0.000



Remove Forming Live Load

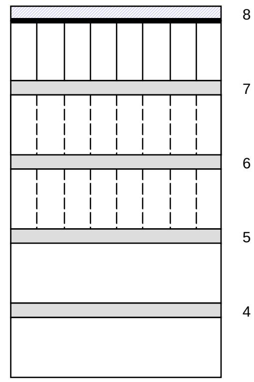
Stage	Floor				Shores
8 - C	8	0.000	0.000	0.000	
					0.200
	7	0.142	1.058	1.200	
					0.050
	6	1.626	-0.529	1.097	
					0.003
	5	1.532	-0.529	1.003	
					0.000
	4	1.000	0.000	1.000	
					0.000



Activate 7th floor - Lower Drop Heads

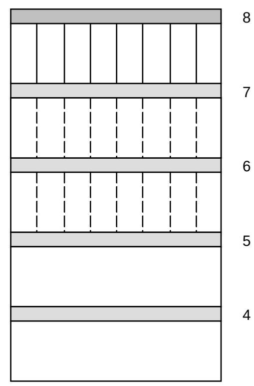
Figure 8.2 (cont.) – Meva method for shoring and reshoring analysis

8 - D	8	0.000	0.000	0.000	
					1.600
	7	1.200	0.467	1.667	
					0.983
	6	1.097	0.467	1.564	
					0.470
	5	1.003	0.467	1.470	
					0.000
	4	1.000	0.000	1.000	
					0.000



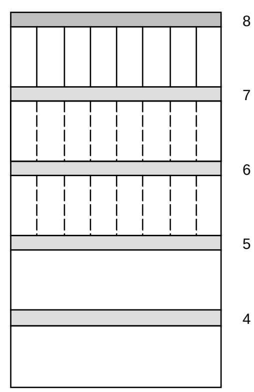
Place 8th floor slab

8 - E	8	0.000	0.000	0.000	
					1.100
	7	1.667	-0.167	1.500	
					0.650
	6	1.564	-0.167	1.397	
					0.303
	5	1.470	-0.167	1.303	
					0.000
	4	1.000	0.000	1.000	
					0.000



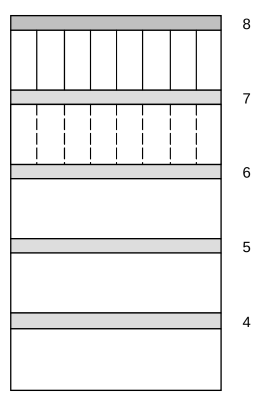
Remove 8th floor live load

8 - F	8	0.000	0.000	0.000	
					1.050
	7	1.500	-0.017	1.483	
					0.617
	6	1.397	-0.017	1.380	
					0.286
	5	1.303	-0.017	1.286	
					0.000
	4	1.000	0.000	1.000	
					0.000



Remove 8th floor panels

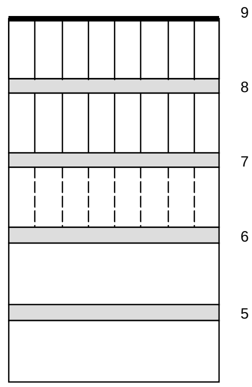
8 - G	8	0.000	0.079	0.079	
					0.971
	7	1.483	0.079	1.562	
					0.459
	6	1.380	0.079	1.459	
					0.000
	5	1.286	-0.286	1.000	
					0.000
	4	1.000	0.000	1.000	
					0.000



Remove 6th floor reshores

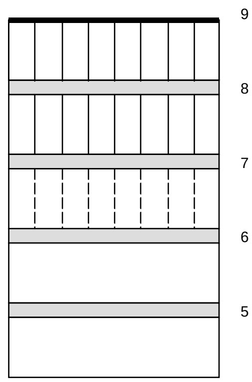
Figure 8.2 (cont.) – Meva method for shoring and reshoring analysis

9 - A	9	0.000	0.000	0.000	
					0.400
	8	0.079	0.133	0.212	
					1.238
	7	1.562	0.133	1.695	
					0.592
	6	1.459	0.133	1.592	
					0.000
	5	1.000	0.000	1.000	
					0.000



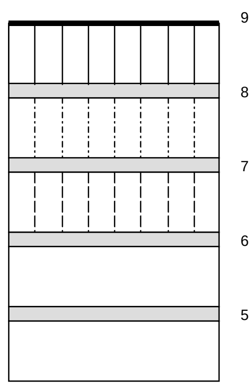
Set up 9th floor panels/shores/reinf'

9 - B	9	0.000	0.000	0.000	
					0.200
	8	0.212	-0.067	0.145	
					1.105
	7	1.695	-0.067	1.629	
					0.526
	6	1.592	-0.067	1.526	
					0.000
	5	1.000	0.000	1.000	
					0.000



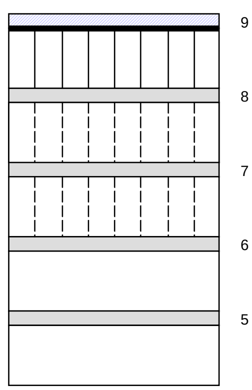
Remove Forming Live Load

9 - C	9	0.000	0.000	0.000	
					0.200
	8	0.145	1.055	1.200	
					0.050
	7	1.629	-0.527	1.102	
					-0.002
	6	1.526	-0.527	0.998	
					0.000
	5	1.000	0.000	1.000	
					0.000



Activate 8th floor - Lower Drop Heads

9 - D	9	0.000	0.000	0.000	
					1.600
	8	1.200	0.467	1.667	
					0.983
	7	1.102	0.467	1.568	
					0.465
	6	0.998	0.467	1.465	
					0.000
	5	1.000	0.000	1.000	
					0.000



Place 9th floor slab

Figure 8.2 (cont.) – Meva method for shoring and reshoring analysis

8.2 Structural Integrity Analysis

This appendix provides the complete analysis results for the load capacity of a slab supported on shores subject to construction loads for cases summarized in Table 8.1. The rate at which various slab thicknesses gained strength were investigated for punching shear, flexure, and beam shear. Each case also considers the effect of the possible distances between the shores when using the different beam sizes available for the Meva Shoring System. For punching shear and flexure, it was also of interest to examine the case in which there was not any reinforcement in the slab on the basis of plain concrete. Slab cracking strength and crack development were examined. For beam shear, the maximum shear to be resisted was assumed at the center of the support.

The slab thicknesses that were investigated ranged from a “d” of 6” to 12”. It did not seem necessary to include slab thickness greater than that of 12”. It also did not seem to be of great benefit to include slabs that would eventually have a concrete compressive strength greater than 4500 psi. Most slabs in multistory construction have a 28-day compressive strength of 4000 to 4500 psi.

Table 8.1 – Analysis cases

	Slab Type	Assumption	Relation to Beam Direction	Beam Size - cm (Figure No.)			
Punching Shear	Reinforced		Parallel	270 (8.3)	210 (8.4)	160 (8.5)	80 (8.6)
	Plain Conc.		Parallel	270 (8.7)	210 (8.8)	160 (8.9)	80 (8.10)
Flexural Strength	Reinforced	One Way	Parallel	270 (8.11)	210 (8.13)	160 (8.15)	80 (8.17)
			Perpendicular	270 (8.12)	210 (8.14)	160 (8.16)	80 (8.18)
		Two Way	Parallel	270 (8.19)	210 (8.21)	160 (8.23)	80 (8.25)
			Perpendicular	270 (8.20)	210 (8.22)	160 (8.24)	80 (8.26)
	Plain Conc.	One Way	Parallel	270 (8.27)	210 (8.29)	160 (8.31)	80 (8.33)
			Perpendicular	270 (8.28)	210 (8.30)	160 (8.32)	80 (8.34)
		Two Way	Parallel	270 (8.35)	210 (8.37)	160 (8.39)	80 (8.41)
			Perpendicular	270 (8.36)	210 (8.38)	160 (8.40)	80 (8.42)
	Crack Development	One Way	Parallel	270 (8.43)	210 (8.45)	160 (8.47)	80 (8.49)
			Perpendicular	270 (8.44)	210 (8.46)	160 (8.48)	80 (8.50)
		Two Way	Parallel	270 (8.51)	210 (8.53)	160 (8.55)	80 (8.57)
			Perpendicular	270 (8.52)	210 (8.54)	160 (8.56)	80 (8.58)
Beam	Reinforced		Parallel	270 (8.59)	210 (8.60)	160 (8.61)	80 (8.62)

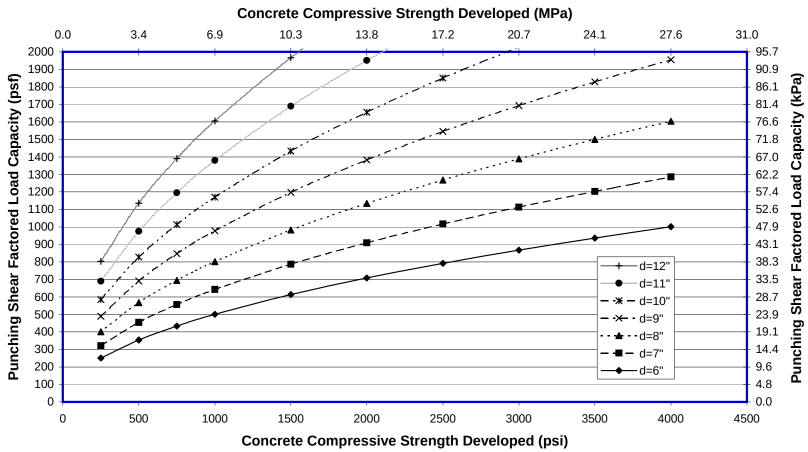


Figure 8.3 - Reinforced slab shore punching shear design strength load capacity (100x100 mm shore, 2.70 m primary beam, 1.6 m panel)

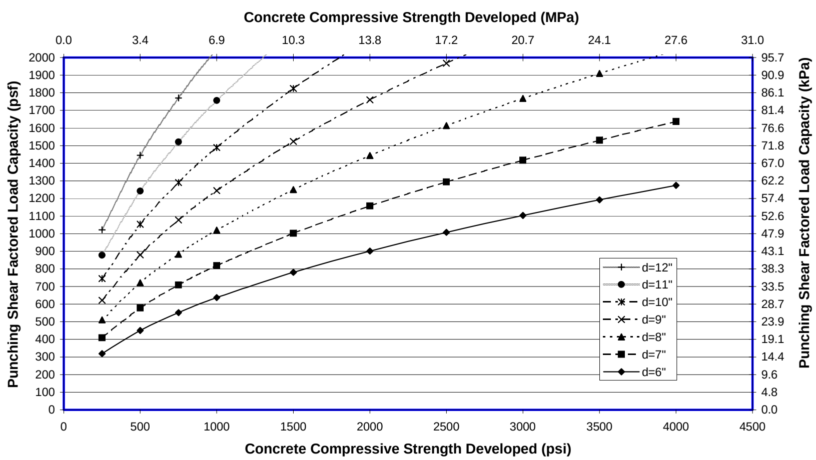


Figure 8.4 – Reinforced slab shore punching shear design strength load capacity (100x100 mm shore, 2.10 m primary beam, 1.6 m panel)

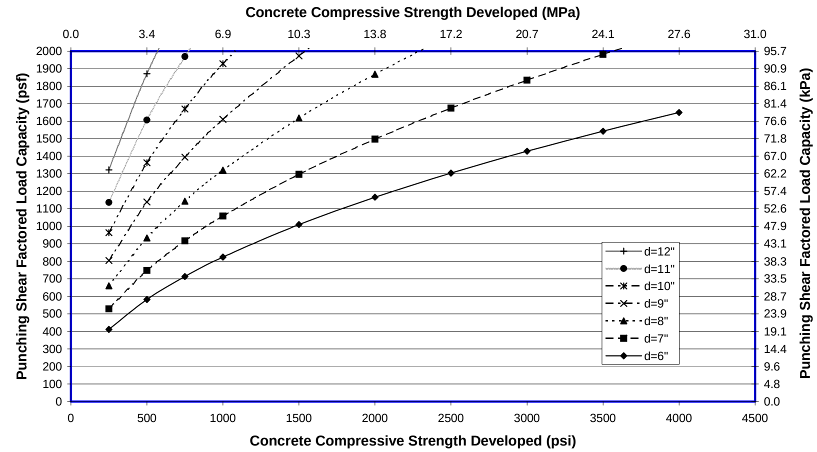


Figure 8.5 - Reinforced slab shore punching shear design strength load capacity (100x100 mm shore, 1.60 m primary beam, 1.6 m panel)

