Tolerances for Precast and Prestressed Concrete

PCI Committee on Tolerances

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COMMITTEE STATEMENT

The PCI Committee on Tolerances has the mission of developing tolerances for the precast prestressed concrete industry. The final committee report consists of four chapters:

1. Introduction
2. Product Tolerances
3. Erection Tolerances
4. Interfacing Tolerances and Typical Details

This report has received considerable review. It has been distributed to all producer members of the PCI, and subsequent questionnaires have been distributed to all hollow-core slab producers so that this product could be handled appropriately. In the development of specific values for product tolerances, close coordination has been maintained with the PCI Plant Certification Committee. The report has received the full consensus review and approval procedure through the PCI Technical Activities Committee and the PCI Board of Directors.

Close coordination has been maintained with ACI Committee 117, Tolerances, and significant agreement exists between this report and the ACI Standard: ACI 117-81, "Standard Tolerances for Concrete Construction and Materials."

The publication of this report represents the first comprehensive industry consensus on the subject of tolerances for use with precast and prestressed concrete products. The use of this report, and resultant confirmation of the information given here, will result in better information regarding actual practice with regard to tolerances. Thus, the recommendations given here should always be used in association with good engineering judgment.

PREFACE

The PCI Committee on Tolerances was formed in early 1977 to develop dimensional tolerances for precast and prestressed concrete including fabrication, erection, and relationships to other construction materials such as masonry, cast-in-place concrete, and structural steel.

The committee recognizes the importance of "feedback" from all categories of the PCI membership. The value of this report will be significantly enhanced through review and evaluation by interested producers, erectors, and designers who design, fabricate, and erect precast products in accordance with the recommendations and principles given here.

Many plants already record dimensions of selected precast members as part of their normal post-pour inspection. In most cases, these records are limited to check marks on a quality control form. A record of actual variations from working dimensions, when compared to the proposed tolerances in this report, would give the industry the basis of an optimum monitoring system. Continuing assessment of such a monitoring system would assure that this report always reflects the latest state-of-the-art conditions, and valuable data for improving quality and productivity would be gained.

Comments and recommendations for revision of this report are invited and encouraged and should be forwarded to the Technical Director, Prestressed Concrete Institute, 201 North Wells, Chicago, Illinois 60606. This information will be reviewed on a continuing basis and will become the basis for revisions to this report.

CONTENTS

CHAPTER 1 — INTRODUCTION

This report is intended to be a working reference for the dimensional control of precast concrete construction. It is intended to cover plant-cast or site-cast precast and precast prestressed concrete. It is designed to improve dimensional control in precast concrete products and construction.

The information contained in this report should be used by architects, engineers, general contractors, precast and precast prestressed concrete producers, erectors, quality control agencies, and related or interfacing trades.

Final component details should conform to groups of tolerances which have been established as part of precast concrete design. These are:

(1) Product Tolerances

Product tolerances are defined as those tolerances relating to the dimensions and dimensional relationships of the individual precast concrete members. Chapter 2 provides a comprehensive compilation of recommended product tolerances for various product types and discusses the specification of these values and the methods to verify tolerances after casting.

(2) Erection Tolerances

Erection tolerances are defined as those tolerances which are required for the acceptable matching of the precast members after they are erected. Chapter 3 provides (with detailed examples) a comprehensive discussion of the principles and considerations relative to precast concrete erection tolerances. Information pertaining to erection tolerances that must be anticipated in the design and construction of precast concrete structures is also provided.

(3) Interfacing Tolerances

Interfacing tolerances are those tolerances which are associated with other materials in contact with or in close proximity to precast concrete, both before and after precast erection. Chapter 4 provides guidelines for the proper dimensional specification of interfacing materials using precast product and erection tolerances. The principles and considerations which apply to interfacing tolerance design are discussed and examples are provided to demonstrate the effects of tolerances of interfacing materials on precast design and detailing.

Tolerances are required for the three categories above for the following reasons:

(1) Structural

To ensure that structural design properly accounts for factors sensitive to variations in dimensional control. Examples include eccentric loading conditions, bearing areas, hardware and hardware anchorage locations, and locations of reinforcing or prestressing steel.

(2) Feasibility

To ensure acceptable performance of joints and interfacing materials in the finished structure.

(3) Visual

To ensure that the variations will be controllable and result in an acceptable looking structure.

(4) Economic

To ensure ease and speed of production and erection by having a known degree of accuracy in the dimensions of precast products.

(5) Legal

To avoid encroaching on building lines and to establish a tolerance standard against which the work can be compared in the event of a dispute.

(6) Contractual

To establish a known acceptability range and also to establish responsibility for developing, achieving, and maintaining mutually agreed tolerance values.

1.1 Responsibility

The assignment of responsibility for dimensional control of a project varies both geographically and from company to company. The conceptual design of a precast project is the place to begin consideration of dimensional control.

Architectural and structural concepts should be developed with the practical limitations of dimensional control in mind. The established tolerances or required performance should fall within generally accepted limits and should not be made more rigid, and therefore more costly, than is absolutely necessary.

While the detailed assignment of responsibility for the dimensional tolerancing and control of the various elements may vary, depending upon the contractual arrangement for a particular project, it is very important that these responsibilities be clearly assigned and that these assignments be communicated to all members of the project team.

Once the tolerances for the various members have been specified, and appropriate connection details which consider those tolerances have been designed, the production of the elements must be organized to ensure that the specified tolerances are recognized and tolerance compliance is verified during the element fabrication process. An organized quality control program with a strong dimensional tolerance control focus is a necessary part of the production effort.

In the erection phase of the project, the various members must be assembled in accordance with the established erection tolerances. An erection quality assurance effort will include a clear definition of responsibilities for tolerance verification and adjustment, if necessary, of both the erected precast concrete structure and any interfacing structure.

1.2 Tolerance Acceptability Range

It should be understood by those involved in the design and construction process that tolerances shown in this report must be considered as guidelines for an acceptability range and not limits for rejection.

If these tolerances are met, the member should be accepted. If these tolerances are exceeded, the member may be accepted if it meets any of the following criteria:

(a) Exceeding the tolerances does not affect the structural integrity or architectural performance of the member.

(b) The member can be brought within tolerance by structurally and architecturally satisfactory means.

(c) The total erected assembly can be modified to meet all structural and architectural requirements.

1.3 Definitions

Tolerance — The definition can include:

(a) The permitted variation from a basic dimension or quantity, as in the length or width of a member.

(b) The range of variation permitted in maintaining a basic dimension, as in an alignment tolerance.

(c) A permitted variation from location or alignment.

Variation — The difference between the actual and the basic dimension. Variations may be either negative (less) or positive (greater).

Clearance — Interface space between two items. Normally specified to allow for tolerance and for anticipated movement.

Basic Dimension — The dimensions shown on the contract drawings or called for in the specifications. The basic dimension applies to size, location, and relative location. It may also be called the "nominal" dimension.

Working Dimension — The planned dimension of the member obtained from both its basic dimension and joint or clearance dimensions. It is to this planned dimension that the product tolerance is applied. For example, if a nominal 8 ft (2.44 m) wide double tee is designed to have a nominal ¾ in. (19 mm) width joint on either side, the working dimension for member width would be 7 ft 11¼ in. (2.42 m).

Actual Dimension — The measured dimension of the member after casting. This dimension may differ from the working dimension due to construction and material induced variation.

Alignment Face — The face of a precast element which is to be set in alignment with the face of adjacent elements or features.

Primary Control Surface — A surface on a precast element, the dimensional location of which is specifically set and controlled in the erection process. Clearance is generally allowed to vary so that the primary control surface can be set within tolerance.

Secondary Control Surface — A surface on a precast element, the dimensional location of which is dependent on the location tolerance of the member primary control surfaces plus the member feature tolerances. An example would be the elevation of a second-story corbel on a multistory column whose first-story corbel is selected as the primary elevation control surface.

Feature Tolerance — The locational or dimensional tolerance of a feature, such as a corbel or a blockout, with respect to the overall member dimensions.

1.4 Relationships Between the Different Tolerances

Product tolerances define the limits of the size and shape of the individual precast members which compose the building or structure.

Erection tolerances control the position of the individual precast members as they are located and placed in the assembled structure. The individual precast member is erected and positioned so that its primary control surface is in conformance with the established erection tolerances.

Product tolerances are not additive to the erection tolerances which govern the setting of the member primary control surfaces.

The secondary control surfaces of a member often are not directly set during the erection process. Thus, the product tolerances for secondary control surfaces and features of the member *are* additive to the erection tolerances for the member.

Since erection tolerances and product tolerances for some features of a precast concrete member may be directly additive, it must be made clear to the erector which member surfaces are the primary control surfaces used to control the erection. In instances where the tolerance of both primary and secondary control surfaces must be controlled during erection, the design details should include provisions for adjustment to achieve this. Surface and feature control requirements should also be clearly outlined in the plans and specifications.

Interfacing tolerances are those associated with other materials or systems which interface with the precast concrete elements. Interfacing tolerances apply whether the interfacing system is erected prior to or following precast erection.

For interfacing situations which involve multiple members, both product and erection tolerance effects may have to be accommodated within the interface tolerance for the system being interfaced.

Product tolerances, erection tolerances, and interface tolerances together determine the dimensions of the completed structure. Which tolerance takes precedence on any given project is a question of economics, which should be addressed by considering fabrication, erection, and interfacing cost implications.

For example, standard window elements have associated installation details which require a certain tolerance on window openings in a precast panel. If the opening is completely contained within one panel, can the required tolerances on the window opening be economically met? If not, is it less expensive to the project to procure special windows or to incur the added cost associated with making the tolerances on the window opening more stringent?

If the window opening is common to two or more panels, the cost of erecting the panels to achieve required window interface tolerances must also be considered.

