

ELEVATED SLABS



Compilation 21

American Concrete Institute

Elevated Slabs

ACI Compilation 21

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Preface

ACI Compilations combine material previously published in Institute periodicals to provide compact and ready reference on specific topics. The Material in a compilation does not necessarily represent the opinion of an ACI technical committee — only the opinions of the individual authors. However, the information presented here is considered to be a valuable resource for readers interested in the subject.

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Chairman, ACI Committee 347

Formwork for Concrete

Hershell Gill

Chairman, ACI-ASCE Committee 421

Design of Reinforced Concrete Slabs

On the cover: A floor cycle of one month was achieved by contractors involved in the erection of a new addition to Toronto's skyline. The headquarters building for the Canadian Broadcasting Corp. is the first phase of a planned development that will later include twin commercial office towers of 28 and 50 stories and a residential tower with several retail levels.



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Point of View

Tolerance Conflicts and Omissions in Suspended Slab Construction

by Eldon Tipping

The goal of suspended slab construction is to produce deflected surfaces that are relatively flat, level, and at the proper elevation. Achieving this goal has presented its share of challenges to the designer and contractor over the years. This article discusses the effect of construction tolerances on conventionally reinforced cast-in-place concrete and concrete slabs on metal deck; post-tensioned and precast structures are not included.

The first section examines selected accepted tolerances for formwork elevation, reinforcing steel fabrication and placement, beam and slab thickness, and floor surface elevation. Tolerances for structural steel fabrication and erection are also addressed to illustrate the large degree to which tolerances for these different kinds of work can conflict with the stated goal.

The second section identifies and discusses tolerance areas that are commonly overlooked in contract documents. An awareness of these will allow the designer and specifier to address those conflicts and omissions as they deem appropriate.

Accepted tolerances and potential conflicts

The American Concrete Institute provides the accepted USA standard for tolerances in concrete construction. Its publication, "Standard Specifications for Tolerances for Concrete Construction and Materials (ACI 117-90)," is intended to be used in its entirety by reference in project specifications.

Tolerances that affect cast-in-place concrete floor construction are those that address reinforcing steel (fabrication and placement), forms (elevation and level alignment), concrete cross-sectional dimensions, and concrete surface (elevation and level alignment). Fig. 1 shows major tolerances governing beam and slab construction.

ACI 117 contains a Specification Checklist that is not part of the standard, but is intended to assist the specifier in "properly choosing and specifying the necessary mandatory and optional requirements for the Project Specification." One admonition of the Checklist states: "Where a specific application uses multiply (sic) toleranced items that together yield a toleranced result, the specifier must analyze the tolerance envelope with

respect to practical limits and design assumptions and specify its value where the standard tolerances (sic) values in this specification are inadequate or inappropriate."

The accepted USA standard for fabrication and erection of structural steel is the "Code of Standard Practice for Steel Buildings and Bridges," published by the American Institute of Steel Construction (AISC) in its *Manual of Steel Construction*. The most recent edition of this document was adopted effective September 1, 1986.

Tolerances that affect construction of concrete floors on metal deck are those that address structural steel (fabrication and erection), concrete cross-sectional dimensions, and concrete surface (elevation and level alignment). Tolerances for structural steel fabrication and erection are contained in the AISC Code (Fig. 2 and 3); the remaining tolerances are taken from ACI 117-90.

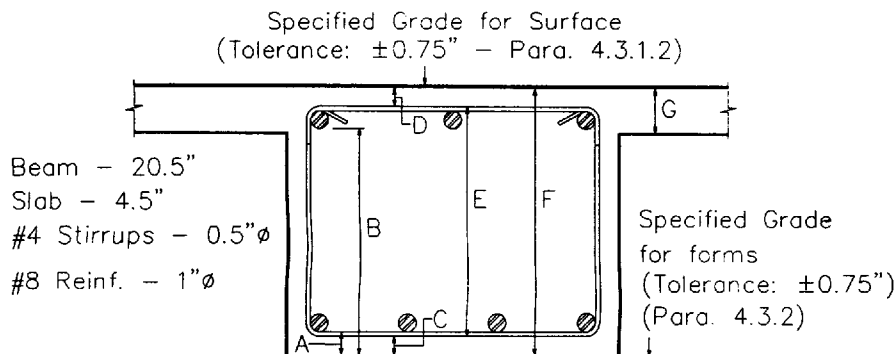
"Standard Mill Practice — General Information" in the AISC *Manual of Steel Construction* contains information pertaining to camber of structural steel beams (Fig. 4), and also contains the following statements about cambering of structural steel beams:

Mill camber in beams of less depth than tabulated [in a table provided on page 1-167] should not be specified.

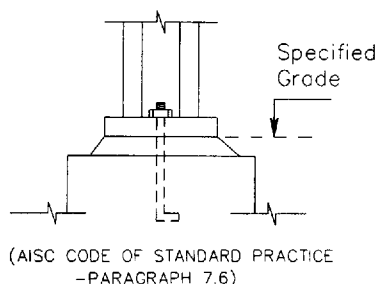
Camber is measured at the mill and will not necessarily be present in the same amount in the section of beam as received due to release of stress induced during the cambering operation. In general, 75 percent of the specified camber is likely to remain.

When evaluating tolerances associated with cast-in-place concrete construction one must keep in mind the

POINT OF VIEW This article was selected for reader interest by the editors. However, the opinions expressed by the author are not necessarily those of the American Concrete Institute. The editors invite comments from our readers about the personal views given in this article.



	Specified Dimension	Tolerance		Range		ACI 117 Para.
		Minus	Plus	Low End	High End	
A	2.0"	.25"	.5"	1.75"	2.5"	2.2.3
B	17.5"	.5"	.5"	17.0"	18.0"	2.2.2
C	1.5"	.25"	-	1.25"	-	2.2.3
D	1.5"	.5"	-	1.0"	-	2.2.3
E	17.5"	.5"	.5"	17.0"	18.0"	2.1
F	20.5"	.375"	.5"	20.125"	21.0"	4.4
G	4.5"	.25"	.375"	4.25"	4.875"	4.4

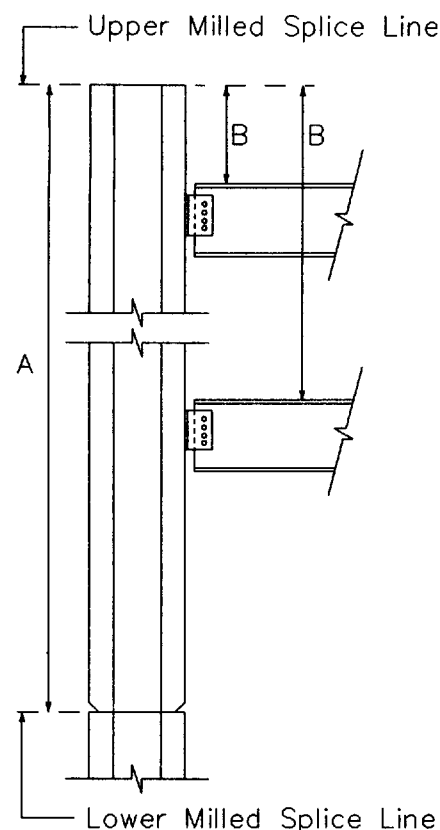


Elevation tolerance relative to established grade is $\pm 0.125"$

above: Fig. 1 — Major ACI tolerances governing beam and slab construction.

right: Fig. 2 — AISC elevation tolerance for column base plates.

left: Fig. 3 — AISC tolerances for column length and connection location.



(AISC CODE OF STANDARD PRACTICE)

ITEM	TOLERANCE	PARAGRAPH
A	$\pm 0.03125"$	6.4.1
B	$+0.1875"$ $-0.3125"$	7.11.3.2.(b)

admonition in Paragraph 1.2.3 of ACI 117 that tolerances are not cumulative.

Examples 1 and 2 below consider cast-in-place concrete construction and show the interaction of tolerances for reinforcing steel fabrication, clear distance (effective depth), concrete cover, and member thickness. Example 3 shows further inconsistencies that must be considered when one attempts to meld tolerance requirements from the structural steel industry with those for concrete construction.

The paragraph references in the three examples are from References 1, 2, and 3 as noted.

Example 1

Given: Beam with a specified depth of 20.50 in. and adjacent slab with a specified depth of 4.50 in. The void adjacent to the beam and below the slab is created by using a 16.00 in. deep prefabricated form. Beam stirrups are fabricated using #4 (0.50 in. diameter) bars. Beam top and bottom main reinforcement is #8 (1.00 in. diameter) bars.

Beam stirrups are fabricated with a vertical dimension 0.50 in. longer or shorter than specified.

Discussion:

- The envelope within which reinforcing steel, clear distance, and cover requirements must coexist is defined by the allowable variation in member thickness (Reference 1: 1.2.3 and 4.4.1). When the slab is formed by a void of constant depth that is supported at the same elevation as the beam bottom, distance from the slab surface to the beam bottom is limited by the allowable variation in the slab thickness from that specified (Reference 1: 4.4.1). In this instance that envelope, measured from the beam bottom, extends from 20.25 to 20.875 in. (a slab thickness of 4.25 to 4.875 in.).
- For stirrups with a length less than 12 ft, the tolerance for the vertical dimension is .50 in. The stirrup in this example conforms to tolerances for fabrication (Reference 1: 2.1).
- Prefabricated supports of the proper height are normally used to support bottom bars in beams. The vari-

ation of bottom bar clear distance to forms and concrete cover between the stirrups and formed surface from that specified is likely to be insignificant. The bottom reinforcing steel meets clear distance and minimum cover requirements (Reference 1: 2.2.2 and 2.2.3).

- Clear distance of beam top reinforcing steel is established by depth of supports and length of stirrups. Since cover at the beam bottom is nearly exact, the clear distance of the top steel to the forms will vary with the stirrup length from -0.50 to $+0.50$ in. This variation conforms to allowable tolerances (Reference 1: 2.2.2).