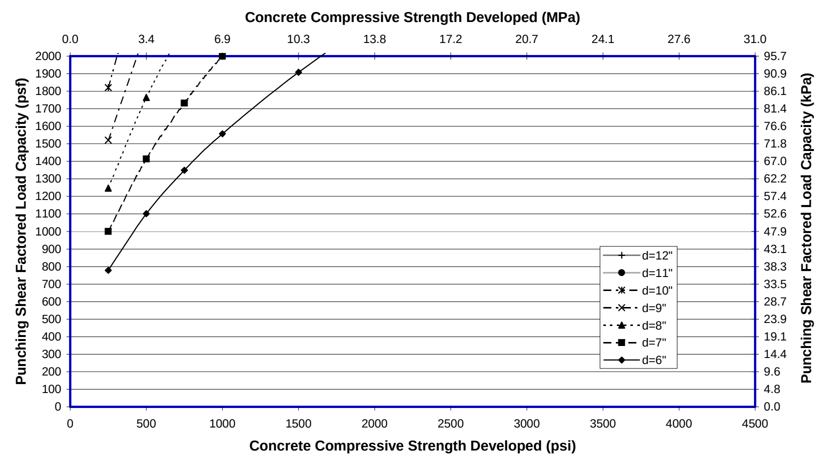


Figure 8.6 – Reinforced slab shore punching shear design strength load capacity (100x100 mm shore, 0.8 m primary beam, 1.6 m panel)

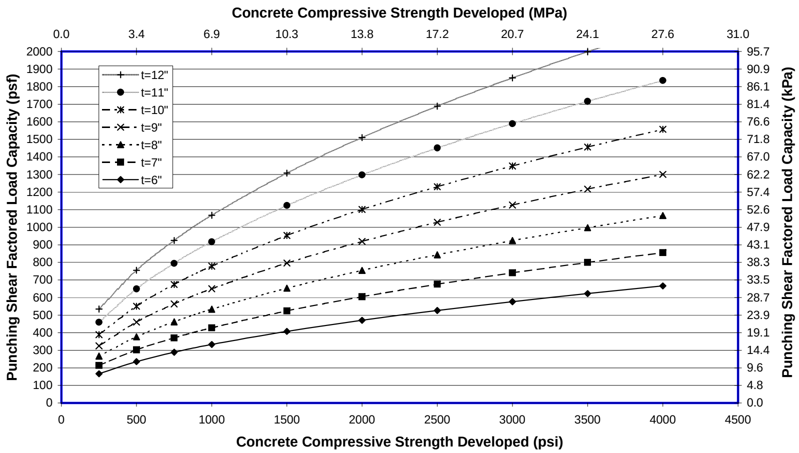


Figure 8.7 – Plain slab shore punching shear design strength load capacity (100x100 mm shore, 2.7 m primary beam, 1.6 m panel)

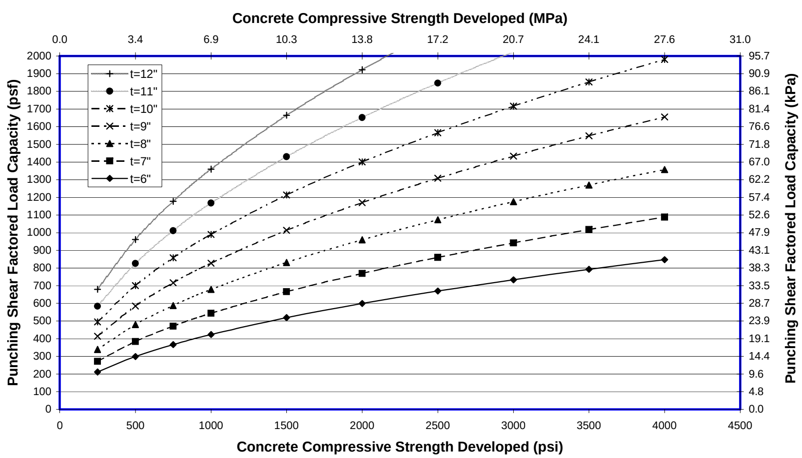


Figure 8.8 – Plain slab shore punching shear design strength load capacity (100x100 mm shore, 2.1 m primary beam, 1.6 m panel)

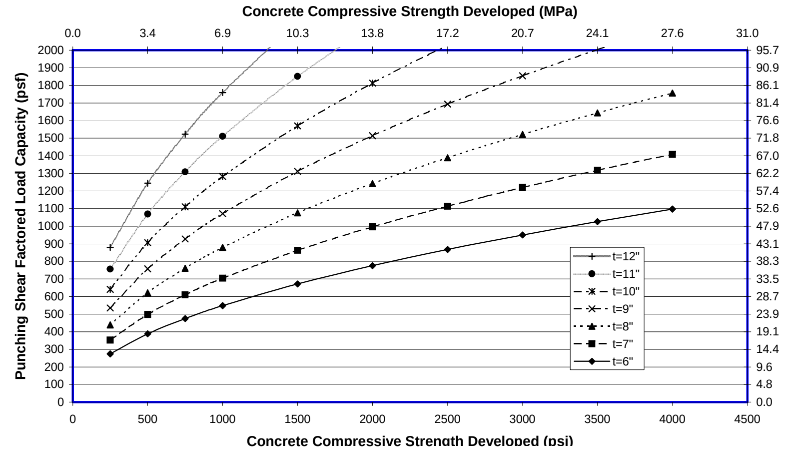


Figure 8.9 – Plain slab shore punching shear design strength load capacity (100x100 mm shore, 1.6 m primary beam, 1.6 m panel)

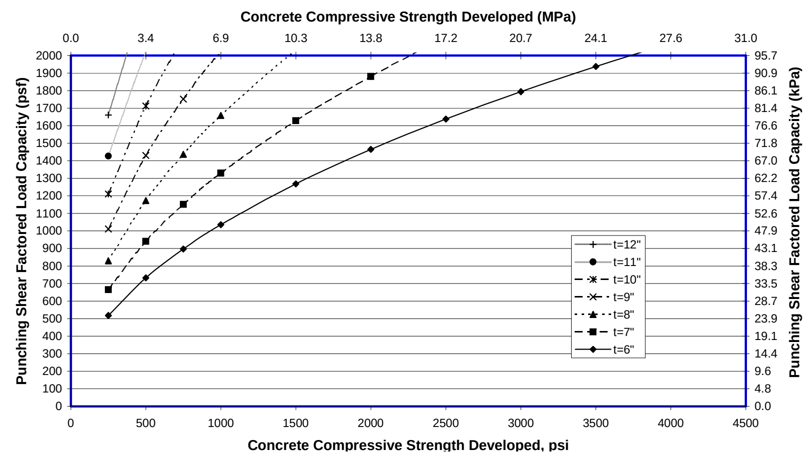


Figure 8.10 – Plain slab shore punching shear design strength load capacity (100x100 mm shore, 0.8 m primary beam, 1.6 m panel)

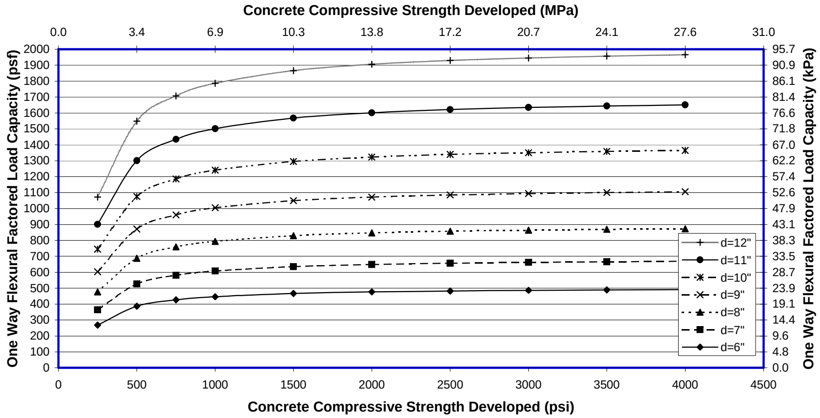


Figure 8.11 - Reinforced slab shore one way flexural factored load capacity (100x100 mm shore, 2.70 m primary beam, 1.6 m panel)

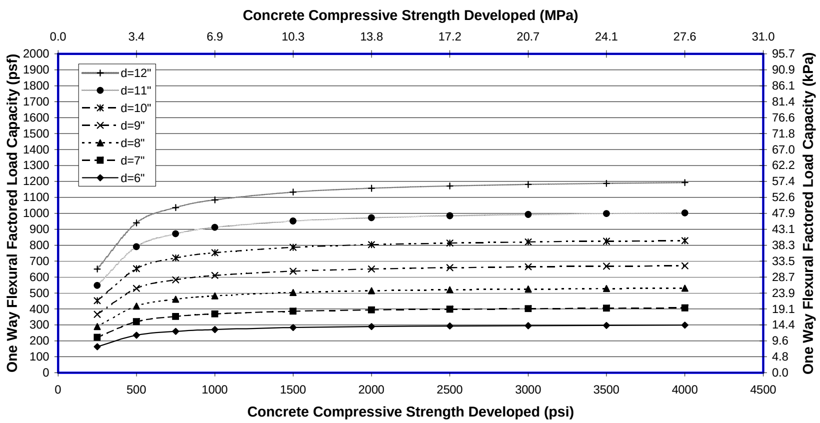


Figure 8.12 - Reinforced slab shore one way flexural factored load capacity (100x100 mm shore, 2.70 m primary beam, 1.6 m panel)

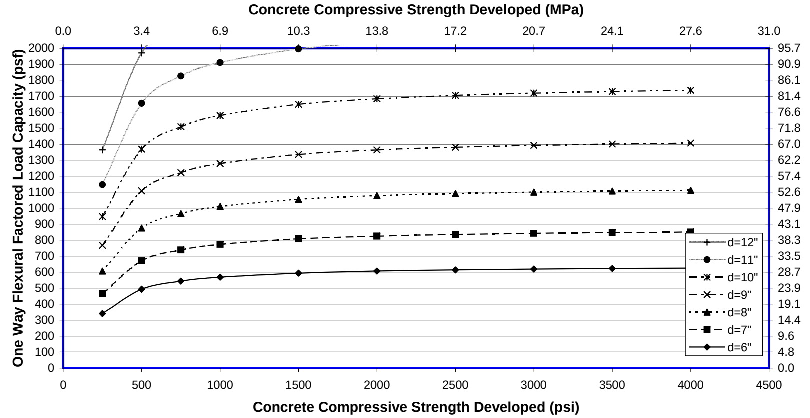


Figure 8.13 - Reinforced slab shore one way flexural factored load capacity (100x100 mm shore, 2.10 m primary beam, 1.6 m panel)

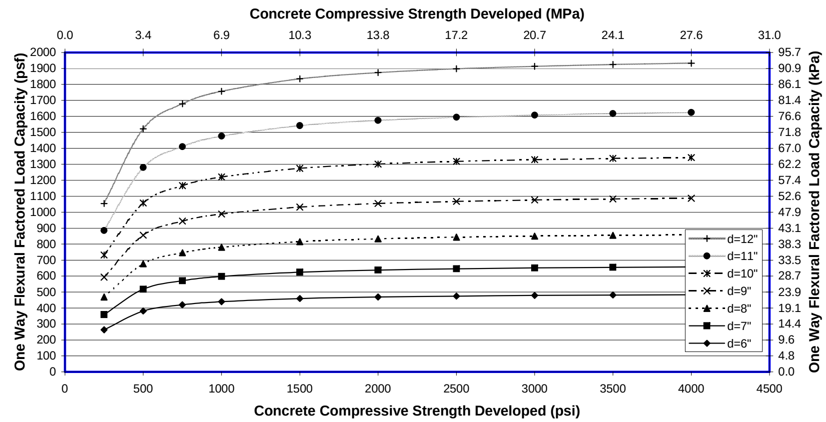


Figure 8.14 - Reinforced slab shore one way flexural factored load capacity (100x100 mm shore, 2.10 m primary beam, 1.6 m panel)