The precedence of product and erection tolerances raises similar questions of project economics. In this instance, the tolerance requirements and other costs associated with the connection details also must be considered. The architect/engineer must evaluate the costs associated with each of the three tolerances, paying special attention to unique and unusual situations, difficult erection requirements, connections which are very tolerance sensitive, and product or erection tolerances which are more stringent than normal practice.

Special tolerances or construction procedures require early decisions based on overall project economics. Once these decisions have been made, they should be reflected in the project plans and specifications. All special tolerance requirements, special details, and special procedures should be clearly spelled out in the specifications. The plans and specifications then define the established tolerance priority to be adhered to on the project.

CHAPTER 2 — PRODUCT TOLERANCES

Product tolerances are those within which precast members should be made. They are a measure of dimensional accuracy of the individual members and ensure, prior to delivery, the probability that the member will fit into the structure without difficulty.

The architect/engineer must specify product tolerances or require performance within a generally accepted range. In any case, he must account for the function of the member in the construction, its fit in the construction, and the compatibility of the member tolerances to those of interfacing materials (see Chapter 4). Tolerances for manufacturing are standardized throughout the industry and should not be made more rigid, and therefore more costly, unless absolutely necessary.

It must be emphasized (see Chapter 1) that the tolerances listed in this chapter are to be used only as an acceptability range and not limits for rejection. These tolerances are generally based on a degree of accuracy which is practical and achievable while preventing costs from becoming prohibitive.

Several considerations should be taken into account in the establishment of product tolerances for a specific project. The two most important relate to the type of forms that are used and the measuring techniques used in the manufacturing process.

Tolerances are normally established by economical and practical production considerations of how the member must fit into the overall construction and its relationship to adjacent units. The cost of manufacturing to close tolerances decreases with increased repetition. These factors, when coupled with the individual skill of the craftsman, will determine the final degree of accuracy. It must be noted that the particular selection of casting forms and measuring techniques is often based on economic and functional reasons rather than on the manufacturer's capability to follow the most sophisticated methods.

To further explain this concept, the following are typical items for which dimensional tolerances are established.

2.1 Overall Plan Dimensions

2.1.1 Effect of Forms on Dimensions — Forms are generally of three types (as shown in Fig. 2.1): rigid, semi-rigid, and flexible.

Rigid forms have all the sides fixed and ensure a higher degree of accuracy than other form types, in both the length and width directions. These forms are often used in the fabrication of customized products, such as architectural precast panels, where appearance or function dictates the need for closer tolerances and where repetitive use makes the initial cost of the form economically viable. Side forms for rigid forms must have suitable draft. Draft is the slope or taper required on the forms to permit stripping of the precast member.

Semi-rigid forms have two sides rigidly fixed. The other sides are made by using end dividers for long line casting or removable side forms for individual pieces. These end dividers or removable side forms are not rigidly attached to the form, and thus may move slightly during the placement and vibration of the concrete. This results in a somewhat lesser degree of precision in linear plan dimensions than when rigid forms are used.

Flexible forms have no rigidly fixed sides. The typical product using such forms is a double tee with blocked-out flanges. Of the three types of forms discussed here, flexible forms result in the least degree of plan dimensional precision in either length or width.

2.1.2 Effects of Prestressing on Dimensions—The application of prestress force to the member can affect the overall length of the member in two ways. First, there is an axial shortening of the member and second, the ends of the member may rotate as a result of member camber. Both of these effects result in length changes which must be taken into account when determining the casting length.

2.1.3 Measuring Techniques—The most common measuring method is the use of metallic measuring tapes. Again, for economic and functional reasons, the use of more sophisticated measuring instruments is almost nonexistent.

The degree of accuracy in using tapes depends on the particular dimension of the member being measured. The greater the linear dimension, the lower is the degree of accuracy. This is due to the fact that in practice, slope, tape sag, and temperature effects are seldom taken into account. To maximize accuracy, members should not be measured in increments, which creates the possibility of a cumulative error.

2.2 Blockouts

A tolerance for both the size and location of the blockout should be considered. It also is important that the eventual function of the blockout be properly considered. For example, the tolerance on a window blockout, into which a prefabricated frame will fit, must be more rigid than a blockout through which a field-installed piping system will be placed. The possible need for draft on the sides of blockouts must also be considered.

2.3 Sweep or Horizontal Alignment

Horizontal misaligment usually occurs as a result of form tolerances and member width tolerances. It can also result from prestressing with lateral eccentricity, thus causing a sweep in the member. Such prestressed induced sweep should be a consideration of design. Joint design should accommodate any dimensional variance resulting from this condition.

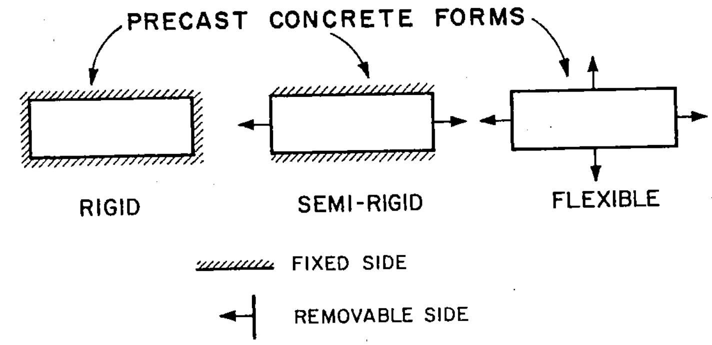

Fig. 2.1. Types of forms.

2.4 Position of Tendons

It is common practice to use ⅝ in. (16 mm) diameter holes in end dividers (bulkheads, headers) for all strand sizes — ½, 7/16, ⅜ in. (13, 11, and 9.5 mm) diameters — since it is costly to switch to new end dividers for different strand diameters. Consequently, better precision is achieved when using larger diameter strands.

2.5 Handling Devices

To facilitate the concrete finishing process, handling devices are often embedded after the placement of concrete. It is important that these devices be embedded as soon as possible. However, it is also necessary to recognize the importance of placing handling device locations in different directions, especially in thin or narrow sections. For example, closer lateral tolerances are necessary to ensure the minimum required cover around the lifting device in the stems of tees.

2.6 Camber and Differential Camber

Some of the factors affecting camber variation and subsequent differential camber are:

(1) Design

(a) Members with the same strand pattern and same length but with special width, skewed ends, or blockouts usually have different cambers.

(b) Adjacent members with different support spans will have substantially different cambers.

(c) Members with a high span/depth ratio are more sensitive to camber variations resulting from all of the mentioned effects.

(2) Time-Dependent Effects

(a) Modulus of elasticity variations due to curing or concrete differences

(b) Different ages of adjacent members at time of erection

(c) Creep differences

(d) Shrinkage effects due to exposure or curing differences

(e) Strand relaxation

(3) Tolerance of Strand Location

(4) Curing Methods

Curing methods can influence concrete strength (and consequent modulus of elasticity) at time of prestress transfer to the member.

(5) Storage Configuration

(a) Member support locations while in storage

(b) Member position with respect to sun

(c) Member position in the storage stack and its effect on the storage loading of the member

If a significant variation in camber is observed, one should look for the cause and determine the effect of the variation on member performance. If differential cambers exceed recommended tolerances, additional effort is often required to erect the members in a manner satisfactory for the intended use.

The final installed differential elevation tolerance between two adjacent cambered members erected in the field may be the combined result of member differential cambers, variations in support elevations, and any adjustments made to members during erection.

It is very important to maintain uniformity at the time of camber measurement. For example, the camber measured on the top member of a stack of double tees in midafternoon on a hot sunny day will be considerably different from the camber of the bottom member or of the same top piece on a cold damp day. The most useful results are obtained by measuring camber in the early hours of the day, before the sun has begun to warm the members, and checking all elements at the same age.

2.7 Squareness of Ends or Variation From Specified End Skew

Squareness of ends will depend greatly on the type of forms used, as previously explained. An end skew which is specified as different from 90 degrees is considerably more difficult to control than is a 90 degree end skew.

2.8 Position of Weld Plates

In general, it is easier to hold plates to closer tolerances at the bottom of the member (or against the side form) than with plates cast on top of the member. The main reason is that bottom and side plates can be fastened to the form and hence are less susceptible to movement caused by vibration.

2.9 Tipping and Flushing of Weld Plates

For the same reasons as outlined above, plates cast on the top of members will tend to tip more than bottom plates. Another reason for a tolerance difference between top and bottom plates is that bottom plates get uniform bearing from the form surface whereas top plates must be supported by removable positioning fixtures which often are not an integral part of the form. Flushness is the relationship of the weld plate surface to the concrete surface.

2.10 Haunches of Columns and Wall Panels

The importance of maintaining tolerance of haunch location dimensions is usually a function of the type of connection used at the base of the member. Since this type of connection most often allows some flexibility, it is usually more important to control dimensions from haunch to haunch in multitiered columns rather than to maintain tight control of actual haunch location dimensions from the end of the member.

2.11 Location of Sleeves Cast in Prestressed Products

This tolerance may be affected by slight relocations of the sleeves necessitated by the location of prestressing strands. For horizontal sleeves, consideration must be given to the location of depressed strands. The sleeve location tolerance is secondary to the location of the strands in most instances.

2.12 Reinforcing Steel Tolerance

Reinforcing steel used in precast prestressed concrete products is controlled by two tolerances. The first is the bar length and bending tolerance, and the second is the bar placement tolerance, which is to an extent dependent on the bar bending tolerance.

Reinforcing steel placement tolerances are controlled by ACI Committee 117. For purposes of reference, the section on placement (Section 6.1.2 from ACI 117-81, "Standard Tolerances for Concrete Construction and Materials") is reproduced below.