- For stirrups that are 0.50 in. too short (deep), the effective cover at the top of the beam could vary from 1.75 to 2.375 in. and meet the member thickness tolerance (Fig. 6). Construction of the floor surface at proper grade presents no problem, since the only restriction on cover for top steel is a 0.50 in. reduction of that specified (Reference 1: 2.2.3 and 4.4.1).

- For stirrups that are 0.50 in. too long (deep), concrete cover at the top of the beam could vary from 0.75 to 1.375 in. if member thickness were the only consideration. Any cover below 1.00 in., however, will violate the minimum cover requirements for top reinforcing steel (Reference 1: 2.2.3) so the actual allowable variation in concrete cover is from 1.00 to 1.375 in. The resulting member thickness will be from a minimum of 20.50 to a maximum of 20.875 in.; only use of the minimum allowable cover will result in a member thickness that matches specified thickness (Fig. 6).

Example 2

Given: Beam and slab construction as stated in Example 1. Beam stirrups are fabricated with a vertical dimension 0.50 in. longer or shorter than specified.

Form elevation varies from specified grade by as much as 0.75 in.

Discussion:

- The envelope within which the supported structure must fit prior to removal of shores is defined by planes that are 0.75 in. above specified grade for the slab and 0.75 in. below specified grade for the beam soffit (Reference 1: 4.3.1.2 and 4.3.2). It should be noted, however, that forms that are more than 0.25 in. higher than specified grade in any location will force the slab surface above specified grade at that location to maintain minimum thickness requirements for both beam and slab (Fig. 7).

- For stirrups that are 0.50 in. too short (deep), the effective cover at the top of the beam could vary from 1.75 to 2.375 in. and meet the member thickness tolerance. Construction of the floor surface at proper grade and meeting all tolerance requirements presents no problem, as long as the supporting forms are no more than 0.25 in. higher than specified grade or more than 0.625 in. below specified grade (Fig. 6 and 7).

- For stirrups that are 0.50 in. too long (deep), member thickness can vary from a minimum of 20.50 to a maximum depth of 20.875 in. If supporting forms are above specified grade, the surface must also be above specified grade or the end result will be a member that violates the minimum thickness requirements (Fig. 6 and 7).

Example 3

Given: Concrete slab with a total specified thickness of 6 in. supported by 3 in. deep composite metal decking supported in turn by a grid of structural steel beams and girders. The structural steel is cambered in some locations as much as 2 in. at midspan and is designed to support the weight of the wet concrete during placement (unshored construction).

Top of structural steel (bottom of metal deck) is at the ideal elevation when beam-to-column connections are at specified grade and when the pre-placement midspan elevation of the steel is at specified grade plus the specified camber. In this example, top of steel elevations vary from the ideal elevation by as much as 1.00 in.

Discussion:

- When evaluating tolerance conformance within the framework of the AISC Code, the first factor that must be recognized by the specifier is that only base plates for column sections are tied to an established grade. All other tolerances are stated in terms relative to the fabricated member.

- The elevation tolerance for column base plates is 0.125 in. (Fig. 2).

- The length tolerance for each column section is 0.03125 in. (Fig. 3).

- The working point for beams and girders framing into columns may be too high by 0.3125 in. or too low by 0.1825 in. from the desired location measured from the splice line of the column section (Fig. 3).

- The top edge of adjustable curb angles and lintels are acceptable if they are within 0.375 in. of the proper distance from the upper milled splice point of the nearest column to which the steel beam is attached (Reference 2: 7.11.3.3).

- There is no specified tolerance for items that are attached directly to steel beams and girders (Fig. 5), other than statements that variations in overall dimensions are acceptable if within the cumulative total permitted for rolling, fabricating and erection (Reference 2: 7.11.1 and 7.11.3.2(c)).

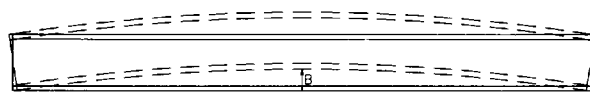
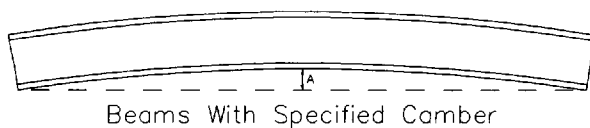
- The tolerance for location of lintels, sills, etc., are not the same in the AISC Code and ACI 117 (References 2: 7.11.3.3; and 1: 4.3.2). The AISC Code references location relative to upper splice elevation of the nearest column; ACI 117 references tolerances to specified grade.

- Initial camber in structural steel beams and girders may not match that required by the drawings (Reference 3: p. 1-167).

- Mill camber should not be specified for steel beams that are 12 in. or less in depth, and those that are expected by the engineer to deflect less than $\frac{3}{4}$ in. under load of the concrete (Reference 3: p. 1-167).

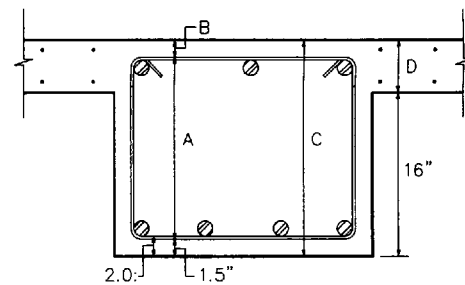
- Beams, girders, and trusses with no specified camber may have a natural camber as a result of rolling or fabrication (Fig. 4). These elements are to be fabricated so that any such camber is upward (Reference 2: 6.4.4).

- Variations in supporting steel platform prior to concreting operations dictate that concrete be placed to a uniform thickness over steel beams to meet slab cross-sectional tolerances (Reference 1: 4.4.1).



(AISC STANDARD MILL PRACTICE)

ITEM	BEAM LENGTH	TOLERANCE	
		PLUS	MINUS
A	50' & Less	0.5"	0.0"
A	Over 50'	0.5" + 0.125"/10'	0.0"
B	All	0.125"/10'	0.0"



Beam - 20.5"
Slab - 4.5"

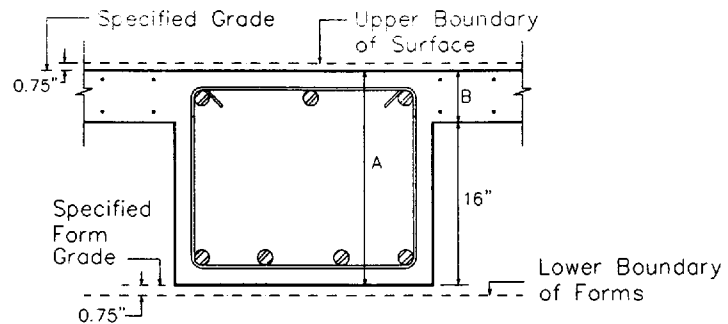
	17"	18"
A	1.75" to 2.375"	1.00" to 1.375"
B	20.25" to 20.875"	20.50" to 20.875"
C	4.25" to 4.875"	4.50" to 4.875"

above: Fig. 4 — AISC tolerances for camber of beams.

below: Fig. 5 — There is no specified tolerance for items attached directly to steel beams and girders.

above right: Fig. 6 — Dimensions for Example 1.

right: Fig. 7 — Dimensions for Example 2.



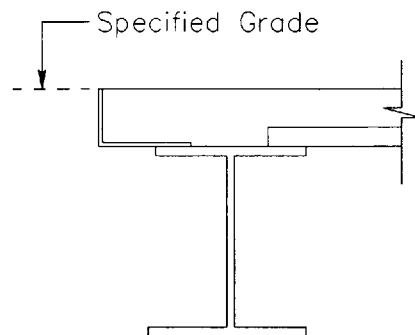
Required Top Surface Elevation (relative to specified grade)
When Stirrup Depth and Form Elevation Vary Within Tolerance Limits

Stirrup Depth	Form Elevation Relative to Specified Grade		
	-0.75"	0.0"	+0.75"
17"	-1.00" to -0.375"	-0.25" to +0.375"	+0.50" to +1.125"
17.5"	-1.00" to -0.375"	-0.25" to -0.375"	+0.50" to +1.125"
18"(b)	-0.75" to -0.375"	0.00" to -0.375"	+0.75" to +1.125"

(a) Given: Beam Depth 20.5" (-0.25" to +0.375")
Slab Thickness 4.5" (-0.25" to +0.375")

Assume all other tolerances are met.

(b) Beam depth and slab thickness cannot be less than specified if cover tolerance requirements are met.



- There is no tolerance on variation of the surface elevation, because the supporting steel framework is not supported during placement of concrete (Reference 1: 4.3.1.2).

Summary of tolerance conflicts

A number of accepted tolerance standards are in potential conflict with the goal of achieving deflected elevated surfaces that are flat, level, and at the proper elevation. The ability of the contractor to achieve this goal is directly affected by these tolerances.

- Example 1 illustrates the manner in which stirrups that are fabricated slightly longer than required by design documents can force the resulting concrete surface above that which might otherwise be necessary.

- Example 2 shows that for elevations that are too high can also force the resulting concrete surface to a level higher than indicated by the contract documents.

- Examples 1 and 2 illustrate that restrictions on cross-section thickness of slabs and beams can restrict the ability of the contractor to produce a surface profile that is at the specified elevation.

- Example 3 shows that failure to place controls on the elevation of the structural steel platform will result in a concrete surface that has essentially the same variations as the supporting structural steel platform.

- Example 3 also points out that concrete cross-section tolerances restrict the ability of the contractor to respond to out-of-level conditions that might exist in a structural steel platform.

Common tolerance omissions

A number of important tolerance areas have been overlooked by the industry and are often not considered by the designer during preparation of contract documents for suspended slab construction. These omissions do result in a lack of control over the ultimate quality produced by the contractor and can result in increased cost to the owner or eventual tenant.

Several of the overlooked tolerance issues are common to all suspended slab construction, regardless of the type floor framing employed by the designer. The following must be considered and addressed by the specifier if control over the quality of the final product is desired.

Acceptable surface quality near slab edges and at projections

Concrete finishing operations near columns and sleeves normally involve hand-held floats and trowels. Finishing on areas unobstructed by projections utilizes bull floats, highway straightedges, power floats and power trowels. There is a general recognition in the industry that surface quality near columns and sleeves can be substantially different from that achieved elsewhere. The current approach in our industry is to either ignore this probable variation in quality or to exclude these areas from evaluation.