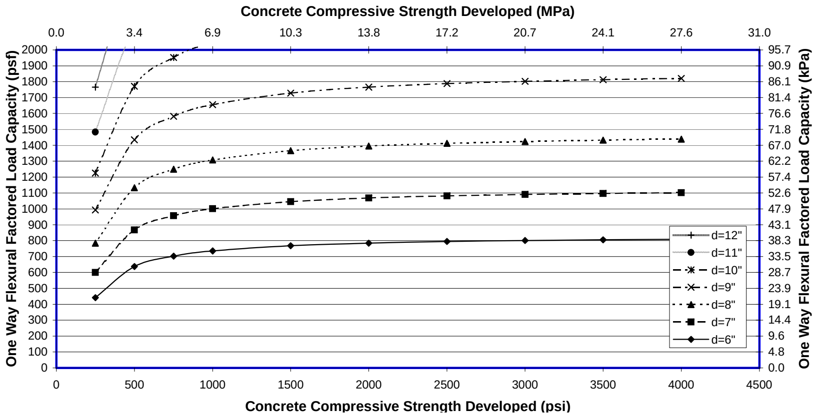


Figure 8.15 - Reinforced slab shore one way flexural factored load capacity
(100x100 mm shore, 1.60 m primary beam, 1.6 m panel)

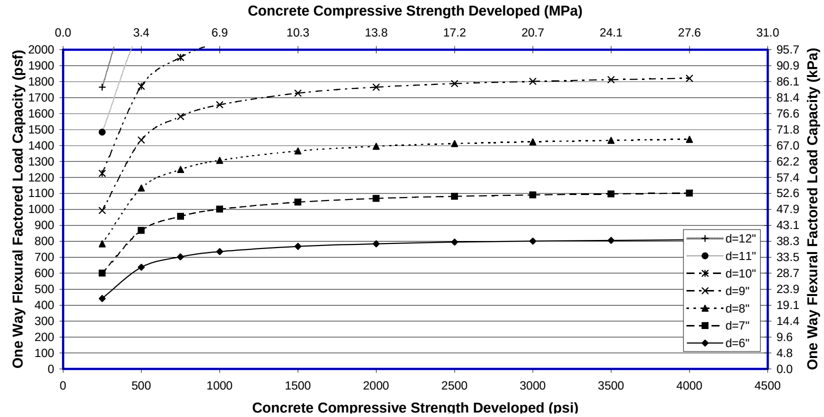


Figure 8.17 - Reinforced slab shore one way flexural factored load capacity
(100x100 mm shore, 0.8 m primary beam, 1.6 m panel)

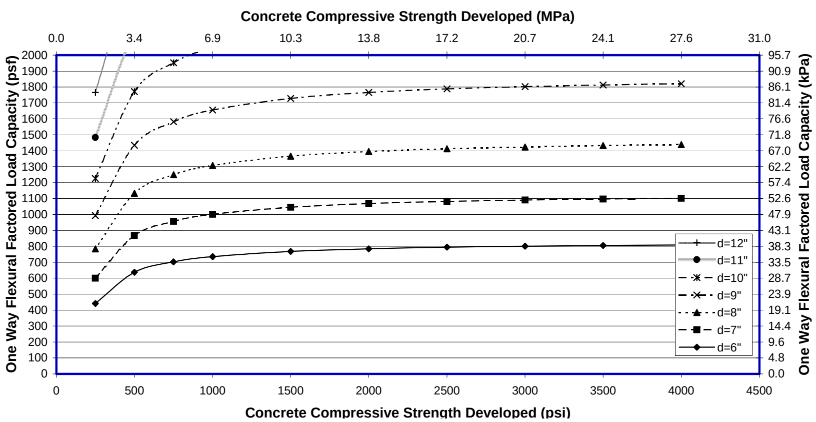


Figure 8.16 - Reinforced slab shore one way flexural factored load capacity
(100x100 mm shore, 1.60 m primary beam, 1.6 m panel)

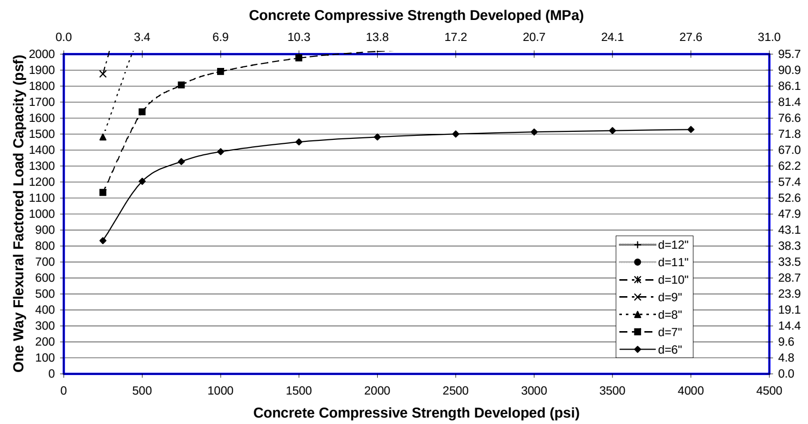


Figure 8.18 - Reinforced slab shore one way flexural factored load capacity
(100x100 mm shore, 0.8 m primary beam, 1.6 m panel)

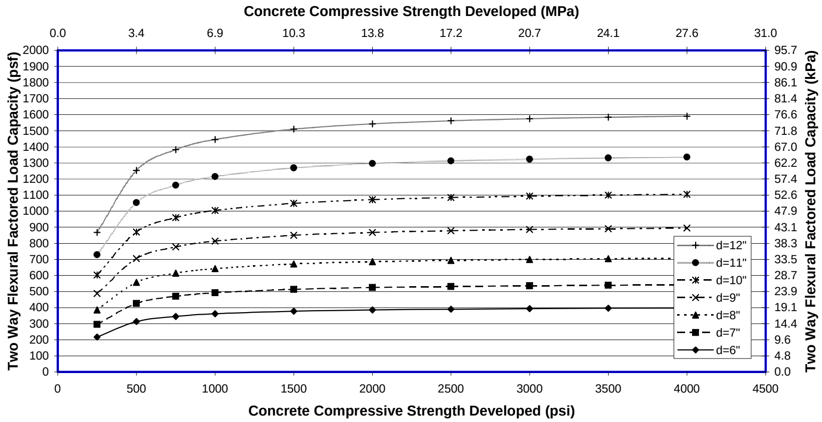


Figure 8.19 - Reinforced slab shore two way flexural factored load capacity
(100x100 mm shore, 2.70 m primary beam, 1.6 m panel)

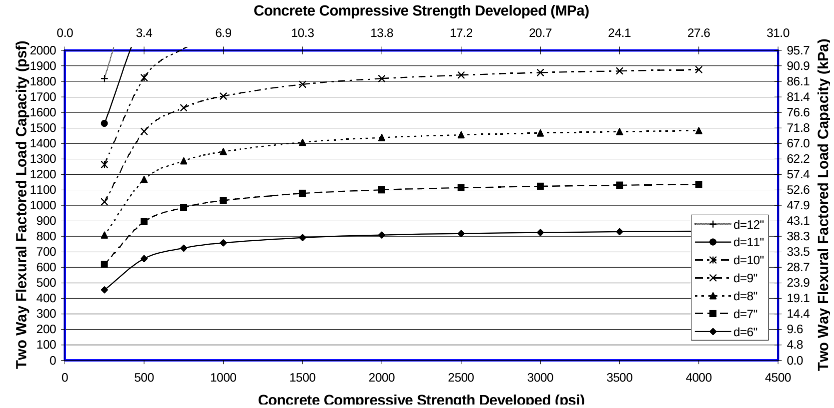


Figure 8.21 - Reinforced slab shore two way flexural factored load capacity
(100x100 mm shore, 2.10 m primary beam, 1.6 m panel)

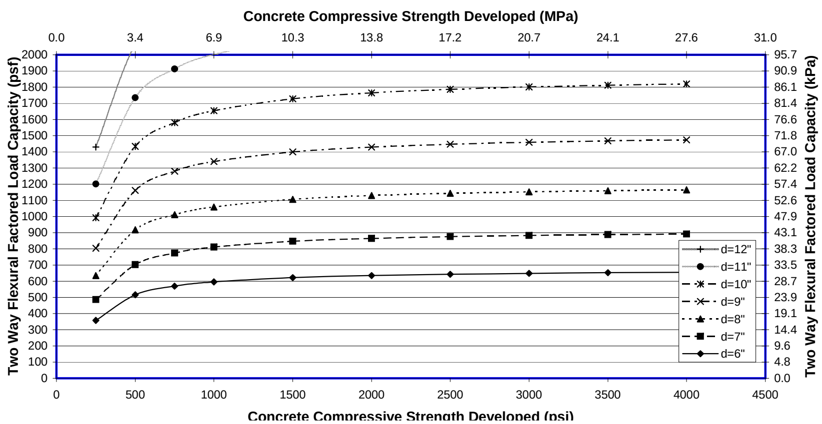


Figure 8.20 - Reinforced slab shore two way flexural factored load capacity
(100x100 mm shore, 2.70 m primary beam, 1.6 m panel)

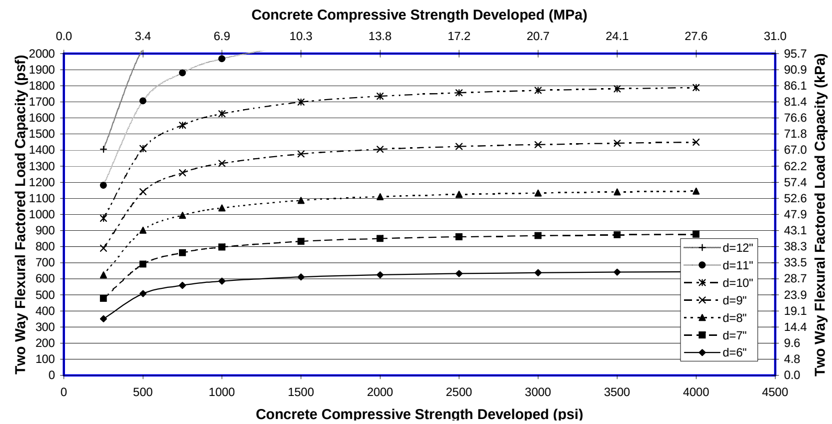


Figure 8.22 - Reinforced slab shore two way flexural factored load capacity
(100x100 mm shore, 2.10 m primary beam, 1.6 m panel)

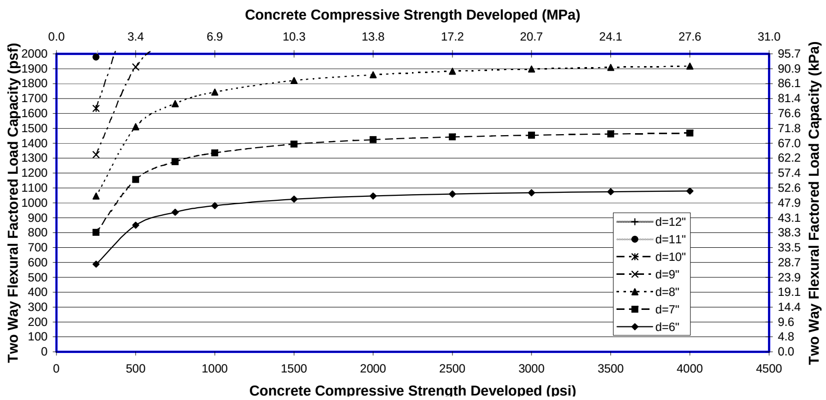


Figure 8.23 - Reinforced slab shore two way flexural factored load capacity (100x100 mm shore, 1.60 m primary beam, 1.6 m panel)

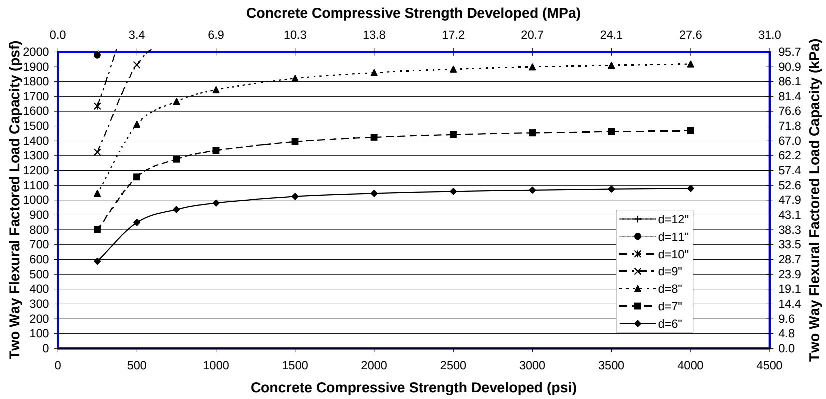


Figure 8.25 - Reinforced slab shore two way flexural factored load capacity (100x100 mm shore, 0.8 m primary beam, 1.6 m panel)