6.1	**— Reinforcing Steel** (Refer to ACI 318, 349, 301, 315, 531, 543)	
6.1.2	Placement for flexural members, walls, and compression members	
6.1.2.1	Tolerance in clear distance to formed soffit	−¼ in. (−6 mm)
6.1.2.2	Tolerance in depth "d" (where d is the distance from the extreme compression fiber to the centroid of tension reinforcement)	
	Depth "d"	
	8 in. (203 mm) or less	±⅜ in. (±9.5 mm)
	More than 8 in. (203 mm)	±½ in. (±13 mm)
6.1.2.3	Tolerance on minimum concrete cover	
	Depth "d"	
	8 in. (203 mm) or less	−⅜ in.* (−9.5 mm)
	More than 8 in. (203 mm)	−½ in.* (−13 mm)
6.1.2.4	Tolerance on minimum distance between bars	−¼ in. (−6 mm)
6.1.2.5	Tolerance in uniform spacing of reinforcement from theoretical location	±2 in. (±51 mm)
6.1.2.6	Tolerance in uniform spacing of stirrups and ties from theoretical location	±1 in. (±25 mm)
6.1.2.7	Tolerance in longitudinal location of bends and ends of bars	
	General	±2 in. (±51 mm)
	Discontinuous ends of members	±½ in. (±13 mm)
6.1.2.8	Tolerance in length of bar laps	−1½ in. (−38 mm)
6.1.2.9	Tolerance in embedded length	
	#3 to #11	−1 in. (−25 mm)
	#14 and #18	−2 in. (−51 mm)
6.1.3	Placement of prestressing steel and prestressing steel ducts in precast element (refer to ACI 301, 318, 349, 543)	
6.1.3.1	Tolerance on depth "d" (see Section 6.1.2.2)	
6.1.3.2	Slabs	
	Horizontal tolerance in any 15 ft (4.57 m) of tendon length	±1 in. (±25 mm)
6.1.3.3	Tolerance for post-tensioning anchor bearing plate concentricity and perpendicularity with tendons and concrete	±1 deg

*But not to exceed one-third specified concrete cover.

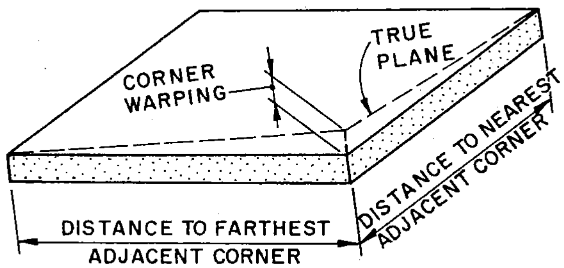

Fig. 2.2. Warping definitions for panels.

2.13 Position of Deflection Points

Strand hold-downs and other strand deflection devices frequently have their positions dictated by being fixed, either to the form itself or to the form support, so that suitable strand hold-down capacity is achieved. This can frequently result in positions being as much as ±20 in. (±510 mm) [for a 40 in. (1020 mm) lattice spacing on form] from the intended position. Cantilevers and other special conditions may require more precise placement of the deflection points, which is reflected in increased cost. The vertical position of the hold-down is more important than the longitudinal position.

2.14 Warping, Bowing, and Local Smoothness of Panels

While warping and bowing occur in both structural and architectural members, it is architectural members that demand special consideration. Warping and bowing tolerances have an important effect on edge matchup during erection and on the visual appearance of the erected panels, both individually and when viewed together.

Warping is generally an overall variation from planeness in which the corners of the panel do not all fall within the same plane. Warping tolerances are stated in terms of the magnitude of the corner variation, as shown in Fig. 2.2. This value is usually stated in terms of the allowable variation per foot of distance from the nearest adjacent corner with a not-to-exceed maximum value of corner warping.

Bowing is an overall out-of-planeness condition which differs from warping in that while two edges of the panel may fall in the same plane, the portion of the panel between the edges is out of plane. Several possible bowing conditions are shown in Fig. 2.3.

Differential temperature effects and differential moisture absorption between the inside and outside faces of a panel, and possible prestress eccentricity, should be considered in design to minimize bowing and warping.

Note that bowing and warping tolerances are of interest at the time the panel is erected. Careful attention to pre-erection storage of panels is necessary, since storage conditions can be an important factor in achieving and maintaining panel bowing and warping tolerances.

Differential bowing is a consideration for panels which are viewed together on

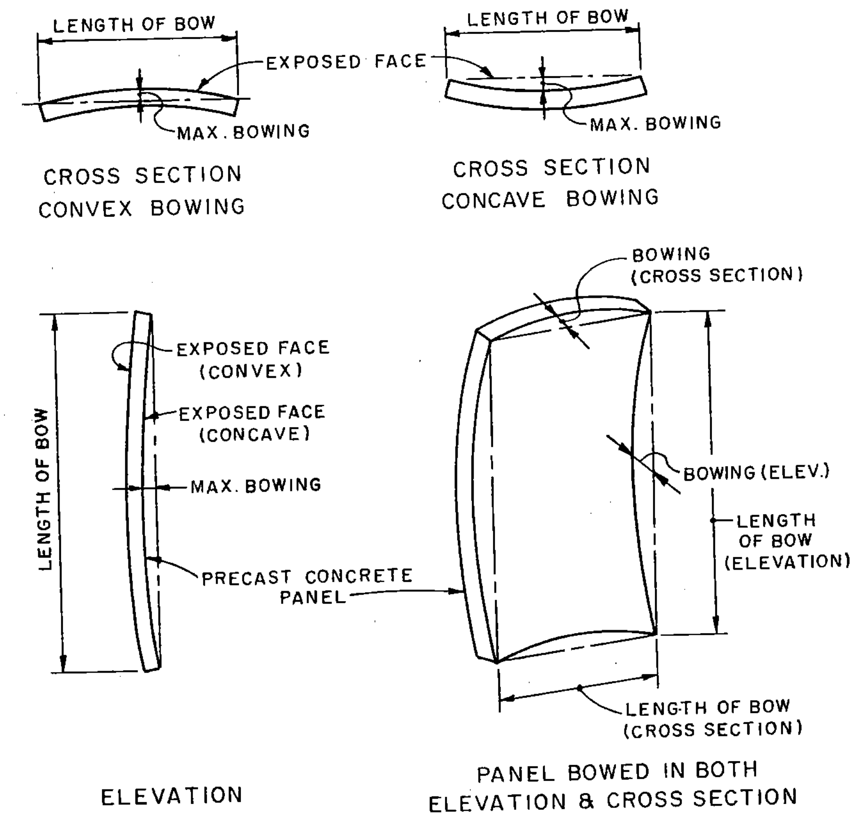

Fig. 2.3. Bowing definitions for panels.

the completed structure. If we say that convex bowing is positive (+) and concave bowing is negative (−), then the magnitude of differential bowing can be determined by adding the bowing values. For example, in Fig. 2.4, if the maximum bowing of Panel 3 was +¼ in. (6 mm) and the maximum bowing of Panel 4 was −¼ in. (6 mm), then the differential bowing between these two adjacent panels is ½ in. (13 mm).

Surface out-of-planeness, which is not a characteristic of the entire panel shape, is defined as a local smoothness

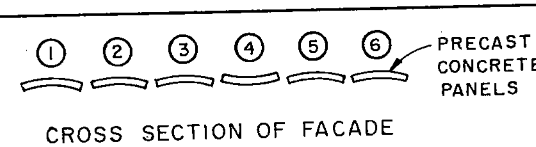

Fig. 2.4. Differential bowing of panels.

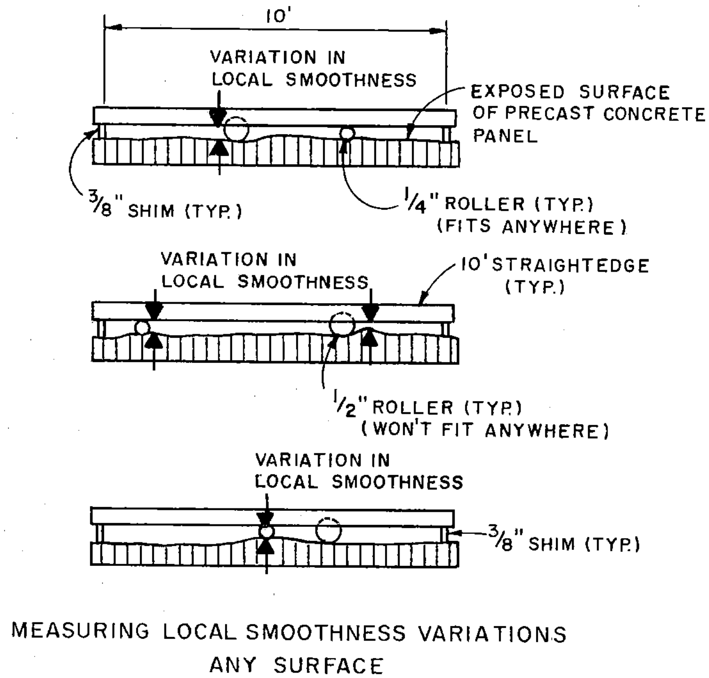

Fig. 2.5. Local smoothness variations.

variation rather than a bowing variation. Examples of local smoothness variation are shown in Fig. 2.5. The tolerance for this type of variation is usually expressed in inches per 10 ft (mm/m). The tolerance is usually checked with a 10 ft (3 m) straightedge or the equivalent, as shown in Fig. 2.5.

Fig. 2.5 also shows how to determine if a surface meets a tolerance of ¼ in. (6 mm) as measured beneath a 10 ft (3 m) straightedge. A ¼ in. (6 mm) diameter by 2 in. (50 mm) long roller should fit anywhere between the straightedge and the element surface being measured when the straightedge is supported at its ends on ⅜ in. (9.5 mm) shims as shown. The ½ in. (13 mm) diameter by 2 in. (50 mm) long roller should not fit between the surface and the straightedge.

The likelihood of a panel to bow or warp is dependent, to a considerable extent, on the design of the panel and its relative stiffness or ability to resist deflection as a plate member. Panels which are relatively thin in cross section, when compared to their overall plan dimensions, are more likely to warp or bow as a result of a number of design, manufacturing, and environmental conditions.