The area near columns is almost invariably included in any grid survey used to establish variations in the floor surface. The result is that data that are not necessarily reflective of the overall floor quality can play a critical role in its evaluation. ASTM E1155,⁴ which is used to evaluate surfaces using the F-Number system, specifically excludes slab areas within 2 ft of projections and slab edges.

Acceptable levelness of deflected slab surfaces

The complex interrelationship between construction of a floor system and the manner in which it deflects is recognized. One rationalization used for not addressing this issue is the fact that neither the contractor nor the engineer has complete control over the relative levelness of the deflected surface. Another is that application of a floor leveling material after construction has historically enabled the contractor to improve surface levelness sufficiently to meet the needs of the owner.

On the other hand, there are several arguments in favor of meeting this challenge. The cost of floor leveling with fill material is often high and its installation can disrupt completion of other scheduled work. Existing technology provides the means for rapid and economical identification of floor frame behavior during construction.⁷ Components critical to construction of successful suspended slabs have been identified.⁸ Procedures have been developed that allow the design/construction team to identify and to respond to local areas in which the floor surface does not deflect as originally anticipated by the engineer.⁷

The team approach to achieving levelness of the deflected surface has produced some exceptional results^{5,6} with an added benefit of fostering team solutions to other challenges that arise during construction. Levelness of deflected surfaces need no longer be avoided by the specifier.

Elevation differences at columns during construction

On high rise projects differential shortening of columns is fairly common. Engineers often adjust column lengths to minimize the effect of anticipated shortening or the effects of building foundation settlement. The relative elevation of the floor surface at adjacent columns can change between the time a particular level is constructed and completion of construction. Consequently, it is important for the engineer to provide guidance and information to the contractor concerning relative elevation differences that should be observed at the time specific levels are constructed. This information gives the contractor a target against which he can evaluate his work during construction.

In structural steel construction, designers generally rely on the AISC Code to establish tolerance requirements for erection and fabrication of the structural steel. The incompatibility of ACI and AISC tolerances, however, dictates that the designer/specifier take steps to resolve differences in the two documents.

One approach is to include in the contract documents provisions that establish elevation control over the erected structural steel. The specifier can establish the degree to which structural steel column splice points should conform to specified grade. This same approach can be applied to the elevation of beam-to-column connections. Finally, the specifier can establish an acceptable variation of closure angle elevations from specified grade prior to concreting.

Conclusion

Current methods of tolerancing suspended slab construction are inadequate and often internally inconsistent. If the contractor is to have the ability to produce deflected floors that are flat, level, and at the proper elevation, the specifier must consider alternatives to common tolerancing methods. Our industry needs to take a serious look at the variables involved and to make some decisions concerning approaches that will allow the designer and contractor to work together to produce the product desired by the owner.

References

1. ACI Committee 117, "Standard Specifications for Tolerances for Concrete Construction and Materials (ACI 117-90)," American Concrete Institute, Detroit, 1991, 12 pp.
2. "Code of Standard Practice for Steel Buildings and Bridges," *Manual of Steel Construction — Load & Resistance Factor Design*, American Institute of Steel Construction, Chicago, 1986, pp. 6-221 to 6-243.
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6. Tipping, E., and Rajagopalan, K.S., "Flatness and Levelness of Elevated Surfaces," *Concrete International*, V. 12, No. 1, Jan. 1990, pp. 52-61.
7. Tipping, E., and Suprenant, B., "Construction of Elevated Concrete Slabs — Measuring and Evaluating Quality," *Concrete Construction*, V. 36, No. 3, 1991, pp. 260-268.
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Point of View

Controlling the Quality of Suspended Slab Construction

by Eldon Tipping

Buildings come in many shapes and sizes. As available materials and floor framing approaches are brought together, unique combinations are created. The resulting floor systems don't always deflect as anticipated by the engineer. One solution is to recognize this uniqueness and to incorporate additional control measures and flexibility in the construction process. This will allow the design/construction team to respond intelligently to observed behavior of the structure during construction.

In "Tolerance Conflicts and Omissions in Suspended Slab Construction," the author showed that accepted standards and tolerances are not capable of controlling the quality of deflected suspended surfaces.⁵ This article presents methods and specification language that can be used to develop the control and flexibility necessary to produce high quality suspended floor surfaces. Use of these measures will involve a trapeze-like process of letting go of accepted ways of approaching problems and grabbing on to new procedures and requirements.

While the concepts presented herein are applicable to all structural systems, the language is generally limited to conventionally reinforced cast-in-place concrete and concrete slabs on metal deck. Post-tensioned structures and precast concrete structures are not specifically addressed.

Creating a surface at the correct elevation

A deflected cast-in-place concrete surface that is at the correct elevation can best be achieved by minimizing the plus tolerances that can force the final surface above specified grade. Those that appear to be most important in this area are the plus tolerance on fabricated length of beam and joist stirrups (Reference 1: 2.1) and the plus tolerance of formed surfaces prior to removal of shores (Reference 1: 4.3.2).⁵ An additional plus tolerance of importance is that for concrete cross-

sectional dimensions for beams and slabs (Reference 1: 4.4).⁵ Allowing increased thickness of slabs and beams provides better ability to achieve a surface that is at the specified elevation.

Alternative tolerances

The following alternatives to tolerances contained in ACI 117 address these issues. Some of these provisions are more strict than the tolerances listed in ACI 117, and may increase construction cost slightly. The specifier should evaluate the relative importance of achieving floor surfaces that are at the specified elevation against these possible increased costs:

1. Reinforcing Steel Fabrication

(a) The fabrication tolerance for bar sizes #3, #4, and #5 (Tolerance Symbol 1 in Fig. 2.1(a), ACI 117) used as column ties or stirrups is +0 and - 1/2 in. [50 percent of the current ACI tolerance] where the gross bar length is less than 12 ft.

(b) The fabrication tolerance for bar sizes #3, #4, and #5 (Tolerance Symbol 1 in Fig. 2.1(a), ACI 117) used as column ties or stirrups is +0 and - 3/4 in. [37 percent of the current ACI tolerance] where the gross bar length is 12 ft or more.

2. Elevation of Formed Surfaces

The elevation tolerance of formed surfaces before removal of shores is +0 and - 3/4 in. [50 percent of the current ACI tolerance].

3. Cross-Sectional Dimensions

(a) The tolerance for thickness of slabs (12 in. or less) is + 3/4 in. [200 percent of current ACI tolerance] and - 1/4 in. [the current ACI tolerance].

(b) The tolerance for thickness of beams (more than 12 in. but less than 3 ft) is + 3/4 in. [150 percent of the current ACI tolerance] and - 3/8 in. [the current ACI tolerance].

Developing successful control measures and tolerances for concrete slabs on metal deck is more difficult. Concrete tolerances control the slab surface elevation relative to specified grade; structural steel tolerances are stated in terms relative to the fabricated member. The contractor does not have the same degree of control over the levelness and elevation of the supporting structural steel platform that he has over that of formwork.

POINT OF VIEW This article was selected for reader interest by the editors. However, the opinions expressed by the author are not necessarily those of the American Concrete Institute. The editors invite comments from our readers about the personal views given in this article.

The solution has several components.

The first is for the specifier to force compatibility of concrete and structural steel tolerances. This can be done by incorporating in project specifications those elements that create elevation tolerances for the erected steel frame that are compatible with the concrete slab that it supports.

The concrete contractor is forced to work with the basic platform that results from erection of prefabricated elements. A second component of the solution is for the specifier to increase the plus tolerance for concrete slab thickness above the supporting steel. A third component is to address overall levelness of the final surface rather than its conformance to specified grade.

Plumbing operations associated with erection of structural steel columns often result in a gap at column splice points. This gap can create as much as $\frac{1}{2}$ to $\frac{3}{4}$ in. growth in column length over ten tiers. The fourth, and final, component of the solution is for the designer/specifier to create a mechanism that allows the contractor to react to job site conditions. This can be done by implementing a monitoring program and by developing a procedure for responding to results of the monitoring program.

Each of the preceding components can be addressed effectively in the contract documents. If these documents do not provide this control, the contractor can resolve many of them during preconstruction planning and by monitoring results during construction. The following language is one approach specifiers might use to address these components:

1. Tolerances

Fabrication and erection shall conform to the AISC "Code of Standard Practice for Steel Buildings and Bridges,"² and "Standard Mill Practice"³ except as noted.

- (a) Elevation tolerance for column splice points at the time the member is erected is $\frac{3}{8}$ in.
- (b) Elevation tolerance for the top surface of steel beams and girders at connections to columns at the time the floor is erected is $\frac{1}{2}$ in.
- (c) Elevation tolerance for closure plates at the building perimeter and at slab openings prior to concrete placement is $\frac{1}{4}$ in.

2. Testing Agency Shop Inspection and Testing of Materials

The testing agency shall measure and record the camber of all beams and girders in a manner that will allow that information to be compared to the as-erected condition of beams and girders. This list of recorded cambers shall be attached to the Shop Inspection Reports.

3. Surveys

- (a) *Initial Survey:* The contractor shall check elevations of concrete and masonry bearing surfaces and anchor bolt locations prior to

erection and submit any discrepancies to the engineer prior to the start of erection. Corrections or compensating adjustments to the structural steel shall be made and accepted prior to start of erection.

(b) *Erection Survey:* Column splice elevations shall be surveyed by the contractor at each tier. All beam-to-column connections and other locations designated by the engineer shall be surveyed by the contractor at each floor prior to concreting operations. Results of this survey shall be furnished to the engineer sufficiently prior to concrete placement to allow for adjustments in slab thickness or floor elevations that might be indicated.

If erection survey results indicate that an adjustment of column splice elevation is required, the engineer will provide written notification of such to the contractor. Subsequent adjustment in splice elevation of designated columns shall be implemented by the contractor at his first opportunity using methods previously accepted by the engineer.