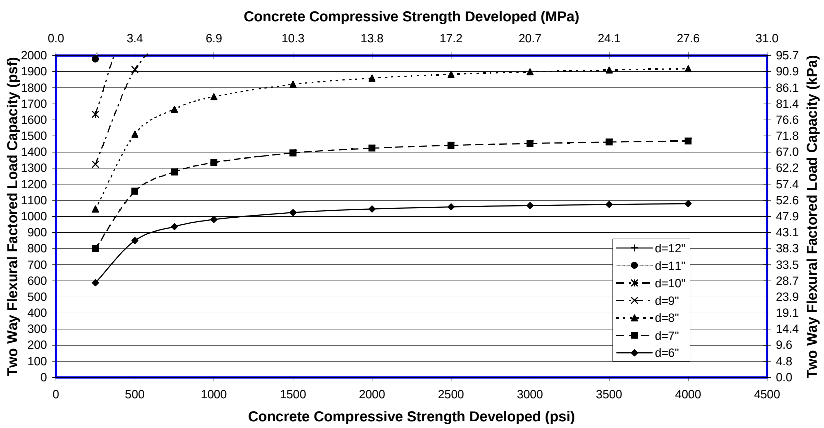


Figure 8.24 - Reinforced slab shore two way flexural factored load capacity (100x100 mm shore, 1.60 m primary beam, 1.6 m panel)

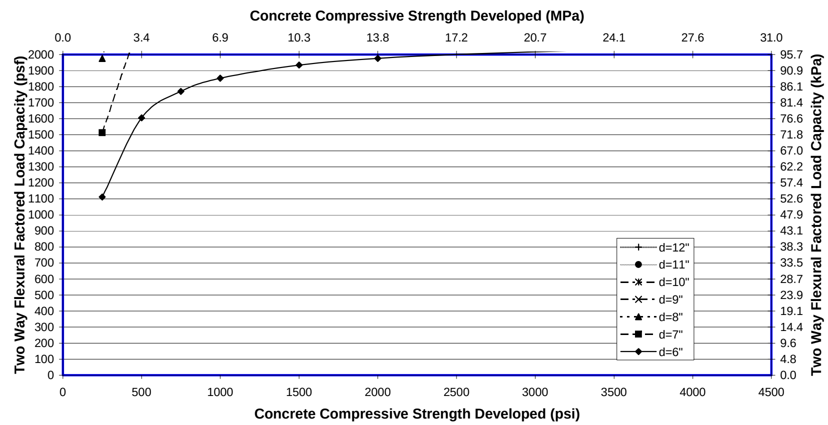


Figure 8.26 - Reinforced slab shore two way flexural factored load capacity (100x100 mm shore, 0.8 m primary beam, 1.6 m panel)

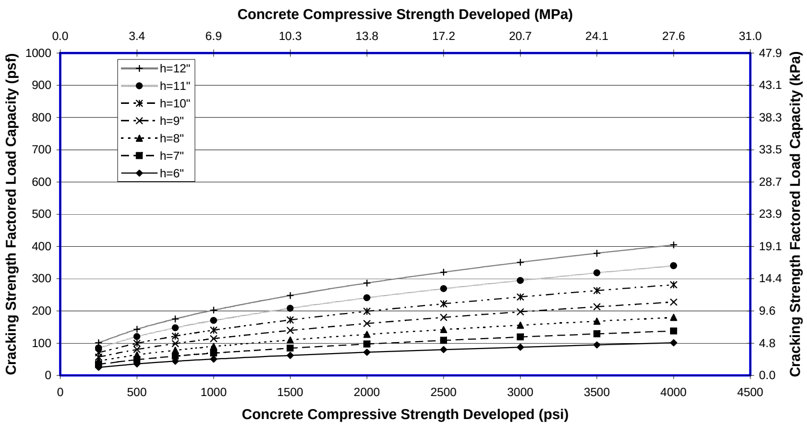


Figure 8.27- Plain slab shore one way cracking strength factored load capacity (100x100 mm shore, 2.70 m primary beam, 1.6 m panel)

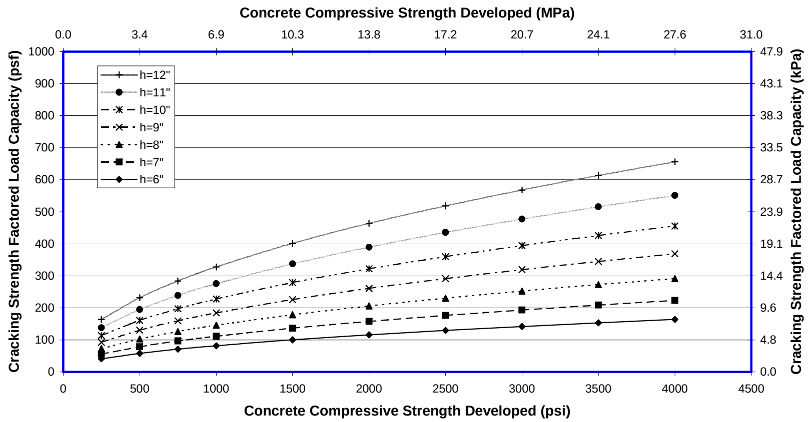


Figure 8.29 - Plain slab shore one way cracking strength factored load capacity (100x100 mm shore, 2.10 m primary beam, 1.6 m panel)

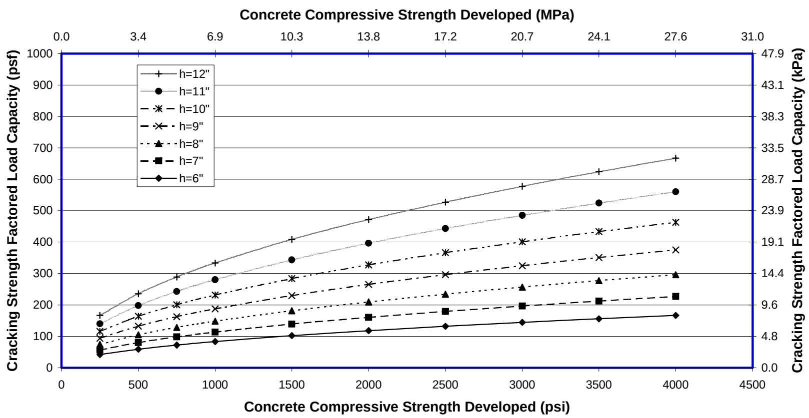


Figure 8.28 - Plain slab shore one way cracking strength factored load capacity (100x100 mm shore, 2.70 m primary beam, 1.6 m panel)

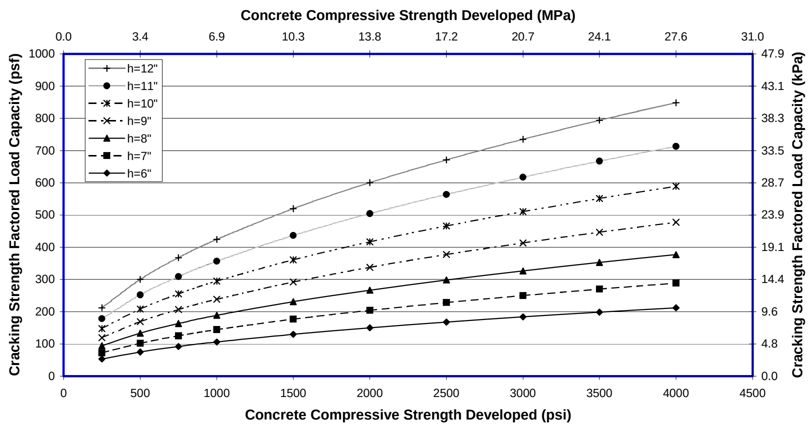


Figure 8.30 - Plain slab shore one way cracking strength factored load capacity (100x100 mm shore, 2.10 m primary beam, 1.6 m panel)

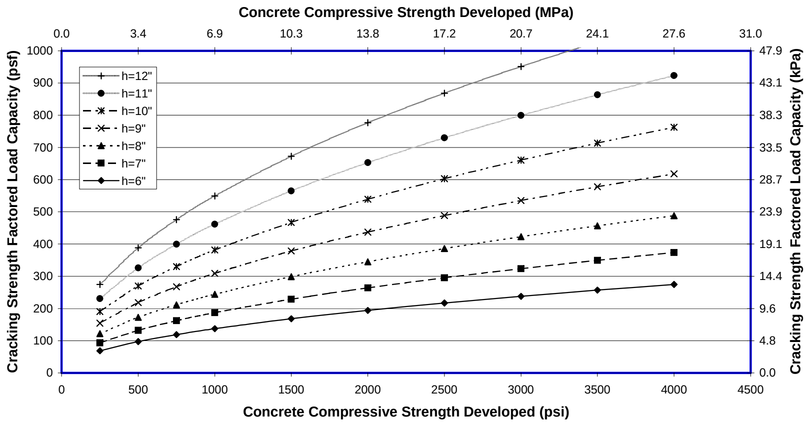


Figure 8.31 - Plain slab shore one way cracking strength factored load capacity (100x100 mm shore, 1.60 m primary beam, 1.6 m panel)

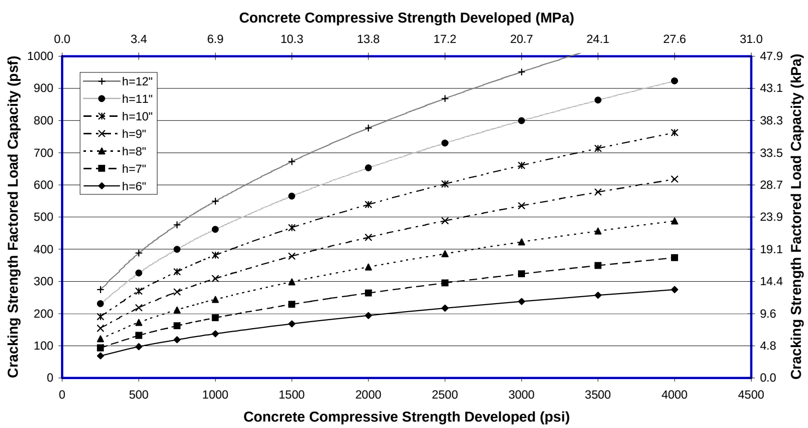


Figure 8.32 - Plain slab shore one way cracking strength factored load capacity (100x100 mm shore, 1.60 m primary beam, 1.6 m panel)

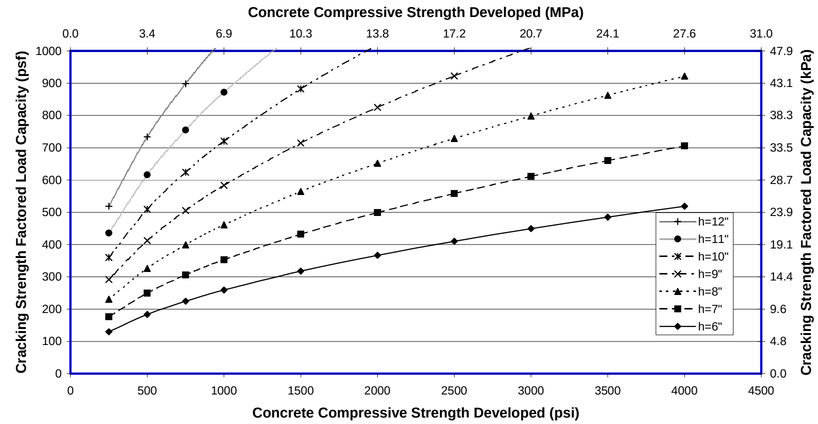


Figure 8.33 - Plain slab shore one way cracking strength factored load capacity (100x100 mm shore, 0.8 m primary beam, 1.6 m panel)

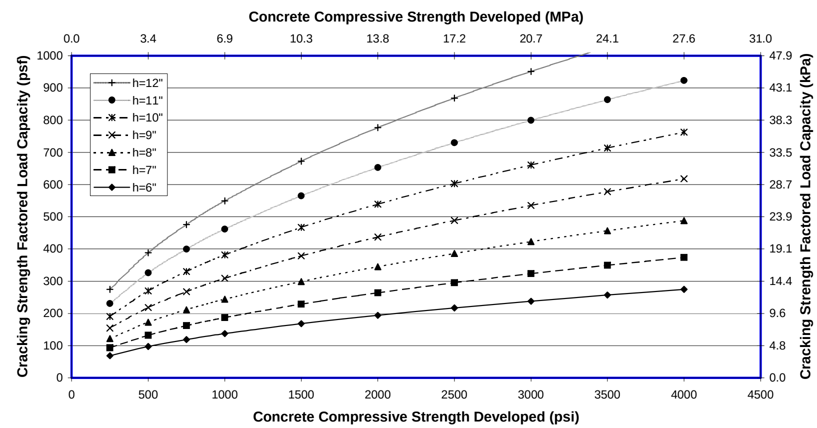


Figure 8.34 - Plain slab shore one way cracking strength factored load capacity (100x100 mm shore, 0.8 m primary beam, 1.6 m panel)