Table 2.1 represents a relationship between overall flat panel dimensions and cross-sectional thicknesses below which suggested warping tolerances should be reviewed for possible increase.

Note that the thickness values in this table should not be considered as limiting values but rather only as an indication that more detailed consideration of the possible magnitudes of

Table 2.1. Guidelines for panel thicknesses for overall panel stiffness consistent with suggested normal panel bowing and warping tolerances.

Panel dimensions	8 ft	10 ft	12 ft	16 ft	20 ft	24 ft	28 ft	32 ft
4 ft	3 in.	4 in.	4 in.	5 in.	5 in.	6 in.	6 in.	7 in.
6 ft	3 in.	4 in.	4 in.	5 in.	6 in.	6 in.	6 in.	7 in.
8 ft	4 in.	5 in.	5 in.	6 in.	6 in.	7 in.	7 in.	8 in.
10 ft	5 in.	5 in.	6 in.	6 in.	7 in.	7 in.	8 in.	8 in.

Note: 1 ft = 0.305 m; 1 in. = 25.4 mm.

warping and bowing is warranted.

For ribbed panels, the equivalent thickness relative to Table 2.1 should be the overall thickness of the ribs, if they run continuous from one end of the panel to the other. Similarly, panels which are manufactured using large aggregate concrete mixes [above ¾ in. (19 mm) aggregate], or units which are fabricated from nonhomogeneous materials such as two significantly different concrete mixes, special veneers, insulating mediums, and the like, require more careful consideration.

The major criteria for maintaining or relaxing bowing and warping tolerances will be the appearance requirements, the required type and spacing of connections, and the advice of the local precaster regarding overall economic and construction feasibility.

2.15 Special Considerations

The function and the manufacturing technique used to produce certain elements warrants special considerations. Architectural panels, for example, require more rigid tolerances due to the visual considerations in the final construction. For this reason, they are usually manufactured as purely architectural panels with a high degree of precision.

In the context of tolerances, "architectural panel or member" refers to the class of tolerances specified and not necessarily to the member's use in the final structure.

Double tees and hollow-core slabs are often used for "architectural" wall panels. The same high degree of accuracy for these panels should not be expected since the manufacturing technique is the same as for the structural product for which these elements are more commonly used. If more rigid tolerances are actually required, special emphasis in the plans and specifications must reflect both this need and the subsequent higher manufacturing cost anticipated.

While statistical concepts are not usually employed in the dimensional control of precast products, it may be advantageous for producers to consider this approach for at least two reasons. First, a random sampling of specific measurements and subsequent statistical analysis of these measurements can create economies on projects which have a large number of identical pieces with stringent tolerance requirements. Concrete railroad tie production is an example of such an instance.

Second, statistical concepts can be used in developing the tolerance control requirements for specific production operations. By proper sampling and analysis, one can determine more precisely which type of measurements require more attention and this can be implemented in the quality control program. For information regarding this type of statistical approach, one should refer to appropriate publications.[4,5]

2.16 Product Tolerances

The following pages show dimensional requirements for a range of standard precast concrete and precast, prestressed concrete products. It must be emphasized that these are guidelines only and that each project must be considered individually to ensure that the tolerances shown are applicable.

2.16.1 DOUBLE TEES (SEE FIG. 2.6)

a = Length .... ±1 in. (±25 mm)
b = Width (overall) .... ±¼ in. (±6 mm)
c = Depth .... ±¼ in. (±6 mm)
d = Stem width .... ±⅛ in. (±3 mm)
e = Flange thickness .... +¼ in., –⅛ in. (+6 mm, –3 mm)
f = Distance between stems .... ±¼ in. (±6 mm)
g = Stem to edge of top flange .... ±¼ in. (±6 mm)
h = Variation from specified flange squareness or skew .... ±⅛ in. per 12 in. width, ½ in. max (±3 mm per 300 mm width, 13 mm max)
i = Variation from specified end squareness or skew
  Greater than 24 in. (600 mm) depth ... ±⅛ in. per 12 in., ±½ in. max (±3 mm per 300 mm, ±13 mm max)
  24 in. (600 mm) or less depth .... ±¼ in. (±6 mm)
j = Sweep (variation from straight line parallel to centerline of member)
  Up to 40 ft (12 m) member length .... ±¼ in. (±6 mm)
  40 to 60 ft (12 to 18 m) member length .... ±⅜ in. (±9.5 mm)
  Greater than 60 ft (18 m) member length .... ±½ in. (±13 mm)
k = Camber variation from design camber .... ±¼ in. per 10 ft, ±¾ in. max* (±6 mm per 3 m, ±19 mm max)
l = Differential camber between adjacent members of the same design .... ¼ in. per 10 ft, ¾ in. max* (6 mm per 3 m, 19 mm max)
m = Position of tendons
  Individual .... ±¼ in. (±6 mm)
  Bundled .... ±½ in. (±13 mm)
n = Position from design location of deflection points for deflected strands†
o = Position of blockouts .... ±1 in. (±25 mm)
p = Size of blockouts .... ±½ in. (±13 mm)
q = Position of plates .... ±1 in. (±25 mm)
r = Position of bearing plates .... ±½ in. (±13 mm)

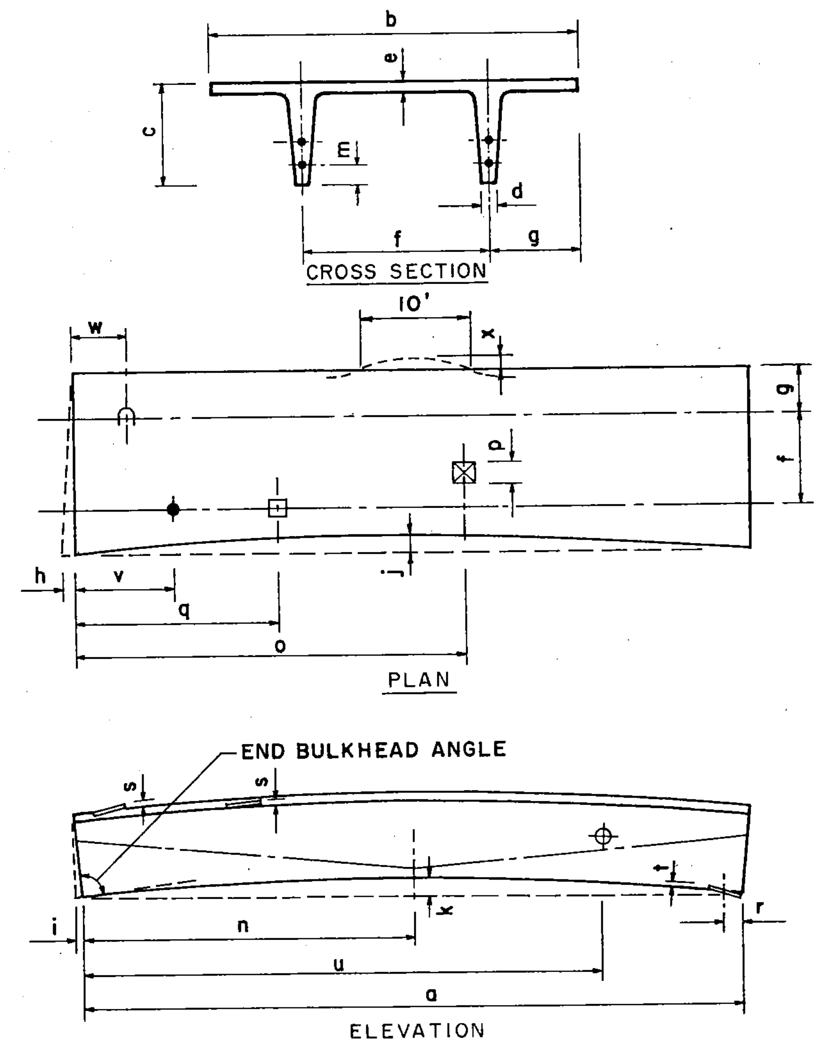

Fig. 2.6. Double tees.

*For members with a span-to-depth ratio approaching or exceeding 30, the stated camber tolerance may not apply. If the application requires control of camber to this tolerance in beams of such span-to-depth ratios, special premium production measures may be required. This requirement should be discussed in detail with the producer.

†The economical location of strand deflection points depends in large measure on the individual bed characteristics. Use of a large location tolerance for this item is often possible with little design consequence. Location tolerances on the order of ±20 in. (±510 mm) will provide benefits of economy.

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s = Tipping and flushness of plates .... ±¼ in. (±6 mm)
t = Tipping and flushness of bearing plates .... ±⅛ in. (±3 mm)
u = Position of sleeves cast in stems .... ±1 in. (±25 mm) in both horizontal and vertical plane
v = Position of inserts for structural connections .... ±½ in. (±13 mm)
w = Position of handling devices
  Parallel to length .... ±6 in. (±150 mm)
  Transverse to length .... ±1 in. (±25 mm)
x = Local smoothness any surface .... ¼ in. in 10 ft (6 mm in 3 m)
  Does not apply to top deck surface left rough to receive a topping or to visually concealed surfaces. (Refer to Fig. 2.5 for definition.)