(c) *Deflection Survey:* At typical beam locations to be accepted by the engineer prior to construction, the contractor shall provide spot grades from below at each end and at mid-span. The first set of spot grades shall be taken prior to concreting and a second set taken in the same locations within 24 hours after completion of finishing operations. Location of the spot grades should be marked to insure that both sets of readings are taken at the same locations.

Creating a level deflected surface

The most effective method of achieving a deflected surface that is essentially level is to strike off the concrete at the proper grade and then allow the floor framing system to deflect from this supported location into a final position that is essentially level. Often, the design/construction team will find that some portions of the frame do not deflect as predicted. These local areas can be identified after initial placements and appropriate adjustments made during construction after adequate information is provided to the engineer.

The author has recommended and the contractor has used the following procedures on previous projects with good results:

- Strike off the surface of concrete to a predetermined grade with the aid of rigid screed guides. Do not use wet screed guides on elevated surfaces.
- For unshored construction, such as slabs on metal deck, set grade for strikeoff a designated distance above the top surface of the steel floor members. Metal deck continues to deflect for a short period after stri-

keoff; restraighen the surface using a wide bull float. The contractor should create an initial slab thickness over beams that will accommodate restraighening of the surface while still maintaining adequate concrete cover over beams.

- In isolated areas where actual deflection of steel beams during concreting operations differs from that anticipated by the engineer, adjust concrete thickness or use loose shores on subsequent placements as necessary to offset observed behavior.
- Evaluate the concrete surface using the F-Number system. The data taken can be tied to specified grade and levelness of the deflected surface established by calculating the percentage of elevation samples that fall within an envelope of a specified depth.

Controlling surface quality at joints, sleeves, columns and walls

On suspended slab surfaces it is not unusual for the slab elevation to rise as it approaches projections like sleeves, columns and walls. Observations by the author indicate that this occurs in large part because finishing operations with the power float tend to push paste up to the projecting element. Subsequent finishing operations, normally by hand, do no more than smooth and compact the paste that is present at the time of that operation.

Surface quality near joints in suspended slabs is another issue that should be addressed in the contract documents. Circumstances that might be important and drive evaluation of slabs on grade do not carry the same importance where suspended slabs are concerned. On higher quality industrial slabs, different finishing techniques near slab edges and at joints make it desirable to eliminate this area from that tested when evaluating the floor surface. Edge curl is not a significant problem on suspended slabs. Tolerance requirements are also generally lower for suspended slabs than they might be for a slab on grade.

The author has recommended with good results on previous projects that the contractor provide bench marks at each column and wall element for use by the finishers. The marks can be used as a guide when concrete finishers are completing work in these areas. Marks can be placed a specified distance above design grade (trowels, for example, are 16 in. long) so the finisher may remove excess material as needed.

There are other procedures, still in the developmental stages, that appear to provide a reasonable opportunity for success:

- Limit the maximum elevation change that may occur within 2 ft of any column or wall element to 0.25 in. Require that the contractor grind or patch any non-conforming slab surface as necessary.
- Test slab surfaces in accordance with ASTM E1155,⁴ except as follows:

- Test the entire surface, including those areas within 2 ft of construction joints and vertical elements that project through the slab surface.
- Allow sample measurement lines that are perpendicular to construction joints to extend no more than 5 ft past the joint into the previous placement.
- Record elevation changes in each of four primary directions where possible at not less than 50 percent of column and wall locations.

Summary

Construction of suspended slabs that are reasonably flat, level, and at the proper elevation requires that current approaches be modified to provide improved control and flexibility during construction. Many of the ideas presented in this article place a heavier burden on both the contractor and engineer than is now the case. The contractor is required to develop additional data and the engineer is required to analyze and to respond to these data during construction. The additional cost of this increased involvement is minor compared to the benefit to be derived from improved quality.

The author's recommendations are generally applicable to normal commercial office building construction, but may not be restrictive enough for some specialized applications such as moveable file tracks. Your thoughts concerning these issues are invited and encouraged.

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ACI COMPILATION

Construction Load Analysis of Slabs and Shores Using Microcomputers

by Pericles C. Stivaros and Grant T. Halvorsen

Formwork failures and failures caused by improper shoring or premature removal of supports have been common throughout the history of concrete construction. A structure need not collapse to be influenced by premature removal of forms and shoring: construction failures or defects such as sagging or cracking can occur in the completed structure. If loaded prematurely, time-dependent deflections under load (creep) will be larger, and sags are more likely to be both noticeable and objectionable.

To avoid such unwanted long term effects or even a collapse during construction the contractor must be able to determine how the construction loads are distributed among the shoring system and the interconnected slabs and what the strength capacities of these young slabs are upon the application of the construction load. ACI's "Guide to Formwork for Concrete"¹ provides some guidelines regarding shoring, reshoring, and backshoring operations. However, it does not prescribe any procedure for determining the loads that will be transferred to the structure upon the new concrete placement or formwork removal. ACI's "Formwork for Concrete"² recommends the Grundy and Kabaila³ method (hereafter referred to as the simplified method) to evaluate the construction load distribution. There have also been several other efforts to evaluate construction loads by applying computer-based analytical methods.⁴⁻⁸ The pioneering work of Grundy and Kabaila has become the basis for most of the research work on the subject.

An alternative computer-based method for construction load analysis and slab safety check has been developed. The computer program is based on a two-dimensional structural model according to the equivalent frame method as described in ACI 318.⁹ This procedure (hereafter referred to as the equivalent frame method) is found to be effective and accurate for the evaluation of construction load effects, and can be implemented in a form usable at the construction site. Analyses conducted using the equivalent frame method showed comparable results to finite element plate bending models.¹⁰

With the advent of widespread use of computers, a more realistic model can and should be employed. The equivalent frame method computer-based procedure

takes into account the effects of structural continuity and the monolithic nature of concrete structures with a reasonable level of computational effort.

Strength adequacy of early age members

The load capacity of an early age slab is influenced primarily by the rate of concrete strength development and the total service live and dead load for which the slab has been designed. The flexure, shear, and bond strength capacities of the early age slab can be reasonably assumed to be proportional to the partially developed concrete compressive strength. Cracking and deflections are also dependent on the early age concrete tensile strength and modulus of elasticity, respectively.

The reserve slab strength for which the slab has been designed for both service live loads and superimposed dead loads (partition loads) is usually sufficient to carry the imposed construction loads when the building is being constructed. When the applied construction load on a slab is more than the slab's available capacity the proposed construction scheme is at risk and a failure may occur. In such a case there are two basic alternatives: either reduce the load on the slab at the critical age, or increase the concrete strength. The first alternative can be achieved by modifying the type of shoring system or the number of shored or reshored floor levels in such a way that the applied construction load is reduced to an acceptable level. The second alternative can be achieved primarily by using high-early strength concrete, by controlling curing temperatures to achieve the required early concrete strength, and by increasing the duration of the construction cycle to permit the concrete to gain enough strength before the application of a heavy construction load.

Construction loads

The construction dead load, which usually includes the slab dead load and the weight of the formwork, can be estimated with reasonable accuracy. The estimation of the construction live loads, however, can be a very difficult task. The construction live load includes the weight of workers, equipment, materials, runways, and impact produced by concrete placement or equipment operations. ACI 347,¹ "Recommended Practice for Concrete Formwork," suggests a construction live load

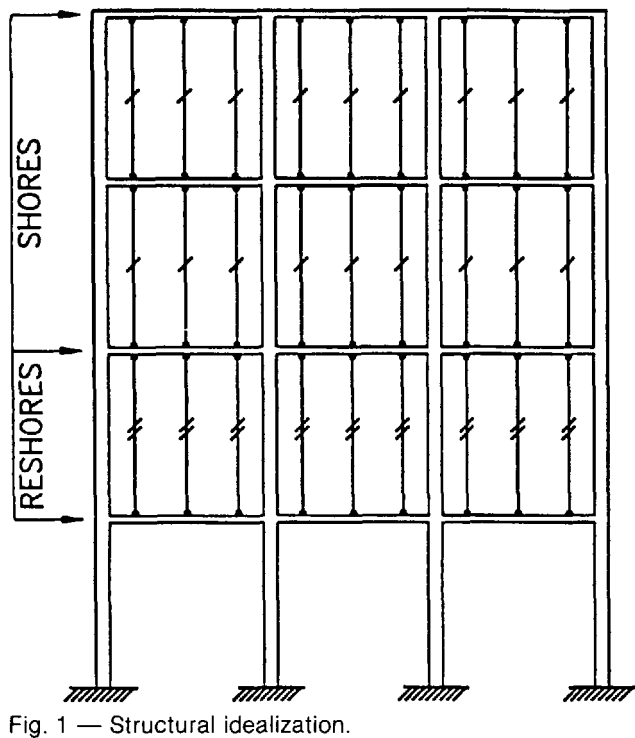


Fig. 1 — Structural idealization.

of 50 psf (2.4 kPa), while OSHA¹¹ requires at least 20 psf (1.0 kPa) for live load. A study by Fattal at the National Institute of Standards and Technology¹² indicates that when concrete is placed by bucket, the live load may be as high as 40 to 50 psf (1.9 to 2.4 kPa). The construction live load is usually applied to the uppermost slab during concrete placement on that slab, and it is assumed to be removed after the concrete placement is done.

Construction load factors

While ACI 318 specifies load factors for many load types it does not specify a load factor for construction loads nor member capacities at early age. ANSI A10.9 (1983)¹³ recommends a load factor of 1.3 for both dead and live construction loads. There is a divergence of opinion on the proper load factors. Gross and Lew¹⁴ contend that "construction loads are generally short in duration and the consequences of a failure are not as great as they would be in a fully occupied building." They suggest that the load factors should be less than those used for service conditions. On the other hand, Gardner¹⁵ maintains that "the construction workers spend all their working life on construction sites and their risk of injury should not be greater than the occupants of the completed structure." The variability of results from the available methods to evaluate construction dead loads and the uncertainty of construction live loads suggest high construction load factors. The writers prefer the more conservative load factors required by ACI 318 for service conditions, i.e., 1.4 and 1.7 for construction dead and live loads.