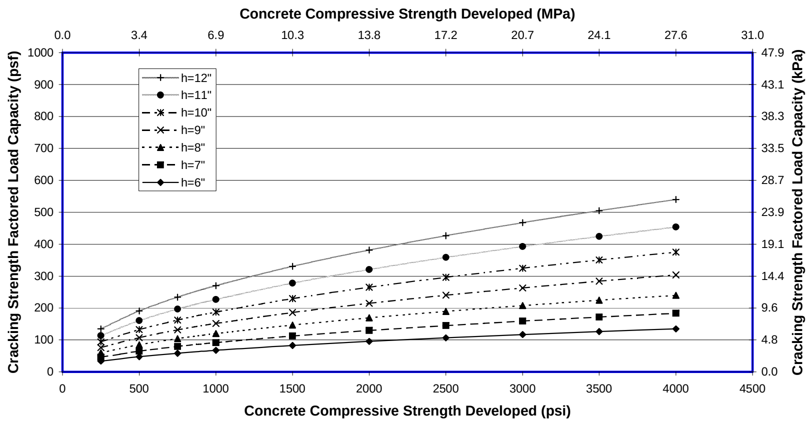


Figure 8.35 - Plain slab shore two way cracking strength factored load capacity (100x100 mm shore, 2.70 m primary beam, 1.6 m panel)

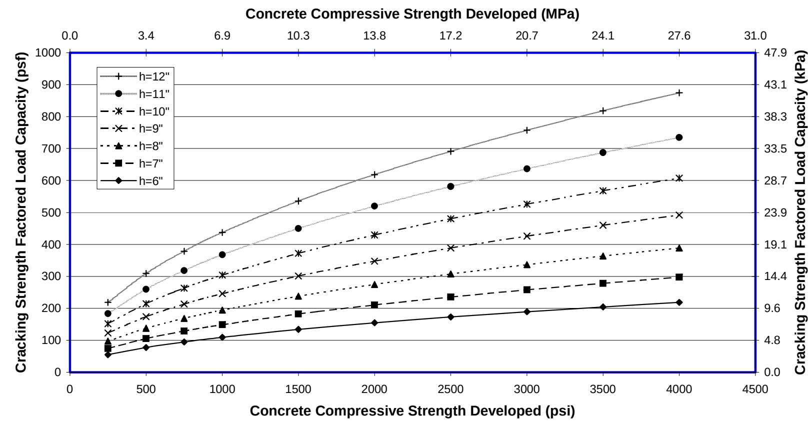


Figure 8.37 - Plain slab shore two way cracking strength factored load capacity (100x100 mm shore, 2.10 m primary beam, 1.6 m panel)

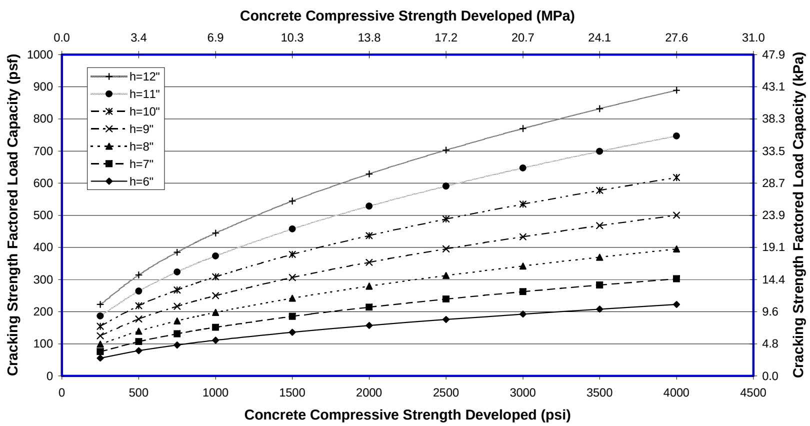


Figure 8.36 - Plain slab shore two way cracking strength factored load capacity (100x100 mm shore, 2.70 m primary beam, 1.6 m panel)

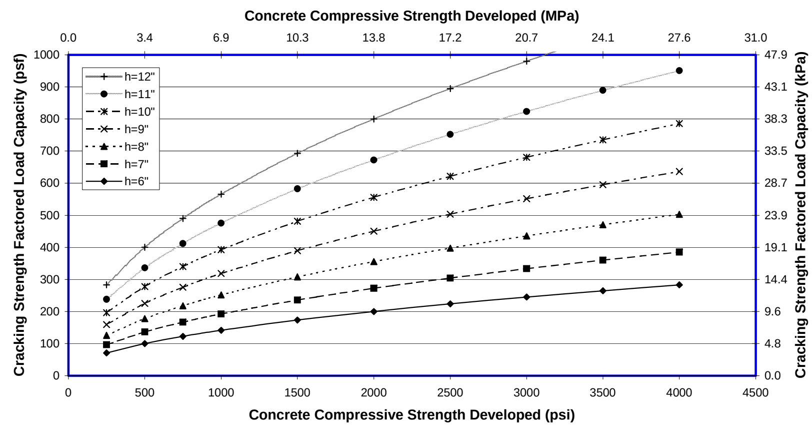


Figure 8.38 - Plain slab shore two way cracking strength factored load capacity (100x100 mm shore, 2.10 m primary beam, 1.6 m panel)

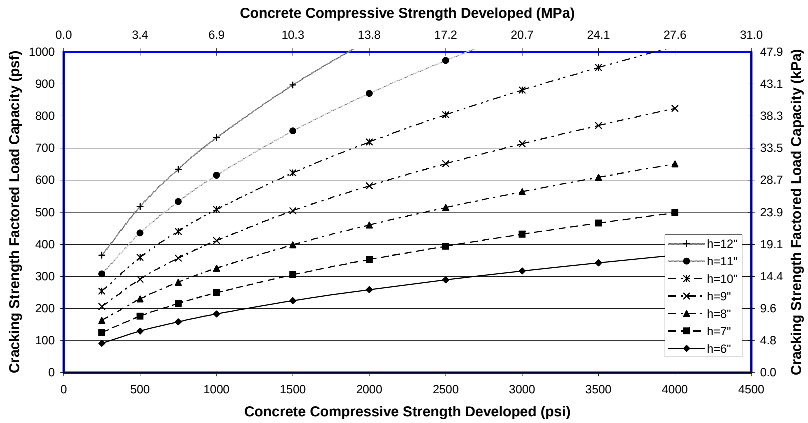


Figure 8.39 - Plain slab shore two way cracking strength factored load capacity (100x100 mm shore, 1.60 m primary beam, 1.6 m panel)

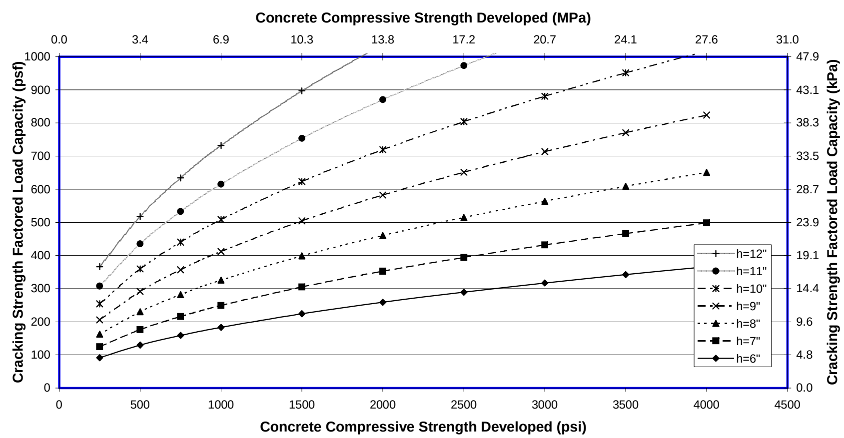


Figure 8.40 - Plain slab shore two way cracking strength factored load capacity (100x100 mm shore, 1.60 m primary beam, 1.6 m panel)

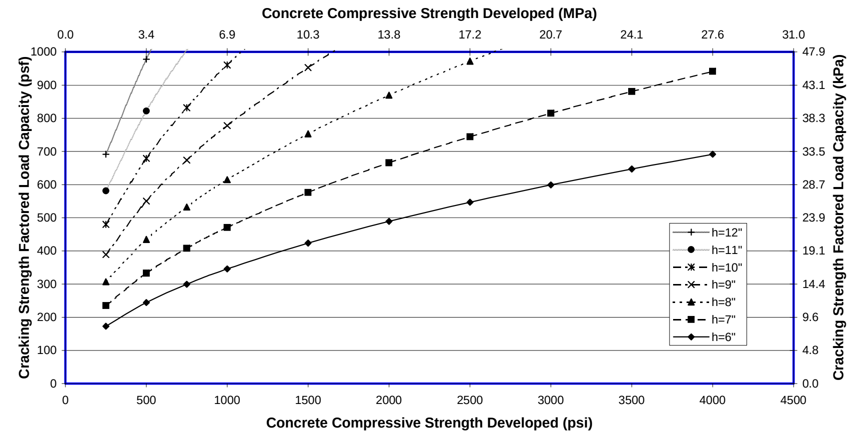


Figure 8.41 - Plain slab shore two way cracking strength factored load capacity (100x100 mm shore, 0.8 m primary beam, 1.6 m panel)

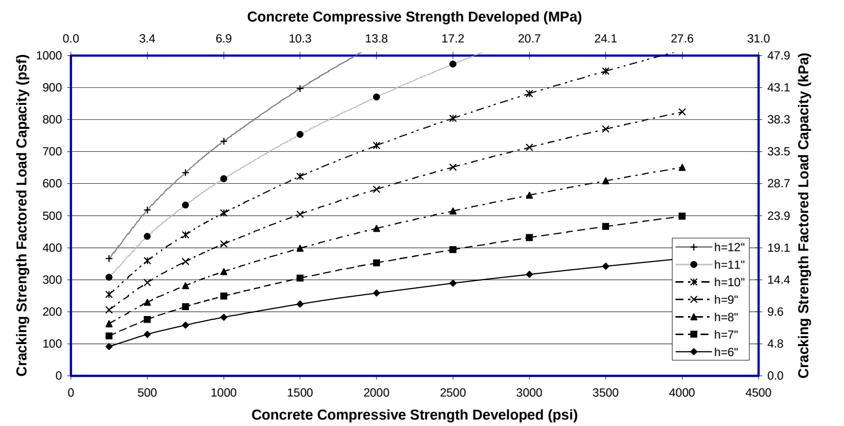


Figure 8.42 - Plain slab shore two way cracking strength factored load capacity (100x100 mm shore, 0.8 m primary beam, 1.6 m panel)

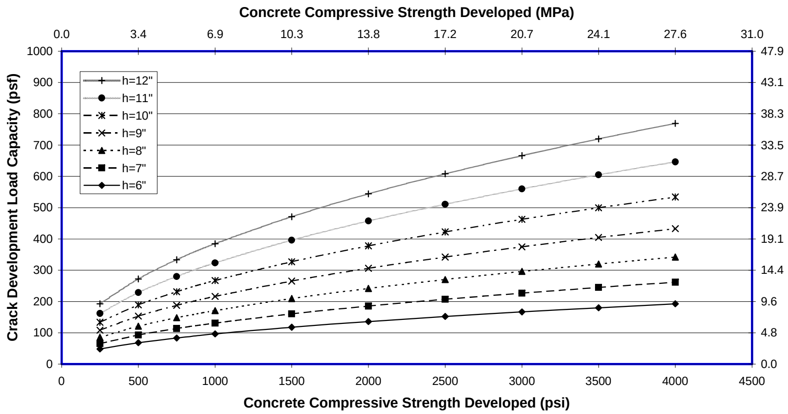


Figure 8.43- Plain slab shore one way crack development load capacity
(100x100 mm shore, 2.70 m primary beam, 1.6 m panel)