2.16.2 SINGLE TEES (SEE FIG. 2.7)

a = Length .... ±1 in. (±25 mm)
b = Width (overall) .... ±¼ in. (±6 mm)
c = Depth .... ±¼ in. (±6 mm)
d = Stem width .... ±¼ in. (±6 mm)
e = Flange thickness .... +¼ in., –⅛ in. (+6 mm, –3 mm)
f = Stem to edge of top flange .... ±¼ in. (±6 mm)
g = Variation from specified flange squareness or skew .... ±⅛ in. per 12 in., ½ in. max (±3 mm per 300 mm, 13 mm max)
h = Variation from specified end squareness or skew
  Greater than 24 in. (600 mm) depth .. ±⅛ in. per 12 in., ±½ in. max (±3 mm per 300 mm, ±13 mm max)
  24 in. (600 mm) or less depth .... ±¼ in. (±6 mm)
i = Sweep (variation from straight line parallel to centerline of member)
  Up to 40 ft (12 m) member length .... ±¼ in. (±6 mm)
  40 to 60 ft (12 to 18 m) member length .... ±⅜ in. (±9.5 mm)
  Greater than 60 ft (18 m) member length .... ±½ in. (±13 mm)
j = Camber variation from design camber .... ±¼ in. per 10 ft, ±¾ in. max* (±6 mm per 3 m, ±19 mm max)

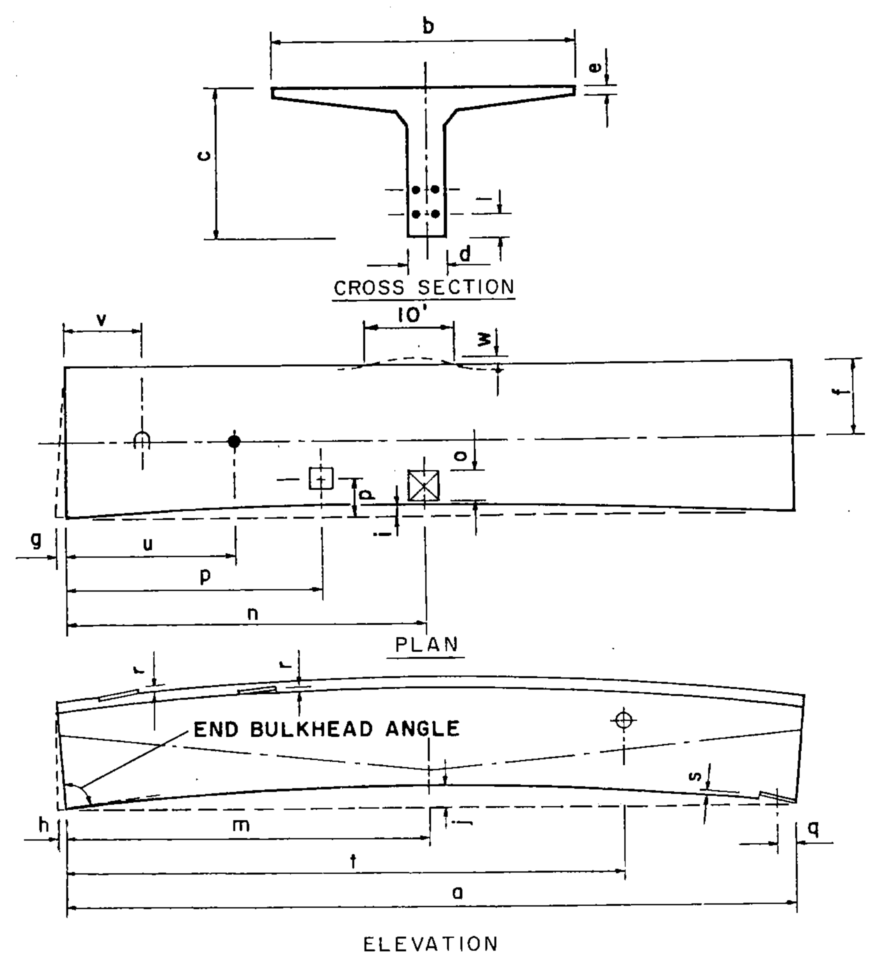

Fig. 2.7. Single tees.

k = Differential camber between adjacent members of the same design .... ¼ in. per 10 ft, ¾ in. max* (6 mm per 3 m, 19 mm max)

l = Position of tendons
  Individual .... ±¼ in. (±6 mm)
  Bundled .... ±½ in. (±13 mm)

*For members with a span-to-depth ratio approaching or exceeding 30, the stated camber tolerance may not apply. If the application requires control of camber to this tolerance in beams of such span-to-depth ratios, special premium production measures may be required. This requirement should be discussed in detail with the producer.

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m = Position from design location of deflection points for deflected strands†

n = Position of blockouts .... ±1 in. (±25 mm)

o = Size of blockouts .... ±½ in. (±13 mm)

p = Position of plates .... ±1 in. (±25 mm)

q = Position of bearing plates .... ±½ in. (±13 mm)

r = Tipping and flushness of plates .... ±¼ in. (±6 mm)

s = Tipping and flushness of bearing plates .... ±⅛ in. (±3 mm)

t = Position of sleeves cast in stems .... ±1 in. (±25 mm) in both horizontal and vertical plane

u = Position of inserts for structural connections .... ±½ in. (±13 mm)

v = Position of handling devices
  Parallel to length .... ±6 in. (±150 mm)
  Transverse to length .... ±1 in. (±25 mm)

w = Local smoothness any surface .... ¼ in. in 10 ft (6 mm in 3 m)
  Does not apply to top deck surface left rough to receive a topping or to visually concealed surfaces. (Refer to Fig. 2.5 for definition.)

†The economical location of strand deflection points depends in large measure on the individual bed characteristics. Use of a large location tolerance for this item is often possible with little design consequence. Location tolerances on the order of ±20 in. (±510 mm) will provide benefits of economy.

2.16.3 BUILDING BEAMS AND SPANDRELS (SEE FIG. 2.8)

a = Length .... ±¾ in. (±19 mm)

b = Width (overall) .... ±¼ in. (±6 mm)

c = Depth (overall) .... ±¼ in. (±6 mm)

d = Depth (ledge) .... ±¼ in. (±6 mm)

e = Stem width .... ±¼ in. (±6 mm)

e_1 = Ledge width .... ±¼ in. (±6 mm)

f = Sweep (variation from straight line parallel to centerline of member)
  Up to 40 ft (12 m) member length .... ±¼ in. (±6 mm)
  40 to 60 ft (12 to 18 m) member length .... ±½ in. (±13 mm)
  Greater than 60 ft (18 m) member length .... ±⅝ in. (±16 mm)

g = Variation from specified end squareness or skew .... ±⅛ in. per 12 in. depth, ±½ in. max (±3 mm per 300 mm depth, ±13 mm max)

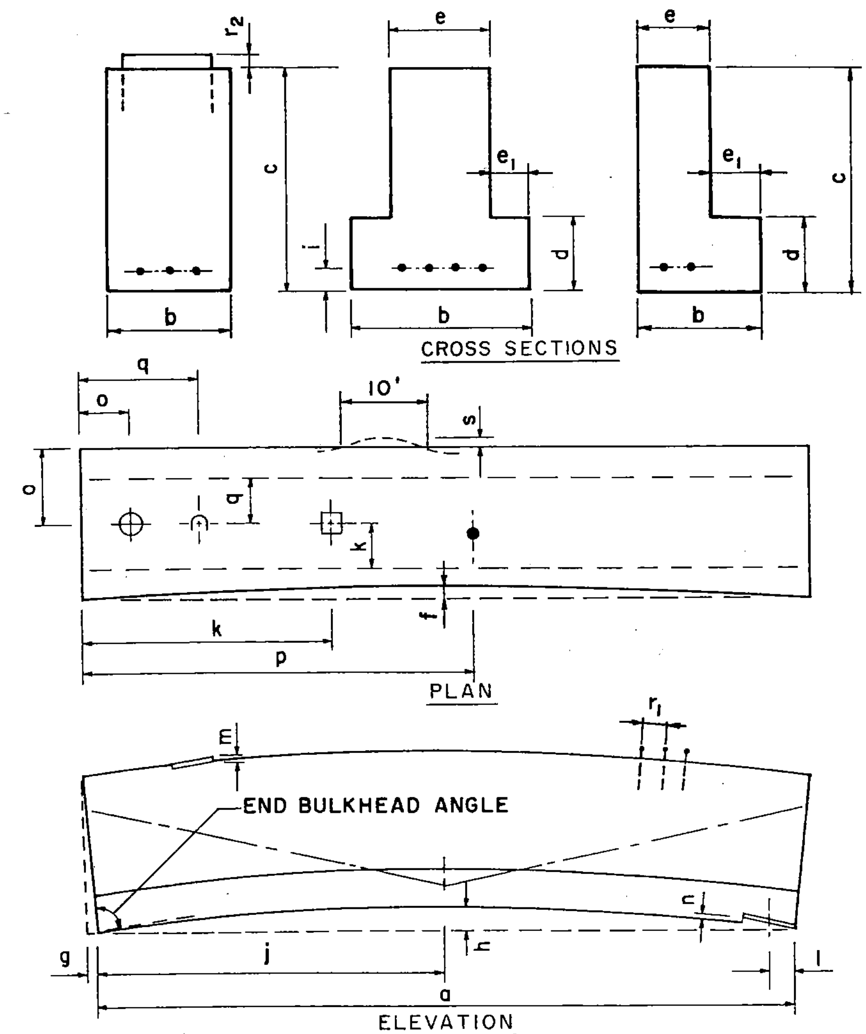

Fig. 2.8. Building beams and spandrels.

h = Camber variation from design camber .... ±⅛ in. per 10 ft, ±¾ in. max* (±3 mm per 3 m, ±19 mm max)

*For members with a span-to-depth ratio approaching or exceeding 30, the stated camber tolerance may not apply. If the application requires control of camber to this tolerance in beams of such span-to-depth ratios, special premium production measures may be required. This requirement should be discussed in detail with the producer.

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i = Position of tendons
  Individual .... ±¼ in. (±6 mm)
  Bundled .... ±½ in. (±13 mm)

j = Longitudinal position from design location of deflection points for deflected strand
  Member length 30 ft (9 m) or less .... ±6 in. (±150 mm)
  Member length greater than 30 ft (9 m) .... ±12 in. (±300 mm)

k = Position of plates .... ±1 in. (±25 mm)

l = Position of bearing plates .... ±½ in. (±13 mm)

m = Tipping and flushness of plates .... ±¼ in. (±6 mm)

n = Tipping and flushness of bearing plates .... ±⅛ in. (±3 mm)

o = Position of sleeves .... ±1 in. (±25 mm) in both horizontal and vertical plane

p = Position of inserts for structural connections .... ±½ in. (±13 mm)

q = Position of handling devices
  Parallel to length .... ±12 in. (±300 mm)
  Transverse to length .... ±½ in. (±13 mm)

r = Position of stirrups
  r_1 longitudinal spacing .... ±2 in. (±50 mm)
  r_2 projection above surface of beam .... +¼ in., −½ in. (+ 6 mm, − 13 mm)

s = Local smoothness any surface .... ¼ in. in 10 ft (6mm in 3 m)
  Does not apply to top surface left rough to receive a topping or to visually concealed surfaces. (Refer to Fig. 2.5 for definition.)