Structural model

A multistory building under construction is modeled according to the assumptions of the equivalent frame method. A schematic representation of an idealized two-dimensional structure is shown in Fig. 1. The shores and reshores are replaced by an equal number of elastic supports, each having a stiffness equal to the total stiffness of the corresponding shores. It is assumed that the reshores carry no load at the time of installation, i.e., the slabs are allowed to deflect under their own weight and construction loads applied prior to the installation of reshores. The load of the freshly cast concrete is applied to the top floor shores as concentrated loads. The load is shared among the shores according to their contributory areas. The connection between the shores or reshores and the floor slabs can be reasonably assumed as pin-jointed. At ground level, shores and reshores are assumed to be supported on a rigid foundation.

The equivalent frame method concepts are applied to determine the frame member stiffness coefficients. The idealized two-dimensional frame is analyzed elastically using conventional stiffness analysis procedures to evaluate the forces in the various frame members, including the shores and reshores.

Construction phases

A building under construction is a structural system that changes with time as the construction advances. Time also affects the strength and elastic properties of concrete.

In a typical construction cycle for a scheme that uses shores and reshores there are four repetitive operations, or construction phases, for each floor slab:

- Installation of the shores and formwork for casting the next floor.
- Removal of lowest level of reshores.
- Removal of lowest level of shores and formwork.
- Placement of upper level of reshores in the story where the shores and forms were removed.

If no reshores are used, the second and fourth phases are eliminated. An example of two levels of shores and one level of reshores in a three-bay multistory structure illustrates the four phases (Fig. 2).

During the first phase, the weight of the fresh concrete is distributed among the previously cast slabs through the shoring system. This load transfer mechanism is disturbed when the lowest level of reshores is removed. The floors below are relieved of the construction load, while the floors above are called on to carry all of the construction load alone. Similar load path disturbance to the structural system occurs when the lowest level of shores is removed. The installation of the reshores during the fourth phase does not disturb the system since the reshores are assumed to be relatively load free upon installation.

Computer program

A computer program was developed to determine the

construction load distribution. The operation of the program is broken down to five stages: data input, phase of construction, structural identification, stiffness analysis, and construction load analysis. These steps, except for data input, are repeated for each floor slab under construction, from the first floor to the roof. An abbreviated flow chart of the computer program is shown in Fig. 3.

Input data

The input data consists of structural, material, construction schedule, and load data.

The structural data includes the number of bays and stories, bay widths and story heights, slab construction type, and slab, beam, column, and shoring member sizes. Only the typical member sizes are input, the rest are generated internally by the program.

The material data includes the modulus of elasticity of the shoring system, the concrete unit weight, the concrete design strength for slabs and columns, and the ratio of the design concrete strength to the early age strength. The user inputs the concrete strength ratio and the program provides the capability of planning construction schedules of irregular rate.

The construction schedule data includes the number of shored and reshored floor levels and the shoring placement configuration, i.e., the number of shore and reshore props in each bay and their location.

The load data includes the construction live load, formwork load, design service dead and live loads, and factors for the construction dead and live loads.

Structural identification

The purpose of the structural identification process is to convert the "raw" input data to a form suitable for the stiffness analysis of the program. During this process all joints and structural members, including shores and reshores, are identified and numbered. The proper coordinates are then assigned to each joint. The joint coordinates are evaluated internally by the program based on the given geometry of the structure. The appropriate restraint conditions are also assigned to each support joint. The modulus of elasticity and concrete strength are assigned to each member according to its age. The shore and reshore members are treated as truss elements during the analysis, i.e., they are assumed to be subjected only to axial compressive forces.

The stiffness of the slab-beam elements is modified to account for the variation of slab thickness due to the presence of beams, drop panels and column capitals. The stiffness of the columns is also modified to include the torsional effects of the adjacent slab strips on both ends of the column. The equivalent column stiffness formula, which is given in ACI 318 Commentary R13.7.4, is strictly valid only for one-story building frames which assumes that the far ends of the columns are fixed. For multistory building construction, where several stories are involved in the construction load distribution, the ACI formula is extended here to take into

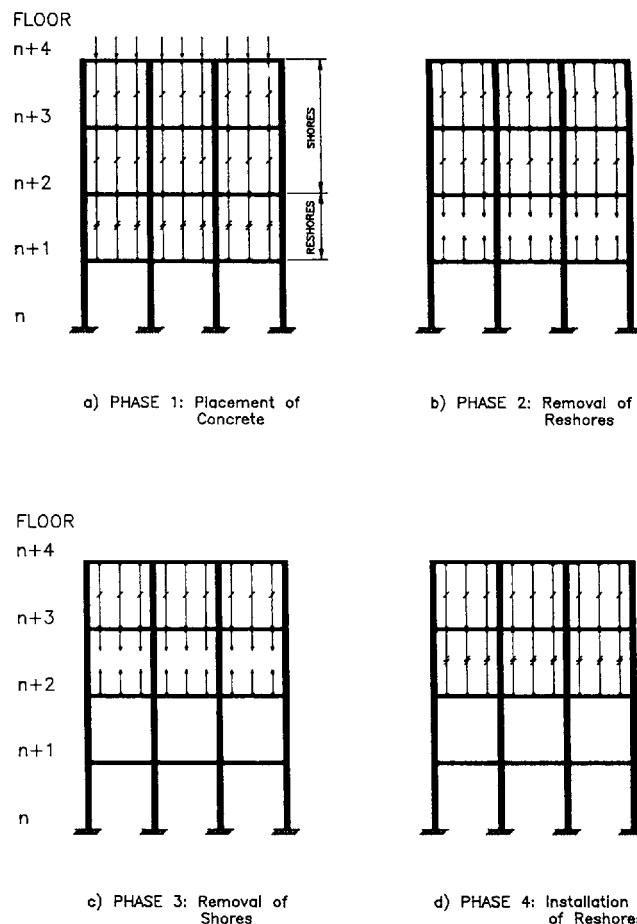


Fig. 2 — Construction phases.

account the torsional effects on columns from slabs on both ends of the column.

Phase of construction

After the structural data is processed, the program calls for the appropriate phase of construction to be considered. The program always starts from phase 1, casting of a slab, and then determines the next phase according to the given method of construction.

During this step the program determines the construction operation to follow, either concrete placement or shore removal, and evaluates the induced loading conditions due to the operation. The applied construction load is then assigned to the proper joints. For example, during phase 1 the applied load consists of the weight of the fresh concrete and the construction live load. This load is determined by the program and shared among the shore members supporting the fresh slab according to their tributary areas. The shore members transfer these loads to the floors below. The loads induced to the system due to the removal of the shores or reshores consist of the axial loading force in the members prior to their removal. These loads are applied at the end-joints of each member removed.

It is important to mention that all these procedures concerning the structural identification, data processing, and loading conditions are done without the interaction of the user. Defining the properties and location of all members, joints, joint coordinates, support re-

CONSTRUCTION LOAD ANALYSIS

COMPUTER PROGRAM

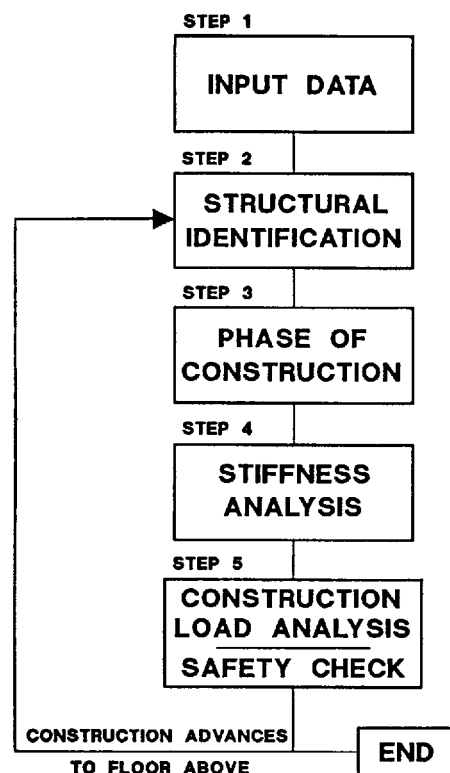


Fig. 3 — Abbreviated flow chart of the computer program.

straints, and loading conditions can sometimes be long and tedious. The user has to do none of these procedures for this computer program. All the required information is generated by the computer based on the basic building frame characteristics and method of construction given by the user.

Stiffness analysis

The structural member forces are determined through the stiffness analysis procedure. A modified direct stiffness method set up for this purpose takes into account non-prismatic members and mixed "beam" and "truss" elements.

For every construction phase a different structural configuration and loading condition is involved. Depending on the construction phase a member can be subjected to additional load or relieved from a previously applied load. Therefore, the member forces are evaluated for each phase and then they are superimposed on those evaluated from the previous phase.

The program insures that the shoring system does not go into tension by removing tensile forces (if they occurred) from the shores or the reshores and redistributing them among the interconnected slabs.

Construction load analysis

During this step, the computed member forces are further processed to determine the construction load carried by individual slabs and the shoring system. The amount of load carried by a slab is equal to the calcu-

lated shear forces of the slab-beam members transferred to the supported columns. The shoring system load is equal to the summation of the axial compressive forces of each individual shore or reshore member supporting that slab.

The evaluated construction loads are then multiplied by the specified construction load factors. Also during this step, the program evaluates the available factored load capacity of each slab, so a comparison can be made with the applied factored construction loads.

Construction example

A multistory flat plate structure was examined to illustrate the application of the computer program. The structure was assumed to have been designed according to ACI 318⁹ with a live-to-dead load ratio of 0.75. One level of a steel flying truss shoring system and three floor levels of wooden reshores with a construction rate of one floor per week was assumed. The shores and reshores were idealized as series of vertical truss-type elements with stiffnesses equivalent to the total stiffness of the shores or reshores on each row. The steel flying truss system was considered infinitely stiff relative to the concrete slabs. However, the compressibility of the wooden reshores was considered. The weight of the shoring system was assumed to be 10 psf (0.5 kPa). It was also assumed that the shores and reshores were removed and relocated a day before casting the top slab. A 50 psf (2.4 kPa) construction live load was considered to be present only during the concrete placement of the top slabs. A typical floor plan of the equivalent frame along with the location of the flying truss screw jacks and wood reshores is shown in Fig. 4.