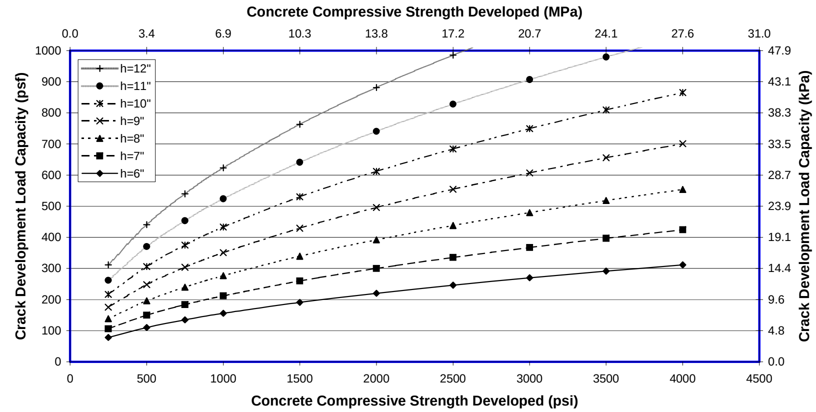


Figure 8.45 - Plain slab shore one way crack development load capacity
(100x100 mm shore, 2.10 m primary beam, 1.6 m panel)

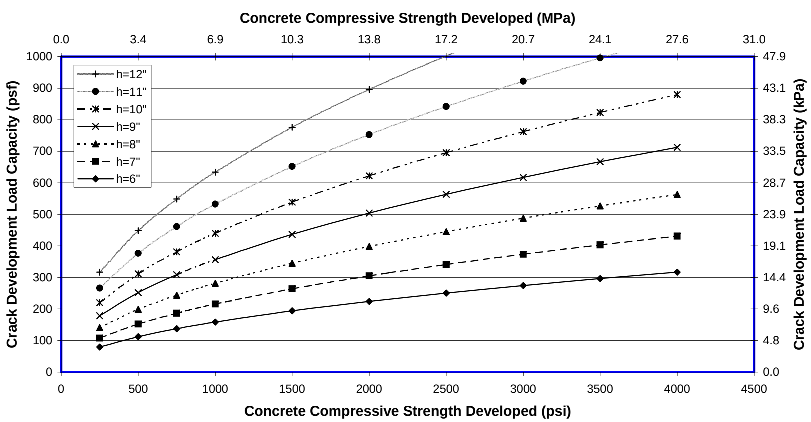
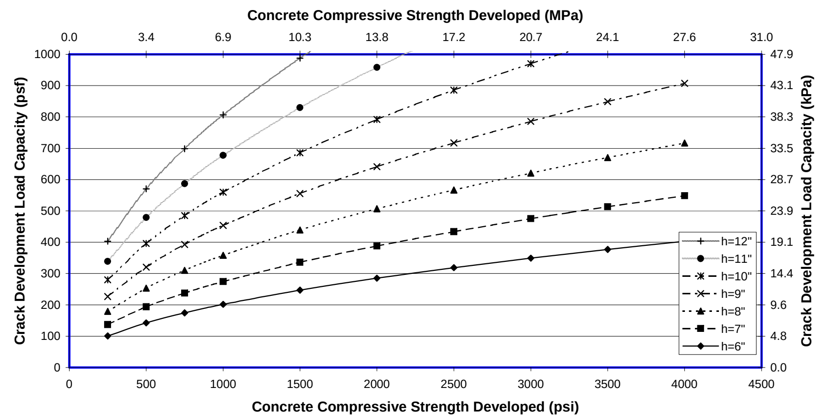


Figure 8.44 - Plain slab shore one way crack development load capacity
(100x100 mm shore, 2.70 m primary beam, 1.6 m panel)

Figure 8.46 - Plain slab shore one way crack development load capacity



(100x100 mm shore, 2.10 m primary beam, 1.6 m panel)

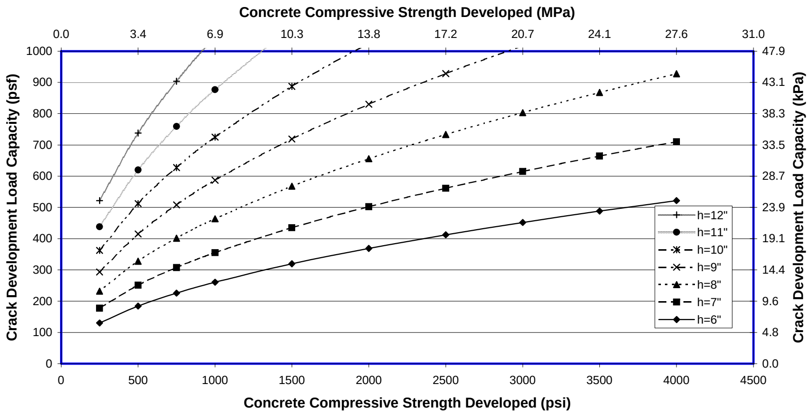


Figure 8.47 - Plain slab shore one way crack development load capacity (100x100 mm shore, 1.60 m primary beam, 1.6 m panel)

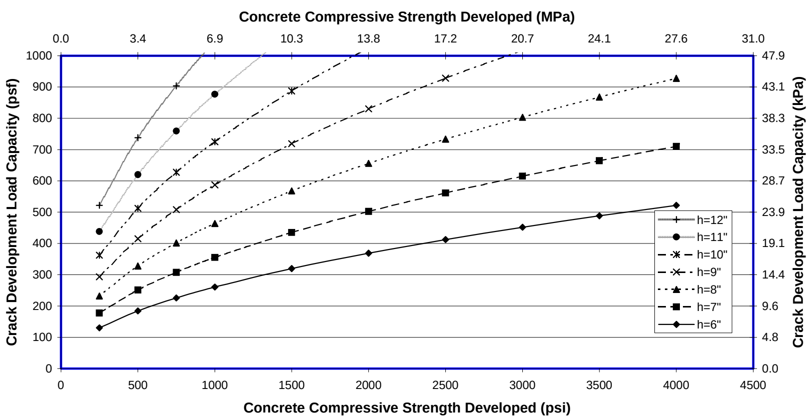


Figure 8.48 - Plain slab shore one way crack development load capacity (100x100 mm shore, 1.60 m primary beam, 1.6 m panel)

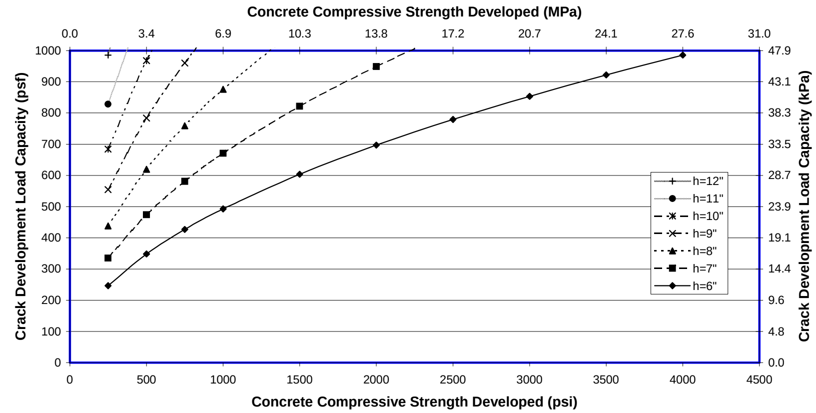


Figure 8.49 - Plain slab shore one way crack development load capacity (100x100 mm shore, 0.8 m primary beam, 1.6 m panel)

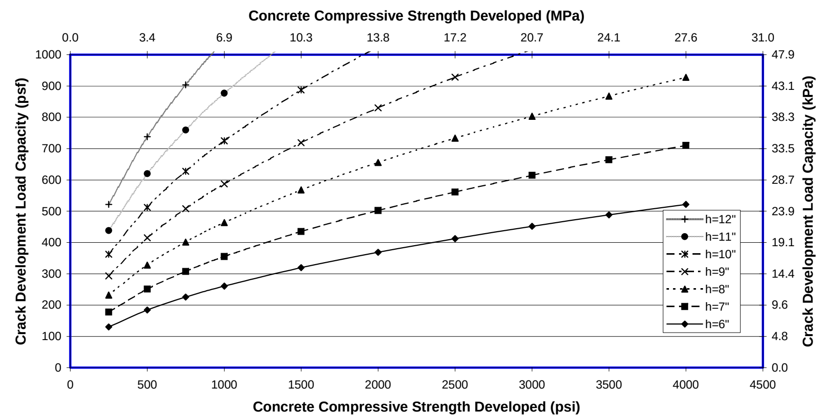


Figure 8.50 - Plain slab shore one way crack development load capacity (100x100 mm shore, 0.8 m primary beam, 1.6 m panel)

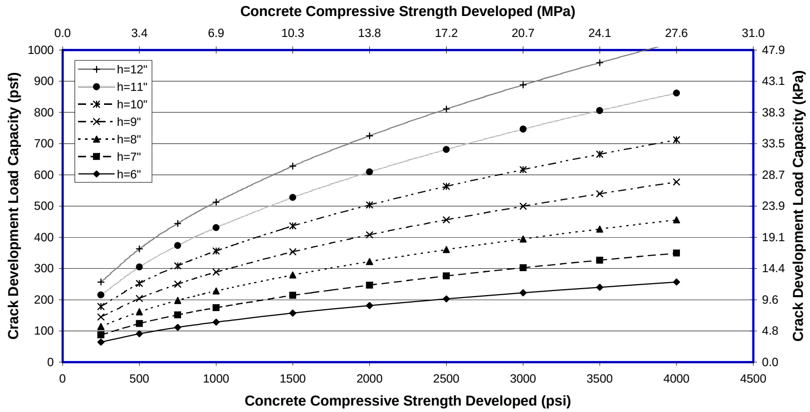


Figure 8.51 - Plain slab shore two way crack development load capacity
(100x100 mm shore, 2.70 m primary beam, 1.6 m panel)

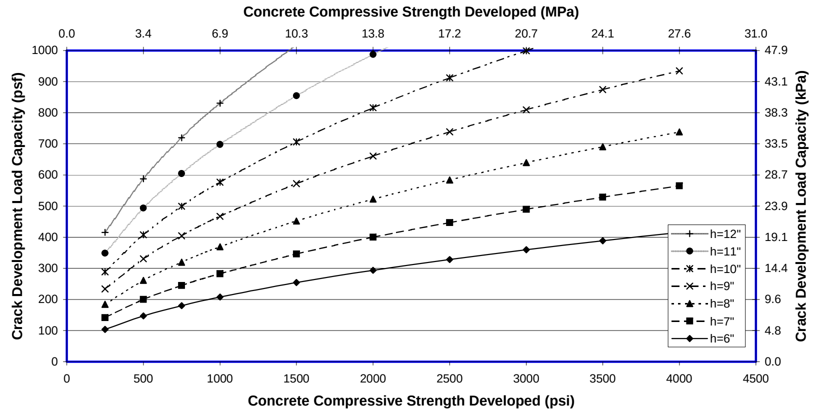


Figure 8.53 - Plain slab shore two way crack development load capacity
(100x100 mm shore, 2.10 m primary beam, 1.6 m panel)

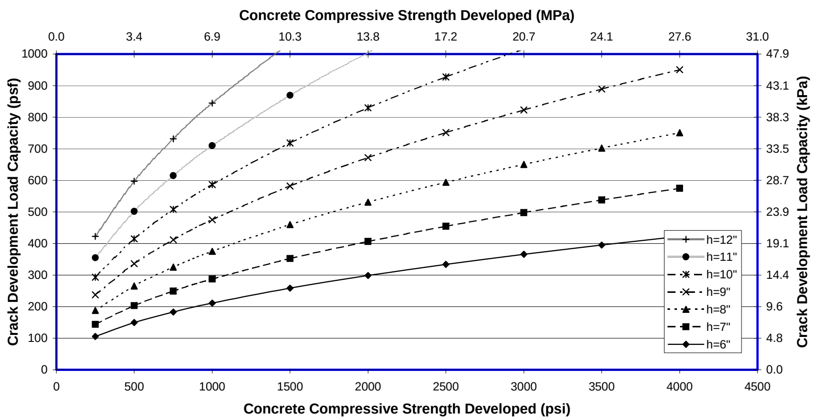


Figure 8.52 - Plain slab shore two way crack development load capacity
(100x100 mm shore, 2.70 m primary beam, 1.6 m panel)