2.16.4 I BEAMS* (SEE FIG. 2.9)

a = Length .... ±¼ in. per 25 ft, ±1 in. max (±6 mm per 10.5 m, ±25 mm)

b = Width (overall) .... +⅜ in., −¼ in. (+9.5 mm, −6 mm)

c = Depth (overall) .... +½ in., −¼ in. (+13 mm, −6 mm)

d = Depth (flanges) .... ±¼ in. (±6 mm)

e = Width (web) .... +⅜ in., −¼ in. (+9.5 mm, −6 mm)

f = Sweep (variation from straight line parallel to centerline of member) .... ⅛ in. per 10 ft (3 mm per 3 m)

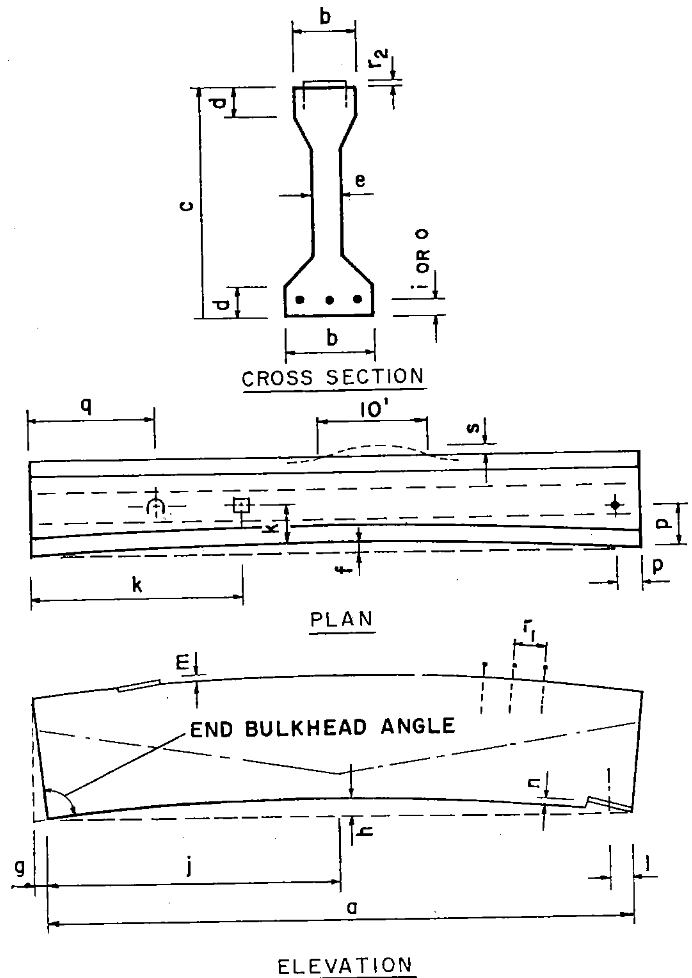

Fig. 2.9. I beams.

g = Variation from specified end squareness or skew .... ±$\frac{3}{16}$ in. per 12 in., ±1 in. max (±5 mm per 300 mm, ±25 mm max)

h = Camber variation† from

*Bridge authorities often specify a full set of tolerances for bridge girders. When these members are used as bridge girders, one must take care to check these tolerances against those specified by the bridge authority controlling the project.

†For members with a span-to-depth ratio approaching or exceeding 30, the stated camber tolerance may not apply. If the application requires control of camber to this tolerance in beams of such span-to-depth ratios, special premium production measures may be required. This requirement should be discussed in detail with the producer.

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design camber ....±⅛ in. per 10 ft (±3 mm per 3 m)
±½ in. (±13 mm) max up to 80 ft (24 m) length
±1 in. (±25 mm) max over 80 ft (24 m) length

i = Position of tendons
Individual .... ±¼ in. (±6 mm)
Bundled .... ±½ in. (±13 mm)

j = Position from design location of deflection points for deflected strands‡

k = Position of plates .... ±1 in. (±25 mm)

l = Position of bearing plates .... ±⅝ in. (±16 mm)

m = Tipping and flushness of plates .... ±¼ in. (±6 mm)

n = Tipping and flushness of bearing plates .... ±⅛ in. (±3 mm)

o = Position of post-tensioning duct .... ±¼ in. (±6 mm)

p = Position of inserts for structural connections .... ±½ in. (±13 mm)

q = Position of handling devices
Parallel to length .... ±6 in. (±150 mm)
Transverse to length .... ±1 in. (±25 mm)

r = Position of stirrups
r_1 longitudinal spacing .... ±2 in. (±50 mm)
r_2 projection above top .... ±¾ in. (±19 mm)

s = Local smoothness any surface .... ¼ in. in 10 ft (6 mm in 3 m)
Does not apply to top surface left rough to receive a topping or to visually concealed surfaces.
(Refer to Fig. 2.5 for definition.)

‡The economic location of strand deflection points depends in large measure on the individual bed characteristics. Use of a large location tolerance for this item is often possible with little design consequence. Location tolerances on the order of ±20 in. (±510 mm) will provide benefits of economy.

2.16.5 BOX BEAMS* (SEE FIG. 2.10)

a = Length .... ±¾ in. (±19 mm)

b = Width (overall) .... ±¼ in. (±6 mm)

c = Depth (overall) .... ±¼ in. (±6 mm)

d_t = Depth (top flange) .... ±½ in. (±13 mm)

d_b = Depth (bottom flange) .... +½ in., −⅛ in. (+13 mm, −3 mm)

e = Width (web) .... ±⅜ in. (±9.5 mm)

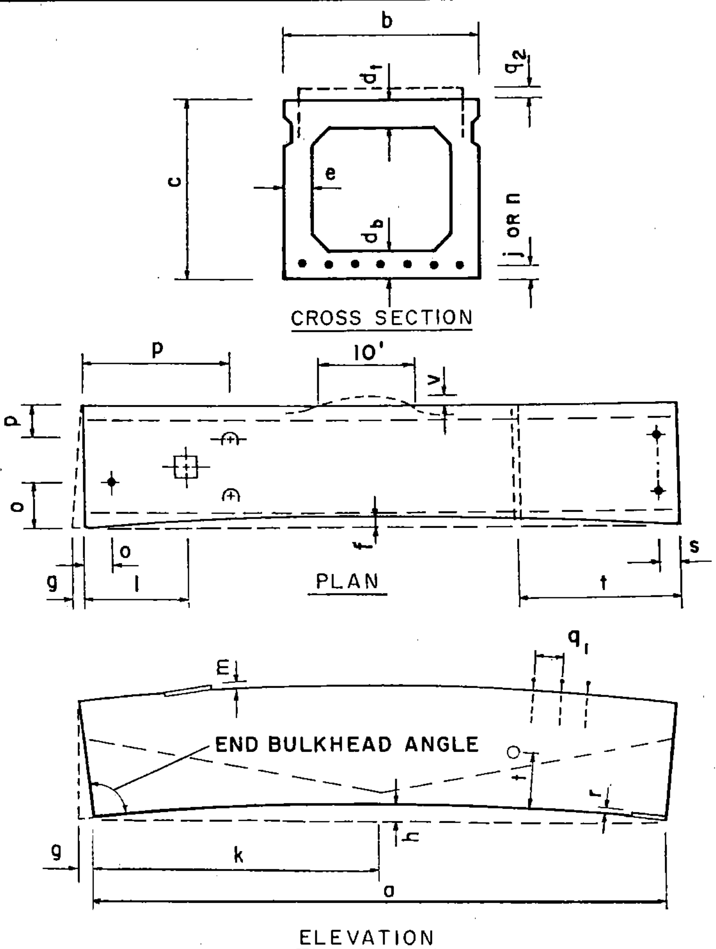

Fig. 2.10. Box beams.

f = Sweep (variation from straight line parallel to centerline of member)
Up to 40 ft (12 m) member length .... ±¼ in. (±6 mm)
40 to 60 ft (12 to 18 m) member length .... ±⅜ in. (±9.5 mm)
Greater than 60 ft (18 m) member length .... ±½ in. (±13 mm)

g = Variation from specified end squareness or skew
Horizontal .... ±⅛ in. per 12 in., ±½ in. max
(±3 mm per 300 mm, ±13 mm max)
Vertical .... ±½ in. (±13 mm)

*Bridge authorities often specify a full set of tolerances for bridge girders. When these members are used as bridge girders, one must take care to check these tolerances against those specified by the bridge authority controlling the project.