Other assumed geometric and material parameters were: 10 ft (3.0 m) story height, 8.0 in. (200 mm) slab thickness, 28-day compressive strength of concrete $f'_c = 4000$ psi (27.6 MPa), and the wooden shores' modulus of elasticity $E_w = 1500$ ksi (10.3 GPa). The assumed ratio of the actual early-age concrete strength f_c to the 28-day strength is given in Table 1.

The results of the computer analysis are summarized in Table 1. The table shows the applied factored load on each slab, the slab age, and the slab load capacity based on the partially developed concrete strength. The load applied on the slabs is normalized to the self weight of each slab. As can be seen, the proposed construction scheme overloads all seven-day old slabs and consequently this construction procedure cannot be safely executed.

To make this building constructible, alternative construction procedures must be considered. One alternative is to increase the number of shored or reshored levels so that the maximum construction loads will be shared by older slabs that have more load capacity. A previous study by the authors¹⁶ shows that an increase of the number of shored levels results in an increase of the construction load applied to the older slabs. Sometimes, the resulting load is higher than the 28-day strength of the slabs. Indeed, that is the case for this example. An increase in the number of shored levels results in a factored construction load that is more than the designed capacity of the slabs. In such a case, a column-mounted shoring system may be used to transfer the construction load directly to the columns, thus eliminating the vertical shoring.

A second alternative is to increase the concrete strength. This can be achieved either by using high-early-strength concrete or by reducing the construction rate. If the construction rate is reduced from seven days to fourteen days per floor the applied construction load of $2.48D$ will be less than the available capacity of a fourteen day old slab which is $2.49D$, where D is the self weight of the slab. However, this alternative may not be a desirable solution because it will delay the completion of the project.

It is important also to mention that had the above structure been designed for live-to-dead load ratio equal to 1.0 then the 7-day slab capacity would have been about $2.5D$. In such a case the proposed construction scheme would have been safe. This shows how critical the coordination task undertaken by the construction engineers and contractors is in multistory building construction.

Simplified method

This construction example was also examined using the simplified method of Grundy and Kabaila. The ultimate factored construction loads on each slab are evaluated using the method suggested by Gardner.^{4,15} Gardner proposed to increase the construction dead loads determined from the simplified method by 10 percent to cover for the inherent errors of the simplified method. He also assumes that the construction live load is equally shared among the interconnected slabs. The dead and live loads were multiplied separately with the appropriate construction load factors.

The results of this analysis are shown in Table 1 in parenthesis. The analysis shows that only the seven-day old slabs No. 4 and 5 are overloaded. The simplified method failed to predict the overloading of the No. 1, 2, and 3 seven-day old slabs even with the 10 percent modification coefficient. This shortcoming of the simplified method stems from the fundamental assumption of the method that the shoring system is infinitely stiff as compared with the supported slabs. Based on this assumption, the No. 1, 2, and 3 seven-day old slabs are supposed to carry their own weight only, since the weight of the newly cast slab, the weight of the formwork, and any construction live load are assumed to be transferred to the foundation through the "rigid" shoring system. It is also interesting to observe that the slab overloading occurred after the first story reshores were removed, i.e., when the direct load path to the foundation was removed.

With compressible shores and reshores the structural system tends to shift the slab loads to the uppermost floors. The upper level floor slabs, being the most recently cast, may not have gained sufficient strength to carry the loads without overload and possible failure. This is especially true when reshored levels are used in combination with the shored floor levels.

Summary

This study recognizes the importance of the close cooperation between the design and construction teams to erect a structure safely, and also to maintain the serviceability of the building during its future use.

The use of multiple shored or reshored levels does

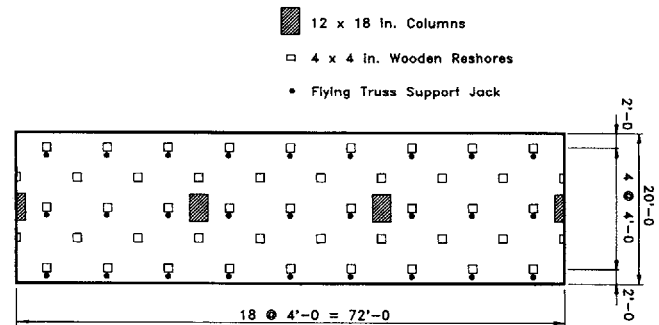


Fig. 4 — Typical floor plan for the construction example.

Table 1 — Shoring analysis computer results

Age (days)	f/f_c	Available strength	Total factored construction load				
			Slab 1	Slab 2	Slab 3	Slab 4	Slab 5
6	0.76	$2.03D$	1.40 (1.54)*	1.40 (1.54)	1.40 (1.54)	1.40 (1.69)	1.40 (1.69)
7	0.79	$2.11D$	2.18 (1.54)	2.33 (1.54)	2.39 (1.54)	2.48 (2.33)	2.48 (2.33)
13	0.92	$2.46D$	1.35 (1.54)	1.33 (1.54)	2.26 (2.12)	2.22 (2.22)	2.22 (2.22)
14	0.93	$2.49D$	1.62 (1.54)	1.69 (1.54)	2.00 (2.33)	1.95 (2.33)	1.95 (2.33)
20	0.99	$2.65D$	1.33 (1.54)	1.80 (2.12)	1.87 (2.22)	1.82 (2.22)	1.83 (2.22)
21	0.99	$2.65D$	1.47 (1.54)	1.90 (2.33)	1.94 (2.33)	1.89 (2.33)	1.91 (2.33)
27	1.00	$2.68D$	1.97 (2.12)	2.04 (2.22)	2.08 (2.22)	2.06 (2.22)	2.06 (2.22)
28	1.00	$2.68D$	2.31 (2.33)	2.33 (2.33)	2.37 (2.33)	2.35 (2.33)	2.35 (2.33)

*Numbers in parenthesis are based on the simplified method.
D = Slab self weight.

not necessarily guarantee safe construction. A rigorous analysis must be performed to evaluate the safety and economy of alternate construction procedures. Special caution should be exercised when compressible wooden shoring systems are used. In such cases the construction load is shifted to the shored slabs of the upper floor. Due to its inherent shortcomings, the simplified method underestimates the construction loads. The 10 percent correction coefficient of the simplified method is not adequate for all construction procedures.

Our computer program model, based on the equivalent frame concept, can analyze multistory buildings having different floor types and thicknesses, spans, and shoring systems. The program runs on a desk top personal computer, thus making the method available to construction sites. The method is a very useful tool to develop rapid and safe construction schedules.

Acknowledgements

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Unbonded P-T Slabs: An Economical Alternative

by Angelo Mattacchione

Hundreds of millions of square feet of post-tensioned (PT) concrete slabs have been built throughout North America since the mid-1960s.¹ On the basis of this quantity of construction alone many owners, contractors, and consultants would consider PT concrete slabs to be a viable method of construction for a variety of structures.

Nevertheless, in some areas there exists a reluctance to consider this alternative system seriously. This can usually be attributed to one or more of the following factors:

- Lack of familiarity with the system by owners and contractors
- Lack of familiarity with analysis and design procedures by consultants
- Insufficient number of suppliers
- Concern about the tensile forces locked into PT tendons
- Cost

However, experience over the past 25 years has proven that fears about the use of PT slabs are unfounded. More importantly, PT slabs carry with them inherent benefits not available with other systems, and are a viable alternative to regularly reinforced (RR) slabs. Combined with the fact that PT concrete slabs can, in many instances, be less costly to construct than their RR counterparts, then the owner may be suffering a disservice if PT slabs are excluded from consideration.

The system

Post-tensioning is a form of prestressing in which high-strength cable, which is used as the primary re-

inforcement, is tensioned after the concrete is placed. In PT concrete slabs, one generally wants to provide sufficient forces in the cables to inhibit tension cracks from forming under working loads. In this way, the entire concrete section is engaged, allowing longer spans with shallower, and therefore lighter, sections. Since the gross section is active, deflections under working loads are either reduced or eliminated.

PT concrete construction is classified as either bonded or unbonded: The high-strength strands are either bonded to the concrete by grouting the tendon sheath after stressing, or they are greased and plastic covered, thus remaining unbonded from the concrete. The system uses either 0.5 or 0.6 in. (12.7 or 15.2 mm) seven-wire strands, with a guaranteed ultimate tensile strength (GUTS) of 41.3 or 58.6 kips (184 or 261 kN), respectively.

Typically, bonded PT is multi-strand, with 2 to more than 50 strands enclosed in either flexible metal or rigid thin-wall sheaths with diameters ranging from 1.5 to over 7 in. (40 to 180 mm). Its applications include bridges, circular liquid storage tanks, and linings of pressure tunnels. Their diameter, as well as the time and cost associated with grouting usually all but precludes their use in slabs.

Unbonded PT is a single-strand system. The individual strand is coated with a rust-inhibiting grease, then is extruded in a seamless plastic sheathing. Its applications include slabs-on-grade, structural mats and raft footings, circular

fluid tanks, and tilt-up walls. Because the unbonded tendon is small in diameter as well as light and flexible, it can make maximum use of the entire depth of a concrete slab, making it particularly well suited to thin slabs, and the time and cost associated with grouting is eliminated.

With unbonded PT construction, the high-strength strand is placed in a draped configuration within the slab depth, usually with high points at supports and low points between supports (Fig. 1). The concrete is then placed, and after sufficient curing [from 1500 to 2500 psi (10.3 to 17.2 MPa), depending on the anchorage system], the steel strand is stressed against the concrete, then locked off under stress at end or intermediate anchor points (Fig. 2).

In addition to inhibiting rust, the grease around the strand reduces friction to eliminate any bond between the plastic and the steel, allowing the strand to slide freely and more uniformly distribute the tensile stresses in the strand.