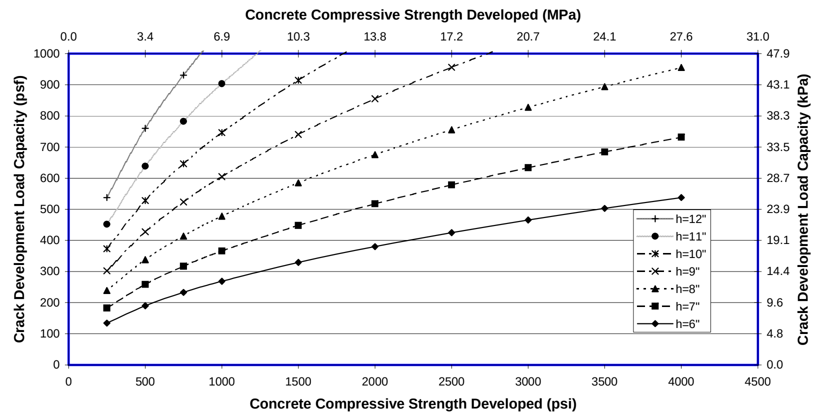


Figure 8.54 - Plain slab shore two way crack development load capacity
(100x100 mm shore, 2.10 m primary beam, 1.6 m panel)

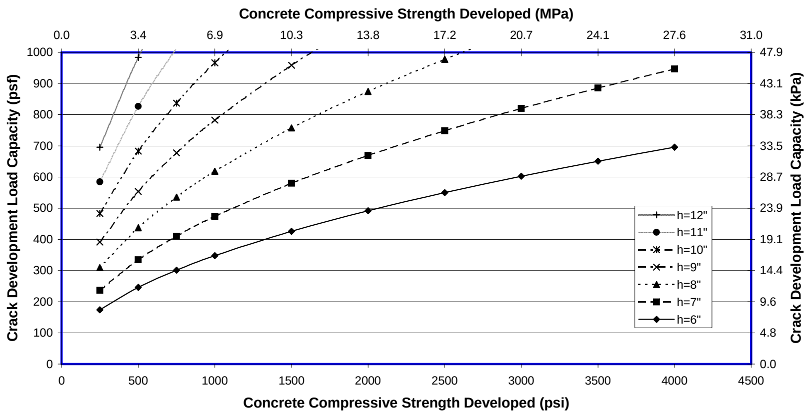


Figure 8.55 - Plain slab shore two way crack development load capacity
(100x100 mm shore, 1.60 m primary beam, 1.6 m panel)

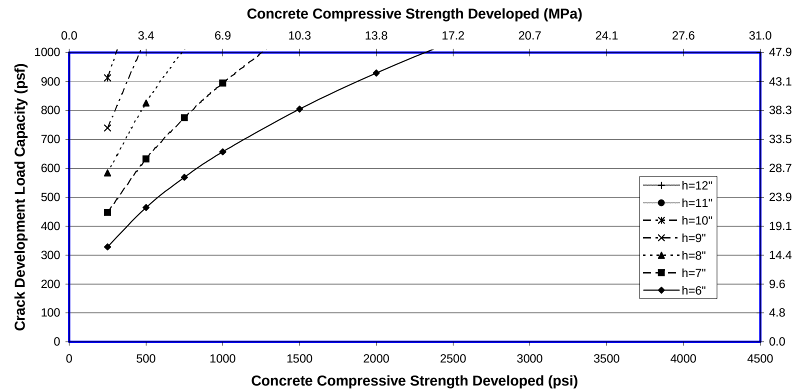


Figure 8.57 - Plain slab shore two way crack development load capacity
(100x100 mm shore, 0.8 m primary beam, 1.6 m panel)

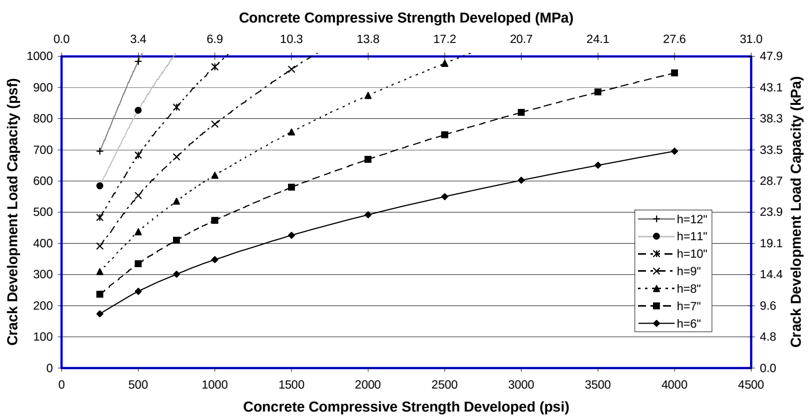


Figure 8.56 - Plain slab shore two way crack development load capacity
(100x100 mm shore, 1.60 m primary beam, 1.6 m panel)

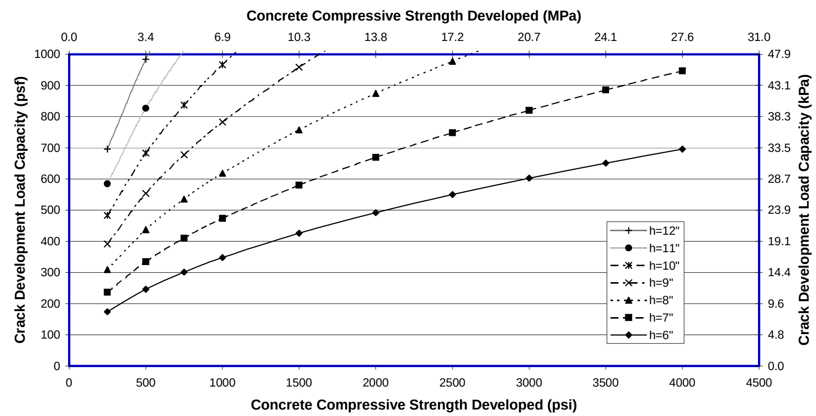


Figure 8.58 - Plain slab shore two way crack development load capacity
(100x100 mm shore, 0.8 m primary beam, 1.6 m panel)

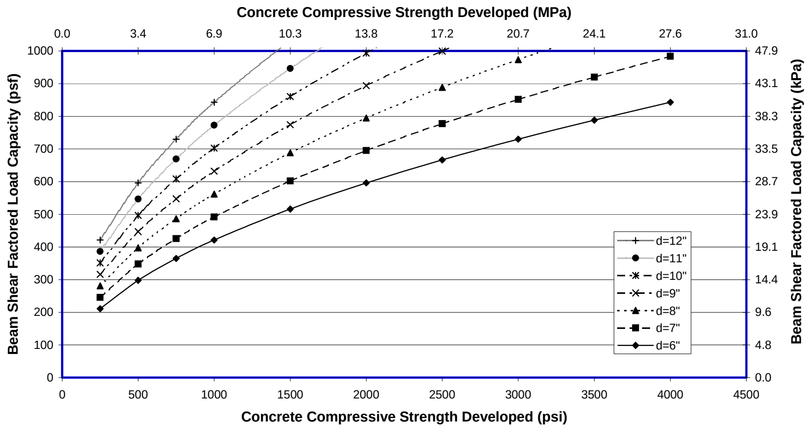


Figure 8.59 – Reinforced slab shore beam shear design strength load capacity (100x100 mm shore, 2.7 m primary beam, 1.6 m panel)

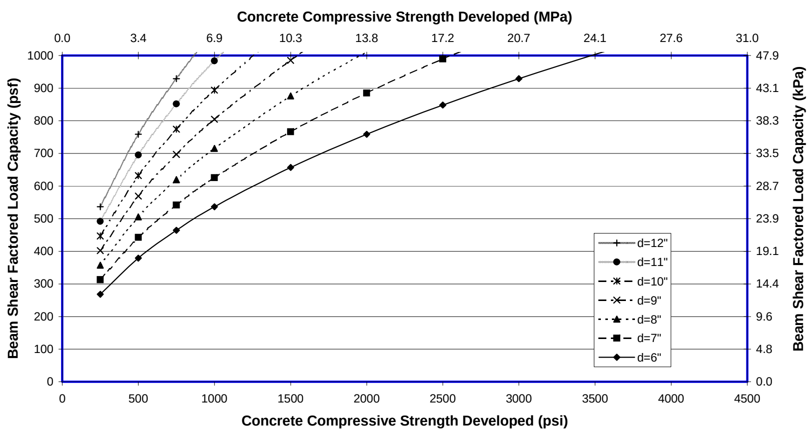


Figure 8.60 – Reinforced slab shore beam shear design strength load capacity (100x100 mm shore, 2.1 m primary beam, 1.6 m panel)

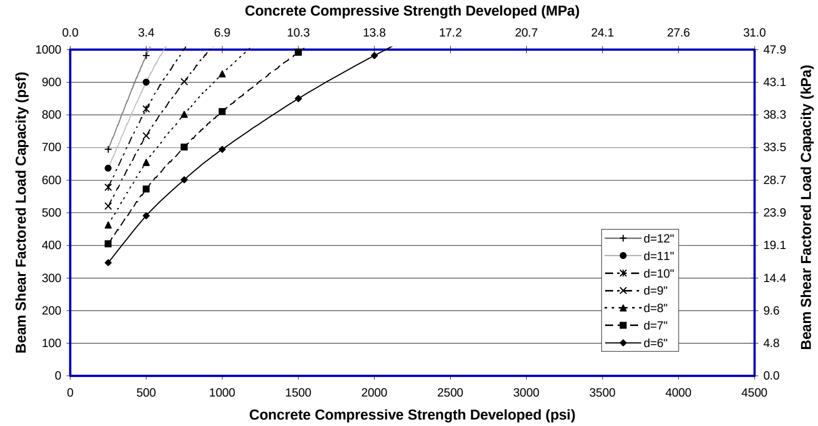
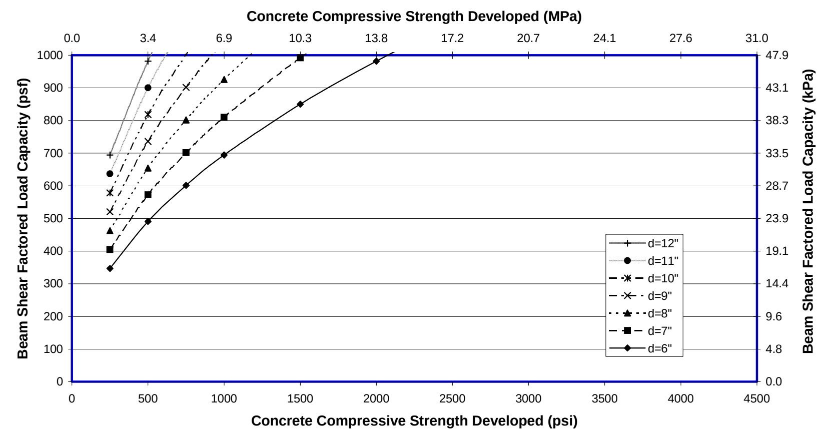


Figure 8.61 – Reinforced slab shore beam shear design strength load capacity (100x100 mm shore, 1.6 m primary beam, 1.6 m panel)

Figure 8.62 – Reinforced slab shore beam shear design strength load capacity

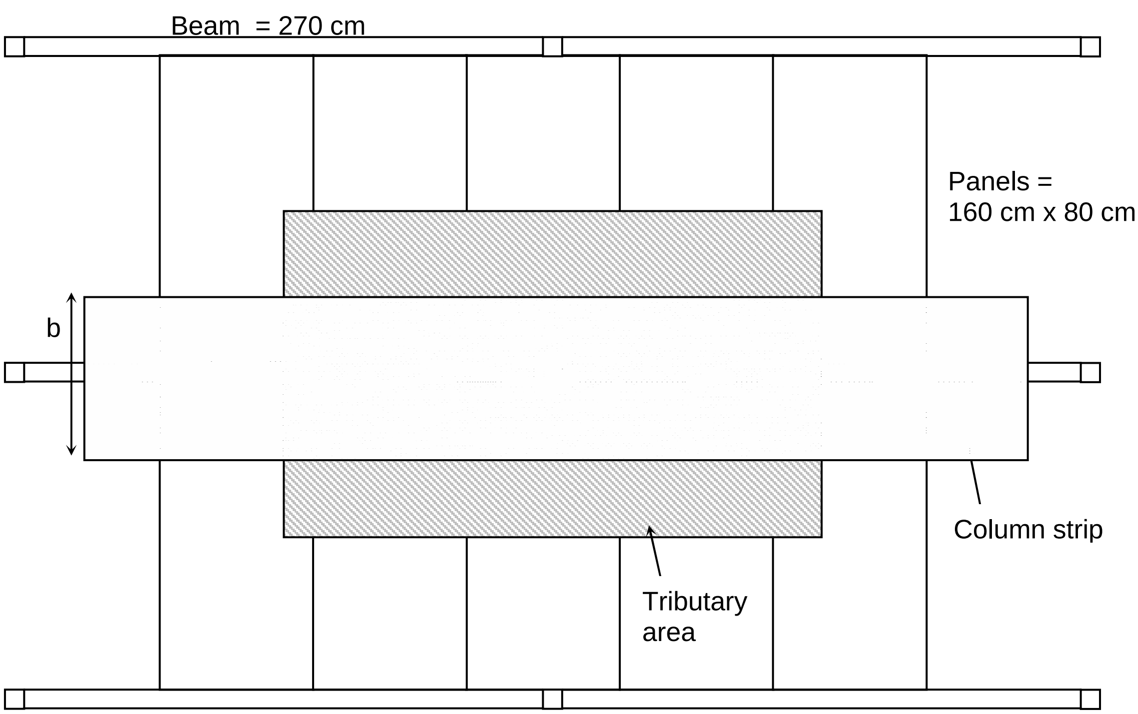


(100x100 mm shore, 0.8 m primary beam, 1.6 m panel)