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h = Camber variation from design camber .... ±⅛ in. per 10 ft, ±½ in. max†
(±3 mm per 3 m, ±13 mm max)

i = Differential camber between adjacent members of the same design .... ¼ in. per 10 ft, ¾ in. max†
(6 mm per 3 m, 19 mm max)

j = Position of tendons
- Individual .... ±¼ in. (±6 mm)
- Bundled .... ±¼ in. (±6 mm)

k = Position from design location of deflection points for deflected strands‡

l = Position of plates .... ±1 in. (±25 mm)

m = Tipping and flushness of plates .... ±¼ in. (±6 mm)

n = Position of post-tensioning duct .... ±¼ in. (±6 mm)

o = Position of inserts for structural connections .... ±½ in. (±13 mm)

p = Position of handling devices
- Parallel to length .... ±6 in. (±150 mm)
- Transverse to length .... ±1 in. (±25 mm)

q = Position of stirrups
- q_1 longitudinal spacing .... ±1 in. (±25 mm)
- q_2 projection above top .... +¼ in., −¾ in. (+6 mm, −19 mm)

r = Tipping of beam seat bearing area .... ±⅛ in. (±3 mm)

s = Position of dowel tubes .... ±⅝ in. (±16 mm)

t = Position of tie rod tubes
- Parallel to length .... ±½ in. (±13 mm)
- Vertical .... ±⅜ in. (±9.5 mm)

u = Position of slab void
- End of void to center tie down ....±½ in. (±13 mm)
- Adjacent to end block ....±1 in. (±25 mm)

v = Local smoothness any surface .... ¼ in. in 10 ft (6 mm in 3 m)
Does not apply to top of beam surface left rough to receive a topping or to visually concealed surfaces.
(Refer to Fig. 2.5 for definition.)

†For members with a span-to-depth ratio approaching or exceeding 30, the stated camber tolerance may not apply. If the application requires control of camber to this tolerance in beams of such span-to-depth ratios, special premium production measures may be required. This requirement should be discussed in detail with the producer.

‡The economical location of strand deflection points depends in large measure on the individual bed characteristics. Use of a large location tolerance for this item is often possible with little design consequence. Location tolerances on the order of ±20 in. (±510 mm) will provide benefits of economy.

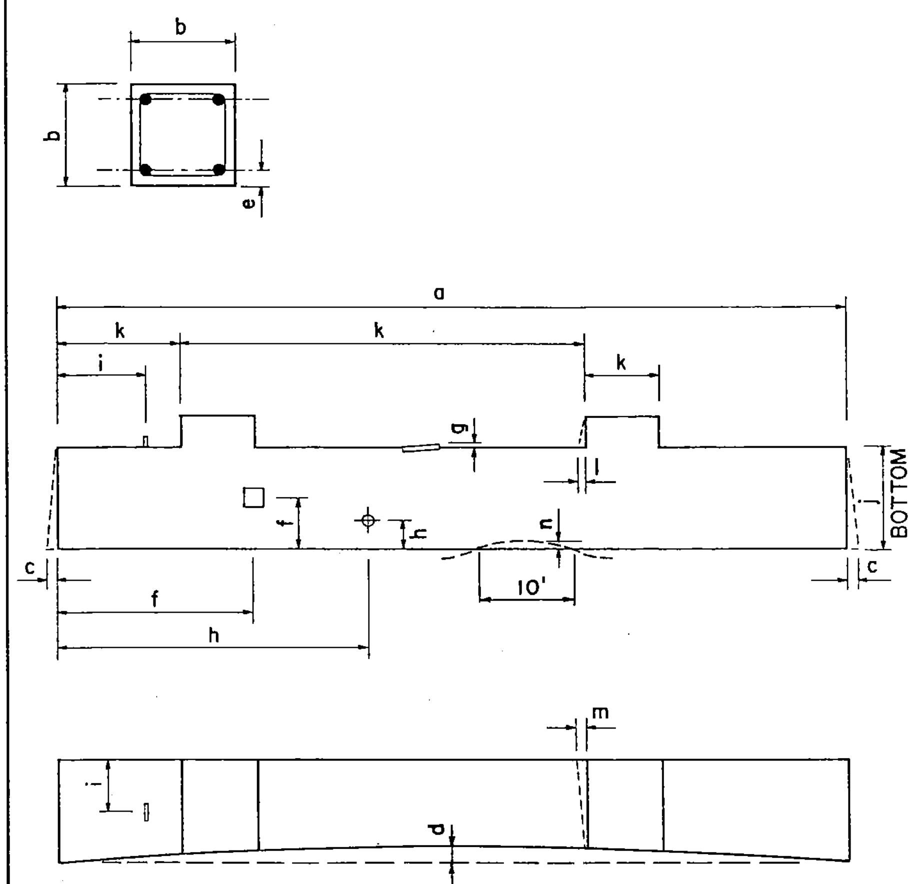

Fig. 2.11. Columns.

2.16.6 COLUMNS (SEE FIG. 2.11)

a = Length .... ±½ in. (±13 mm)

b = Cross section dimensions .... ±¼ in. (±6 mm)

c = Variation from specified end squareness or skew .... ±⅛ in. per 12 in., ±⅜ in. max
(±3 mm per 300 mm, ±9.5 mm max)

d = Sweep (variation from straight line parallel to centerline of member) .... ±⅛ in. per 10 ft, ±½ in. max
(±3 mm per 3 m, ±13 mm max)

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- e = Position of tendons .... ±¼ in. (±6 mm)
- f = Position of plates .... ±1 in. (±25 mm)
- g = Tipping and flushness of plates .... ±¼ in. (±6 mm)
- h = Position of inserts for structural connections .... ±½ in. (±13 mm)
- i = Position of handling devices
  - Parallel to length .... ±6 in. (±150 mm)
  - Transverse to length .... ±1 in. (±25 mm)
- j = Baseplate overall dimensions .... ±¼ in. (±6 mm)
- k = Haunch size and locations (not cumulative) .... ±¼ in. (±6 mm)
- l = Squareness of bearing .... ±⅛ in. (±3 mm)
- m = Squareness of bearing .... ±⅛ in. per 12 in., ±⅜ in. max (±3 mm per 300 mm, ±9.5 mm max)
- n = Local smoothness any surface .... ¼ in. in 10 ft (6 mm in 3 m)
  Does not apply to visually concealed surfaces.
  (Refer to Fig. 2.5 for definition.)

2.16.7 HOLLOW-CORE SLABS (SEE FIG. 2.12)

- a = Length .... ±½ in. (±13 mm)
- b = Width .... ±¼ in. (±6 mm)
- c = Depth .... ±¼ in. (±6 mm)
- d_t = Top flange thickness
  Top flange area defined by the actual measured values of average d_t x b shall not be less than 85 percent of the nominal area calculated by d_t nominal x b nominal.
- d_b = Bottom flange thickness
  Bottom flange area defined by the actual measured values of average d_b x b shall not be less than 85 percent of the nominal area calculated by d_b nominal x b nominal.
- e = Web thickness
  The total cumulative web thickness defined by the actual measured value Σe shall not be less than 85 percent of the nominal cumulative width calculated by Σe nominal.
- f = Blockout location .... ±2 in. (±50 mm)
- g = Flange angle .... ⅛ in. per 12 in., ½ in. max (3 mm per 300 mm, 13 mm max)
- h = Variation from specified end squareness or skew .... ±½ in. (13 mm)

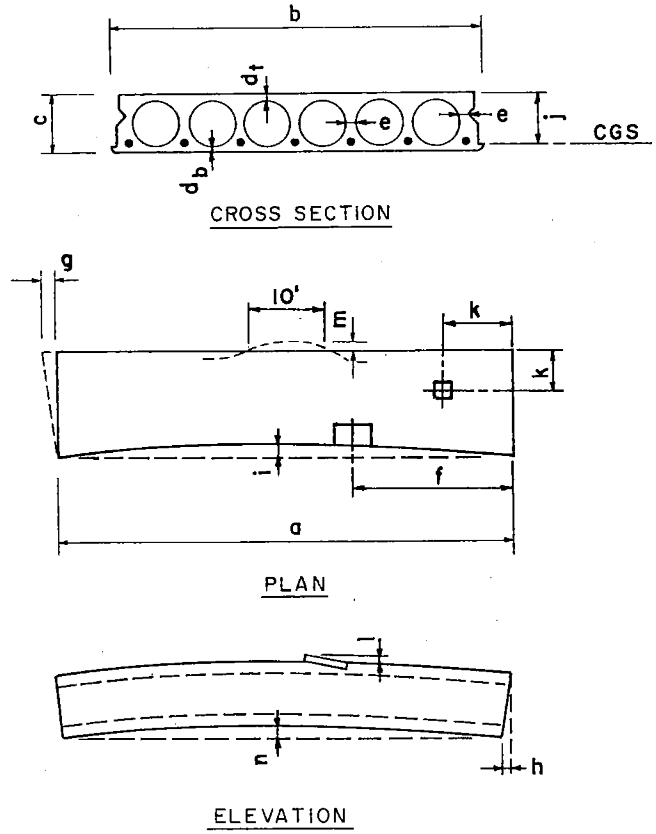

Fig. 2.12. Hollow-core slabs.

- i = Sweep (variation from straight line parallel to centerline of member) .... ±⅜ in. (±9.5 mm)
- j = Center of gravity (CG) of strand group
  The CG of the strand group relative to the top of the plank shall be within ±¼ in. (±6 mm) of the nominal strand group CG.
  The position of any individual strand shall be within ±½ in. (±13 mm) of nominal vertical position and ±¾ in. (±18 mm) of nominal horizontal position and shall have a minimum cover of ¾ in. (18 mm).
- k = Position of plates .... ±2 in. (±50 mm)
- l = Tipping and flushness of plates .... ±¼ in. (±6 mm)
- m = Local smoothness .... ¼ in. in 10 ft (6 mm in 3 m)
  Does not apply to top deck surface left

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rough to receive a topping or visually concealed surface.
(Refer to Fig. 2.5 for definition.)

Plank Weight

Excess concrete material in the plank internal features is within tolerance as long as the measured weight of the individual plank does not exceed 110 percent of the nominal published unit weight used in the load capacity calculations.

n = Camber applications requiring close control of differential camber between adjacent members of the same design should be discussed in detail with the producer to determine applicable tolerances.