Structural behavior

Previously, it was believed that tendon placement into column and middle strips was required for a PT slab to behave as a flat plate. Distribution of forces from 50 to 100 percent to the column strips, and up to 50 percent in the middle strips, have been used successfully. Such distributions were normally adaptations of recognized values for regular reinforced concrete design factors. These arrangements produced a two-way weave that required

careful sequencing procedures, greatly reducing productivity in tendon placement and increasing the system's overall cost.

More recently, tests and in-situ applications have shown that a PT slab behaves as a flat plate regardless of the tendon placement.¹ Not only were the sequenced PT slabs time consuming to build, but their distributions failed to recognize that tendons exert downward forces at the high points joining parabolic profiles.

Tendon high points should be, as nearly as possible, resisted by columns, walls, or upward tendon forces to achieve minimum deflections and maximum shear capacities. This implies that all tendons in one direction should be placed over or immediately adjacent to column locations, and the tendons in the perpendicular direction should be uniformly spaced across the bay width. This tendon placement is referred to as a banded system. Such a statically rational, yet simple and elegant, distribution of tendons is especially advantageous in allowing the easy tracing of load paths to supports in slabs with irregular column layouts (Fig. 3).

Apart from its obvious structural advantage, the banded tendon arrangement simplifies construction sequencing. First, all of the banded tendons in one direction are placed, then all of the uniform tendons in the perpendicular direction are placed. No weaving of tendons is required, reducing labor costs. Since its introduction, the banded system of tendon placement has gained wide acceptance and preference over its sequenced counterpart and is now used almost universally for PT concrete floor slabs.

Precut tendons usually are shipped to the site in coils of about 5 ft (1.5 m) in diameter. Each tendon is individually marked, clearly identifying its location in the slab. Tendons longer than about 100 ft (30 m) usually require stressing at intermediate points along their length. Dead end anchorages are locked into place at the shop and nailed to the slab edge form on site. Intermediate and stressing end an-

chorages are placed loosely on the cable (at the approximate location required) to allow exact placement on site. To minimize slab shortening, and to limit friction losses, the maximum distance between construction joints in the slab should not exceed about 150 ft (45 m).

As mentioned previously, with the banded system, all the tendons in one direction are placed first over (or adjacent to) the columns. Then the tendons are placed uniformly in the other direction, spaced at about 6 to 8 times the slab thickness. The concrete is placed once all of the slab reinforcement is in place and securely tied. Any admixtures non-injurious to steel may be used in the concrete.

The concrete forms are left in place until stressing is complete. Unlike RR slabs, in which deflection must occur to activate the reinforcement, the reinforcement is active in PT slabs, usually providing uplift forces between supports. Consequently, engineers and contractors have found reshoring of PT slabs to be less onerous than for RR slabs.

Approximate span/depth ratios found to provide both economy and adequate performance for members with live load less than dead load are:¹

one way slab	48
two way slab	45
two way slab with drop panel (minimum drop $L/6$ each way)	50
two way slab with two way beams	55
waffle slab, 5 ft (1.5 m) square grid	35
beams $b \approx h/3$	20
beams $b \approx 3h$	30

Applications

Flat plates and slabs employing PT reinforcement have been used in offices, apartments, condominiums, and parking structures throughout North America, where several million square feet of post-tensioned flat plates are completed each year.² Surveys carried out by the Post-Tensioning Institute indicate that over 250 million ft² (23 million m²)

of PT slabs were built between 1967 and 1976.¹

The longer spans possible with PT reinforcement provide the advantage of additional versatility in column layout. In addition, the use of PT reinforcement is not limited to regular or orthogonal placement, but is adaptable to irregularly shaped floor plans.

Most modern parking structures are built of cast-in-place concrete. When these structures are properly detailed and reinforced with PT tendons, cracks associated with bending moments can be either reduced or eliminated, helping to relieve some of the problems historically associated with deck deterioration.

Post-tensioning has been found to be economical for slabs, beams, transfer girders, and foundations, as well as for a variety of other applications. It has been successfully used in both low- and high-rise structures, including apartments and condominiums, hospitals, commercial buildings, industrial buildings, and parking facilities.

Discussion

During the mid-1950s, when PT was first being used in the United States, little was known of the behavior of PT reinforced slabs. Consequently, the analysis methods were awkward and time consuming, and the design was conservative. The advent of T.Y. Lin's load-balancing technique, introduced in 1963, removed the mystery of post-tensioning, and the analysis of PT structures was greatly simplified.³

The load-balancing method is the most widely used procedure for analyzing and designing PT structures. To simplify matters even further, as with its RR concrete counterpart, computer programs are now available for analyzing and designing PT slabs and beams. Although they are no substitute for experience and sound engineering judgment, such programs make the repetitive and calculation-intensive aspects of PT analysis and design less tedious.

Even in areas where only one PT supplier is available, it still is possi-

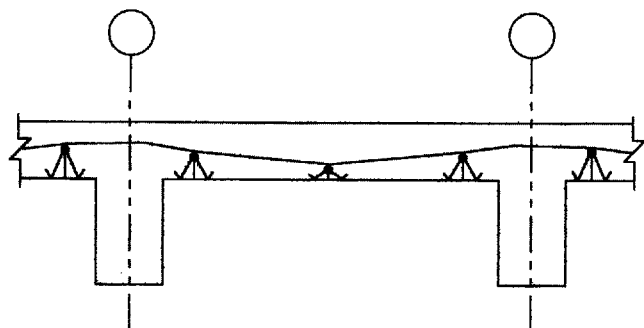


Fig. 1 — Typical slab section showing drape of post-tensioning cable between supports.

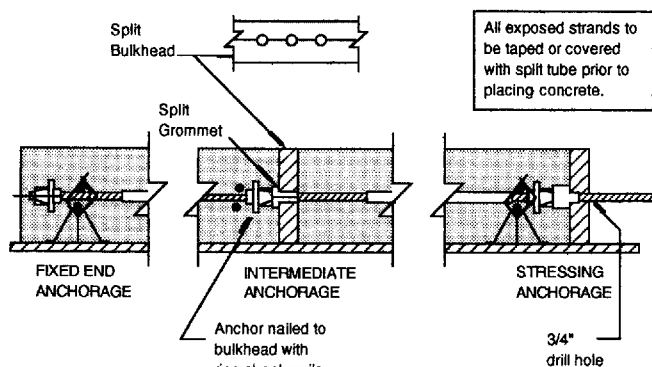


Fig. 2 — Anchorage details.

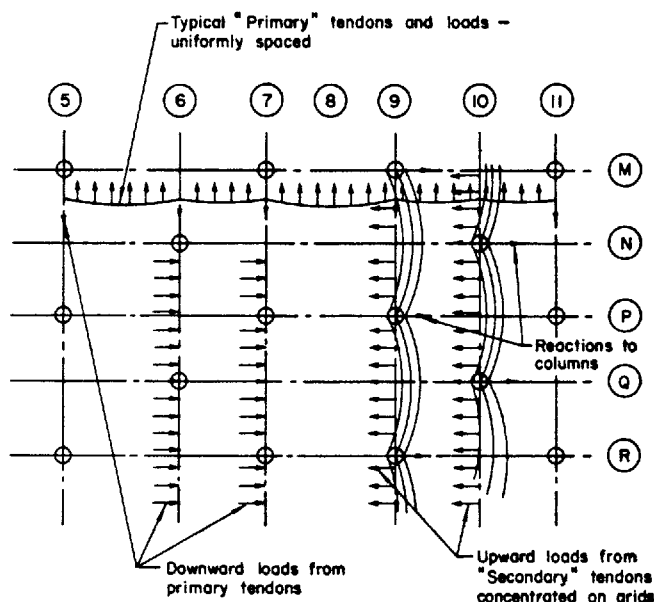


Fig. 3 — Flat plate with irregular column layout. Note tendon forces easily traced to column locations with banded system of tendon placement.

ble to include PT concrete in competitive bidding. Typically, PT suppliers have in-house engineering capabilities. In areas of limited PT competition, the structure can be designed in regular reinforced concrete, with the bid package set up to accept unbonded post-tensioning as an alternate to the regular reinforced concrete slabs. The PT proposal can be set up so that the PT supplier includes the analysis and design of the slab for post-tensioning, as well as the material and labor for its construction. In this way, an owner can automatically be assured that the system used is the least expensive available without incurring the front-end cost of multiple designs.

Still, the reluctance of some to use unbonded post-tensioning is rooted not in a lack of suppliers or insufficient familiarity with PT systems or their design principles, but in a concern about the tensile forces locked in unbonded post-tensioned tendons. This concern takes three main forms.

The first is a belief that the loss of one span of this continuous member system will precipitate the collapse of the entire structure.

Model tests carried out in Switzerland and the U.S., as well as accidents on buildings with construction in progress, have shown that the PT tendons do not respond to localized areas of overloading, and that PT slabs exhibit good structural redundancy and toughness.⁴ Further, a survey of PT tendon performance in a variety of structures concludes that properly detailed and constructed PT structures exhibit excellent performance with respect to durability against corrosion.⁵

The second common concern that surfaces about PT slabs is that of the new opening required after the slab is built. The banded system used in PT construction provides large spaces between tendons within which to locate small openings. Cable-locating equipment helps to ensure that the opening is located away from tendons. Until recently, large openings such as are sometimes required for new stairs between floors presented more of a problem, since tendon cutting could not be avoided. But a procedure now has been developed for cutting large openings in PT slabs.⁶ The method is fast, relatively simple, and — most importantly — con-

trolled. It is possible because of a newly developed detensioning jack that makes a previously dangerous undertaking a safe procedure.

The third common issue raised about unbonded post-tensioned slabs is the question of what happens if and when the structure needs to be demolished. Many have visions of unbonded tendons shooting out from the slab edges. In a report on the demolition of 100,000 ft² (10,000 m²) of PT slab, only a small percentage exited from the slab edge, and in no event was the exit more than 18 in. (450 mm).² The conclusion was that an unbonded PT slab can be demolished safely and economically.