8.3 Verification of Calculations

The purpose of this appendix is to verify the results plotted in the load capacity charts. An example of each of the major types of analysis is presented and compared to the results in the charts of Chapter 5. The example slabs are assumed to have a thickness, t, of 7.5” and a concrete strength of 1500 psi at the time the load application is contemplated.

h = 7.5”	d = 6”	w = 10 cm	b = 85 cm
f'_c = 1500 lbs/in^2	f_y = 60,000 lbs/in^2		
ϕ = 0.85 (for shear)	ϕ = 0.9 (for flexure)	β = 160 cm/160 cm = 1.0	



Tributary Area = (270 cm + 10 cm)*(160 cm + 10 cm) = 47600 cm^2

47600 cm^2 * (1 m/100 cm)^2 * (3.28 ft/1 m)2 = 51.24 ft^2

8.3.1 Punching Shear : Reinforced

$V_u < \phi V_n$

$\phi V_n = \phi*4*(f'_c{}^{0.5})*[4*(w+d)*d]$

$\phi V_n = 0.85*4*(1500^{0.5}$ lbs/in^2)*[4*((10cm*3.28*12/100)+6")*6"]

$\phi V_n =$ 31,401.18 lbs

Dividing by the Critical Area: (31,401 lbs / 51.24 ft^2)

Results in an Ultimate Punching Shear Load Capacity of 612.87 lbs/ft^2

8.3.2 Punching Shear : Plain Concrete

$V_u < \phi V_n$

$\phi V_n = \phi*[4/3 + 8/3\beta]*(f'_c{}^{0.5})*[4*(w+h)*h]$

but not greater than $2.66*(f'_c{}^{0.5})*[4*(w+h)*h]$

but not greater than $2.66*(1500^{0.5}$ lbs/in^2)*[4*((10cm*3.28*12/100)+7.5")*7.5"]

but not greater than 30,045.53 lbs

ϕV_n = 0.85*[4/3 + 8/(3∗1.0)]*(1500$^{0.5}$ lbs/in^2)*[4*((10cm*3.28*12/100)+7.5")*7.5"]

ϕV_n = 45,181.25 lbs (this is larger than 30,045 lbs)

Therefore,

ϕV_n = 30,045.53 lbs

Dividing by the Critical Area: (30,045 lbs / 51.24 ft^2)

Results in an Ultimate Punching Shear Load Capacity of 586.41 lbs/ft^2

8.3.3 Flexure : Reinforced : One Way : Parallel to Beam

$M_u < \phi M_n$

$\phi M_n = \phi[A_s f_y(d-a/2)]$

where $a = A_s f_y / 0.85 f'_c b$

where $A_s = A_{s,min} = (3*(f'_c{}^{0.5})/(f_y))*b_w*d$

but not less than $200*b_w*d/f_y$

but not less than 200*(85cm*3.28*12/100)*6" / 60,000 lbs/in^2

but not less than 0.669 in^2

$A_s = A_{s,min}$ = (3*(1500$^{0.5}$ lbs/in^2)/(60,000 lbs/in^2))* (85cm*3.28*12/100)*6"

$A_s = A_{s,min}$ = 0.3889 in^2

Therefore,

$A_s = A_{s,min}$ = 0.669 in^2

a = (0.669 in^2)*(60,000 lbs/in^2) / 0.85*(1500 lbs/in^2)*(85cm*3.28*12/100)

a = 0.94"

ϕM_n = 0.9*[0.669 in^2*(60,000 lbs/in^2)*(6" - 0.94"/2)]

ϕM_n = 199,791 in.lbs

M_u < 199,791 in.lbs

M_u < (lbs/ft^3)*l^2/8 < 199,791 in.lbs

M_u < (lbs/ft^3) < (199,791 in.lbs)*(1 ft/12 in)*(8)/[(270 cm + 10 cm)*(1 m/100 cm)*(3.28 ft/1 m)]^2

= 1578 lbs/ft

M_u < (1578 lbs/ft) / [(160 cm + 10 cm)*(1 m/100 cm)*(3.28 ft/1 m)] = 283 lbs/ft^2

Therefore,

The Ultimate Load Capacity based on Flexure is 283 lbs/ft^2

8.3.4 Flexure : Reinforced : One Way : Perpendicular to Beam

$M_u < \phi M_n$

$\phi M_n = \phi[A_s f_y(d-a/2)]$

where $a = A_s f_y / 0.85 f'_c b$

where $A_s = A_{s,min} = (3*(f'_c{}^{0.5})/(f_y))*b_w*d$

but not less than $200*b_w*d/f_y$

but not less than 200*(85cm*3.28*12/100)*6" / 60,000 lbs/in^2

but not less than 0.669 in^2

$A_s = A_{s,min} = (3*(1500^{0.5}\ lbs/in^2)/(\ 60{,}000\ lbs/in^2))*\ (85cm*3.28*12/100)*6"$

$A_s = A_{s,min} = 0.3889\ in^2$

Therefore,

$A_s = A_{s,min} = 0.669\ in^2$

$a = (0.669\ in^2)*(60{,}000\ lbs/in^2)\ /\ 0.85*(1500\ lbs/in^2)*(85cm*3.28*12/100)$

$a = 0.94"$

$\phi M_n = 0.9*[0.669\ in^2*(60{,}000\ lbs/in^2)*(6" - 0.94"/2)]$

$\phi M_n = 199{,}791$ in.lbs

$M_u < 199{,}791$ in.lbs

$M_u < (lbs/ft^3)*l^2/8 < 199{,}791$ in.lbs

$M_u < (lbs/ft^3) < (199{,}791\ in.lbs)*(1\ ft/12\ in)*(8)/[(160\ cm + 10\ cm)*(1\ m/100\ cm)*(3.28\ ft/1\ m)]^2$

= 4282 lbs/ft

$M_u < (4282\ lbs/ft)\ /\ [(270\ cm + 10\ cm)*(1\ m/100\ cm)*(3.28\ ft/1\ m)] = 466\ lbs/ft^2$

Therefore,

The Ultimate Load Capacity based on Flexure is 466 lbs/ft^2

8.3.5 Flexure : Reinforced : Two Way : Parallel to Beam

For two-way action, it is assumed that the moment is distributed in both directions throughout the slab. Because of this, a factor can be applied to the one-way case to increase the flexural capacity of the section. This factor is 75%

$M_u < (283\ lbs/ft^2)\ /\ 0.75 = 378\ lbs/ft^2$

Therefore,

The Ultimate Load Capacity based on Flexure is 378 lbs/ft^2

8.3.6 Flexure : Reinforced : Two Way : Perpendicular to Beam

For two-way action, it is assumed that the moment is distributed in both directions throughout the slab. Because of this, a factor can be applied to the one-way case to increase the flexural capacity of the section. This factor is 75%

$M_u < (466\ \text{lbs/ft}^2) / 0.75 = 622\ \text{lbs/ft}^2$

Therefore,

The Ultimate Load Capacity based on Flexure is 622 lbs/ft^2

8.3.7 Flexure : Plain Concrete : One Way : Parallel to Beam

$M_u < \phi M_n$

$\phi M_n = \phi[5*f'_c{}^{0.5}*S]$

$S = bh^2/6 = [(85\ \text{cm})*(1\ \text{m}/100\ \text{cm})*(3.28\ \text{ft}/1\ \text{m})]*(7.5")^2/6 = 313\ \text{in}^3$

$\phi M_n = 0.9*[5*(1500^{0.5})*313\ \text{in}^3] = 60{,}753.61\ \text{in.lbs}$

$M_u < 60{,}753\ \text{in.lbs}$

$M_u < (\text{lbs/ft}^3)*l^2/10 < 60{,}753\ \text{in.lbs}$

$M_u < (\text{lbs/ft}^3) < (60{,}753\ \text{in.lbs})*(1\ \text{ft}/12\ \text{in})*(10)/[(270\ \text{cm} + 10\ \text{cm})*(1\ \text{m}/100\ \text{cm})*(3.28\ \text{ft}/1\ \text{m})]^2$

$= 600\ \text{lbs/ft}$

$M_u < (600\ \text{lbs/ft}) / [(160\ \text{cm} + 10\ \text{cm})*(1\ \text{m}/100\ \text{cm})*(3.28\ \text{ft}/1\ \text{m})] = 107.5\ \text{lbs/ft}^2$

Therefore,

The Ultimate Load Capacity based on Flexure is 107.5 lbs/ft^2

8.3.8 Flexure : Plain Concrete : One Way : Perpendicular to Beam

$M_u < \phi M_n$

$\phi M_n = \phi[5 * f'^{0.5}_c * S]$

$S = bh^2/6 = [(85\ cm)*(1\ m/100\ cm)*(3.28\ ft/1\ m)]*(7.5")^2/6 = 313\ in^3$

$\phi M_n = 0.9*[5*(1500^{0.5})*313\ in^3] = 60{,}753.61\ in.lbs$

$M_u < 60{,}753\ in.lbs$

$M_u < (lbs/ft^3)*l^2/10 < 60{,}753\ in.lbs$

$M_u < (lbs/ft^3) < (60{,}753\ in.lbs)*(1\ ft/12\ in)*(10)/[(160\ cm + 10\ cm)*(1\ m/100\ cm)*(3.28\ ft/1\ m)]^2$

$= 1628\ lbs/ft$

$M_u < (1628\ lbs/ft) / [(270\ cm + 10\ cm)*(1\ m/100\ cm)*(3.28\ ft/1\ m)] = 177\ lbs/ft^2$

Therefore,

The Ultimate Load Capacity based on Flexure is 177 lbs/ft^2

8.3.9 Flexure : Plain Concrete : Two Way : Parallel to Beam

For two-way action, it is assumed that the moment is distributed in both directions throughout the slab. Because of this, a factor can be applied to the one-way case to increase the flexural capacity of the section. This factor is 75%

$M_u < (108\ lbs/ft^2) / 0.75 = 143\ lbs/ft^2$

Therefore,

The Ultimate Load Capacity based on Flexure is 143 lbs/ft^2

8.3.10 Flexure : Plain Concrete : Two Way : Perpendicular to Beam

For two-way action, it is assumed that the moment is distributed in both directions throughout the slab. Because of this, a factor can be applied to the one-way case to increase the flexural capacity of the section. This factor is 75%

$M_u < (177\ \text{lbs/ft}^2) / 0.75 = 236\ \text{lbs/ft}^2$

Therefore,

The Ultimate Load Capacity based on Flexure is 236 lbs/ft^2

8.3.11 Beam Shear

$V_u < \phi V_n$

$\phi V_n = \phi * 2 * (f'_c{}^{0.5})bd$

$\phi V_n = 0.85 * 2 * (1500^{0.5}\ \text{lbs/in}^2) * [(85\ \text{cm} * 3.28 * 12/100) * 6"]$

$\phi V_n = 13{,}219{,}97\ \text{lbs}$

This is only ½ of the actual load being resisted (assuming a simply supported beam)

Therefore, it must be multiplied by 2 before dividing by the critical area

$(13{,}220\ \text{lbs} / 51.24\ \text{ft}^2) * 2$

Results in an Ultimate Punching Shear Load Capacity of 530 lbs/ft^2