Thus, the most common problems and concerns associated with PT slabs have been either resolved or shown to be unwarranted.^{2,6-9*}

Advantages

One of the main benefits of unbonded post-tensioning is the abil-

*Aalami, Bijan O., and Barth, Florian G., "Restraint Cracks and their Mitigation in Unbonded Post Tensioned Building Structures," ACI Convention presentation, Baltimore, Maryland, Nov. 1986.

ity to achieve longer clear spans. If the use or layout does not permit longer spans, then using PT results in a reduction of both concrete quantity and reinforcing steel. Thinner slabs produce a decreased dead load and a possible consequent reduction in foundation size.

Thinner slabs also produce a decreased floor-to-floor height, resulting in an overall shorter building; thus, exterior skin, vertical piping, ducting, and conduit costs are reduced. In addition, reduced floor-to-floor heights may make it possible to squeeze in an extra floor.

Forming costs can sometimes be lowered as a result of the reduction

(or removal) of drops and beams. Because tendons stressing can occur at relatively low concrete strengths, earlier formwork stripping is possible, and with it comes faster placing cycles,¹⁰ which can translate into decreased borrowing costs due to earlier project completion. All of these factors point to a saving in the overall cost of a building.¹¹

Cost comparison

In the final analysis, hard costs play a significant role in the decision to use or not use a particular system. To carry out a relative cost comparison between RR and PT concrete, computer runs were carried out on

a number of slabs, with spans varying from 18 to 40 ft (5.5 to 12.2 m), for residential, office, and retail loading. The cost of framed concrete slabs consisted of the concrete, reinforcement and formwork components. The programs used for the both the RR and PT concrete analysis are based on the equivalent frame method.

In the analysis, 4000 psi (27.6 MPa) concrete was assumed throughout. A reinforcement cover of 1 in. (25 mm) was used for the RR slabs, and 3/4 in. (19 mm) for the PT slabs. The comparisons made are shown in Table 1.

The material quantities obtained

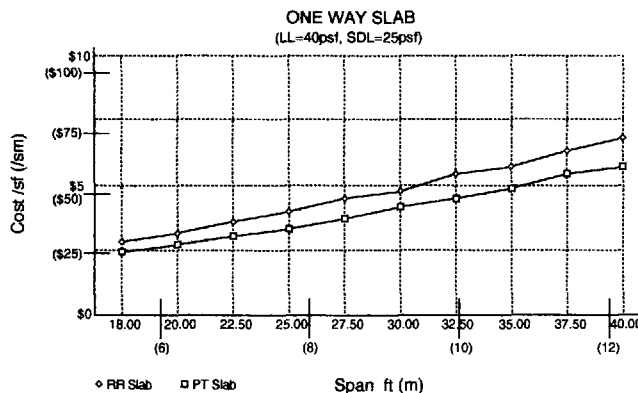


Fig. 4 — Cost versus span for RR and PT slabs.

Table 1 — Slab types used to compare costs for RR versus PT concrete

Occupancy	Framing method	Span variation, ft (m)	Live load, psf (kN/m ²)	Superimposed dead load, psf (kN/m ²)
Residential	One-way slab	18 - 40 (5.5 - 12.2)	40 (1.9)	25 (1.2)
Office	Two-way slab	18 - 40 (5.5 - 12.2)	50 (2.4)	25 (1.2)
Retail	Two-way slab	18 - 40 (5.5 - 12.2)	100 (4.8)	25 (1.2)

Table 2 — Unit cost values for RR/PT slab comparison

Concrete	\$105.00/yd ³	(\$137.24/m ³)
Regular reinforcing	\$0.55/lb	(\$1.21/kg)
Post-tensioned reinforcing	\$2.30/lb	(\$5.07/kg)
Forming	\$5.30/ft ²	(\$57.03/m ²)

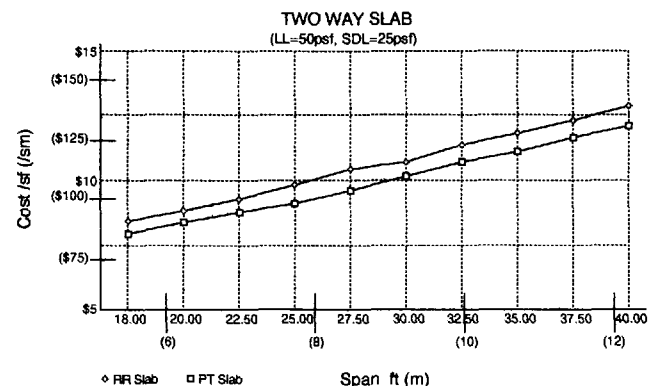


Fig. 5 — Cost versus span for RR and PT slabs.

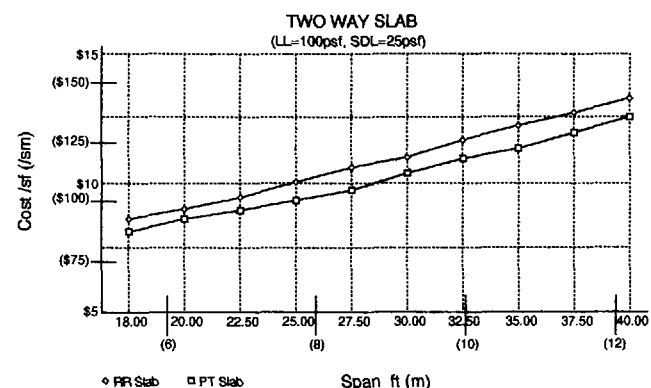


Fig. 6 — Cost versus span for RR and PT slabs.

were for a typical interior bay, since the exterior bay is normally affected by the particular building configuration and the exterior skin used. Also, square bays were assumed for the two-way slabs.

The unit cost values used for the comparison are shown in Table 2. The prices include material and labor for supplying and placing the components listed. These values are considered valid at the time of writing and for the geographical area with which the author is familiar.

The results of the comparisons for one- and two-way slabs were graphed, with cost per square foot versus span (Fig. 4 through 6), to provide the information in an easily digestible form.* The results consistently show that PT concrete slabs

offer a sizeable cost saving relative to their RR concrete counterparts.

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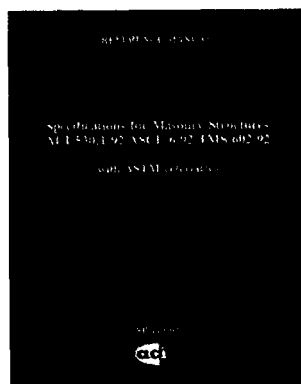
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Selected for reader interest by the editors.

*Unit quantity tables that allow comparisons based on individual experience and knowledge are available from the author at Prosum Engineering, Ltd., 56 Goddard St., North York, Ontario M3H 5E2, Canada.

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ELEVATED SLABS

Interactive Horizontal Formwork Selection System

by Awad S. Hanna and Victor E. Sanvido

In multistorey reinforced concrete building construction, selection of formwork to shape and support freshly cast concrete is largely based on the experience of the person selecting the system. There are typically one or two such experts in a company who make these decisions. This may often result in the selection of a particular system that is not the optimum. Also, the expert "Knowledge Czar" is not always available and an outside consultant is expensive to hire. This fact imposes some limitations especially for small and medium size contractors.

This article describes a computer system the authors developed to help the contractor select a horizontal formwork system. The expertise of several formwork engineer/planners was used to produce a body of decision rules and knowledge represented as an expert system.¹

Horizontal formwork systems

Horizontal formwork systems are those used to form the horizontal concrete work (slabs or floors). There are seven horizontal formwork systems that can be used to support slabs and roofs. Two of the systems — joist-slab and dome forms — are usually placed above the plywood sheathing. They are therefore considered special cases, because they can be used in conjunction with any of the five basic systems described below.^{1,2}

Conventional wood systems

Conventional wood systems consist of wood shores supporting wood stringers and joists, with the deck

surface made of plywood (Fig. 1). The deck may also be made of large panels tied or ganged together and supported by scaffold-type shoring.^{3,4}

Conventional metal systems

Conventional metal systems usually consist of wood or aluminum shores supporting aluminum stringers and joists, with the sheathing made of plywood (Fig. 2). The same type of deck forms can be made up of large panels tied or ganged together and supported by steel scaffold-type shoring.^{2,5}

Flying truss systems

Flying truss forms usually consist of large sheathing supported by steel or aluminum trusses (beams) that rest on screw jacks that allow the forms to be adjusted to the right level (Fig. 3). For stripping, after the concrete has gained enough strength, the system can be lowered away from the slab by turning down the jacks. The truss-mounted forms are then moved by crane from one casting position to the next.⁶

Column-mounted shoring systems

Formwork under cast-in-place slabs has typically been supported by shores resting on the floor or ground below. Another way to cast the slab without using the shoring system is to use long span form panels supported by up-and-down adjustable brackets attached to the bearing walls or columns of the building itself. The system consists of plywood decking that can be assembled on the ground or in place once the form has been flown and set on the columns (Fig. 4). The plywood decking is supported by

joists (headers) attached to the stringers (steel beams). For stripping, jacks are lowered until the framing beam of the deck rests on the rollers attached to the bracket and the entire form assembly is rolled out and carried by crane to the next floor.⁸

Tunnel formwork systems

Tunnel forms (also known as tunnels) are U-shaped or inverted L-shaped prefabricated steel forms that allow slab and supporting walls to be cast simultaneously (Fig. 5). After the concrete has gained enough strength the tunnels are removed and moved to the next level (Fig. 6). Half tunnels are used for spanning wide rooms, full tunnels for spanning narrow rooms. Standard modular panels offer a wide range of flexibility in room size. Forms range in width from 13 ft to 24 ft, in height from 7.5 ft to 10 ft, and in length up to 40 ft.^{9,10}

Joist-slab formwork systems

A one-way joist slab is a monolithic combination of regularly spaced joists arranged in one direction and a thin slab cast in place to form an integral unit with the beams and columns. One-way joist slabs have frequently been formed with standard steel pans. Any spacing between pans more than 30 in. is referred to as a wide-modular or skip-joist system.^{3,12}

Dome formwork systems

Standard size domes are usually used for waffle slab construction. They are based on two, three, four, and five foot modules.³ The two foot modules utilize 19 x 19 in. domes, with 5 in. ribs between