2.16.8 RIBBED WALL PANELS (SEE FIG. 2.13)

a = Length .... ±½ in. (±13 mm)
b = Width .... ±¼ in. (±6 mm)
c = Depth .... ±¼ in. (±6 mm)
d = Stem width .... ±⅛ in. (±3 mm)
e = Flange thickness .... +¼ in., −⅛ in. (+6 mm, −3 mm)
f = Distance between stems .... ±⅛ in. (±3 mm)
g = Stem to edge of top flange .... ±⅛ in. (±3 mm)
h = Variation from specified flange squareness or skew .... ±⅛ in. per 12 in. width, ±¼ in. max (±3 mm per 300 mm width, ±6 mm max)
i = Variation from specified end squareness or skew .... ±⅛ in. per 12 in. (±3 mm per 300 mm)
j = Sweep (variation from straight line parallel to centerline of member)
  Up to 40 ft (12 m) member length .... ±¼ in. (±6 mm)
  40 ft (12 m) and greater member length .... ±⅜ in. (±9.5 mm)
k = Position of tendons .... ±¼ in. (±6 mm)
l = Position of blockouts .... ±1 in. (±25 mm)
m = Size of blockouts
  Finished opening .... ±½ in. (±13 mm)
  Rough opening .... ±1 in. (±25 mm)
n = Position of plates .... ±1 in. (±25 mm)
o = Tipping and flushness of plates .... ±¼ in. (±6 mm)
p = Position of inserts for structural connections .... ±½ in. (±13 mm)

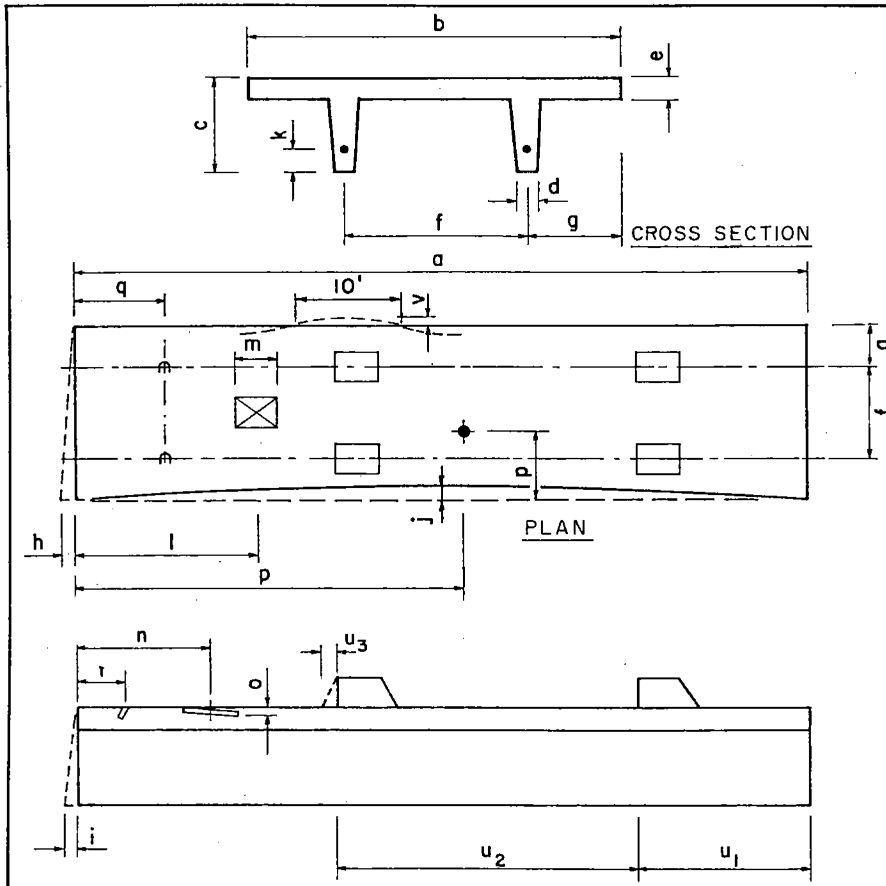

Fig. 2.13. Ribbed wall panels.

q = Position of handling devices
  Parallel to length .... ±6 in. (±150 mm)
  Transverse to length .... ±1 in. (±25 mm)
r = Bowing .... L/360 max*
s = Differential bowing between adjacent panels of the same design .... ½ in. (13 mm)*
t = Position of flashing reglets .... ±¼ in. (±6 mm)
u = Haunches (noncumulative)
u_1 = Bearing elevation from bottom of panel .... ±¼ in. (±6 mm)
u_2 = Relative position of bearing elevation in vertical plane .... ±⅛ in. (±3 mm)

*Refer to Chapter 2 for description of how bowing, warping, and smoothness variations are measured, and for dimensional limitations for which the tolerances apply.

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u_3 = Haunch bearing surface squareness perpendicular to applied major load .... ±⅛ in. per 18 in., ±¼ in. max (±3 mm per 460 mm, ±6 mm max)

v = Local smoothness any surface .... ¼ in. in 10 ft (6 mm in 3 m)* Does not apply to visually concealed surfaces. (Refer to Fig. 2.5 for definition.)

w = Warping .... ¹⁄₁₆ in. per ft (1.5 mm per 300 mm) of distance from nearest adjacent corner*

2.16.9 INSULATED WALL PANELS, SINGLE-STORY STRUCTURES (SEE FIG. 2.14)

a = Length .... ±½ in. (±13 mm)

b = Width .... ±¼ in. (±6 mm)

c = Depth (overall) .... ±¼ in. (±6 mm)

d = Wythe thickness .... ±⅜ in. (±9.5 mm)

e = Wythe squareness .... ±⅛ in. per 12 in. width, ±½ in. max (±3 mm per 300 mm width, ±13 mm max)

f = Variation from specified end squareness or skew .... ±⅛ in. per 12 in. (±3 mm per 300 mm)

g = Sweep (variation from straight line parallel to centerline of member) .... ±⅛ in. per 20 ft, ±⅜ in. max (±3 mm per 6 m, ±9.5 mm max)

h = Position of tendons .... ±¼ in. (±6 mm)

i = Position of blockouts .... ±1 in. (±25 mm)

j = Size of blockouts .... ±½ in. (±13 mm)

k = Position of plates .... ±1 in. (±25 mm)

l = Tipping and flushness of plates .... ±¼ in. (±6 mm)

m = Position of inserts for structural connections .... ±½ in. (±13 mm)

n = Bowing .... L/360 max*

o = Differential bowing between adjacent panels of the same design .... ½ in. (13 mm)*

*Refer to Chapter 2 for description of how bowing, warping, and smoothness variations are measured, and for dimensional limitations for which the tolerances apply.

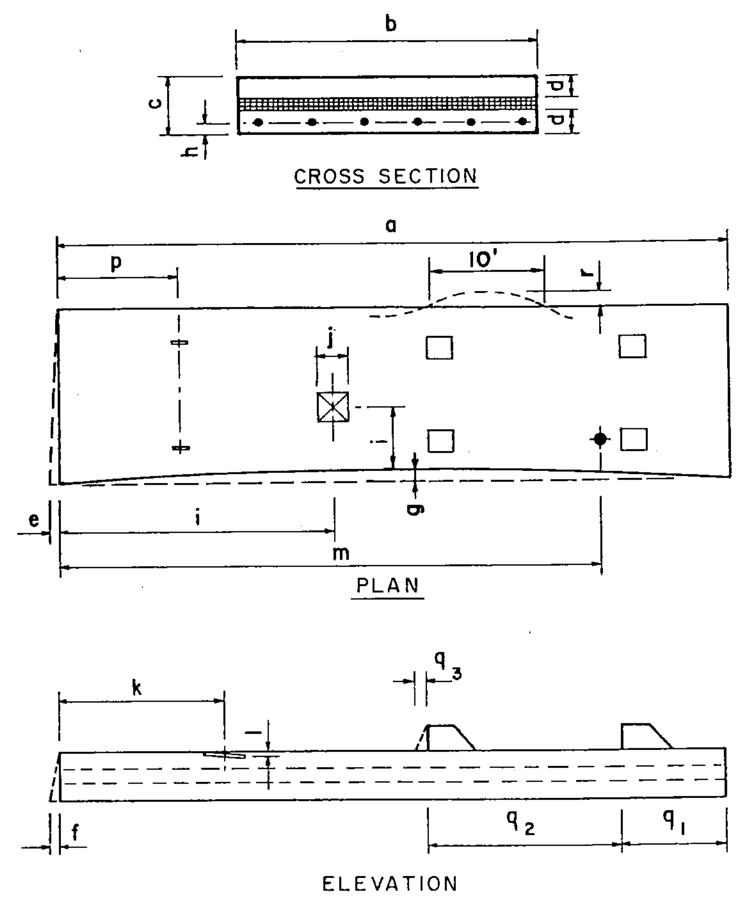

Fig. 2.14. Insulated wall panels, single-story structures.

p = Handling devices
Parallel to length .... ±6 in. (±150 mm)
Transverse to length .... ±1 in. (±25 mm)

q = Haunches (noncumulative)

q_1 = Bearing elevation from bottom of panel .... ±¼ in. (±6 mm)

q_2 = Relative position of bearing elevation in vertical plane .... ±⅛ in. (±3 mm)

q_3 = Haunch bearing surface squareness perpendicular to applied major load .... ±⅛ in. per 18 in., ±¼ in. max (±3 mm per 450 mm, ±6 mm max)

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r = Local smoothness any surface .... ¼ in. in 10 ft (6 mm in 3 m)*
Does not apply to visually concealed surfaces.
(Refer to Fig. 2.5 for definition.)

s = Warping .... 1/16 in. per ft (1.5 mm per 300 mm) of distance from nearest adjacent corner*

*Refer to Chapter 2 for description of how bowing, warping, and smoothness variations are measured, and for dimensional limitations for which the tolerances apply.

2.16.10 ARCHITECTURAL PANELS (SEE FIG 2.15)

a = Overall length and width (measured at neutral axis of ribbed members)
- 10 ft (3 m) or under .... ±⅛ in. (±3 mm)
- 10 to 20 ft (3 to 6 m) .... +⅛ in., −3/16 in. (+3 mm, −5 mm)
- 20 to 40 ft (6 to 9 m) .... ±¼ in. (±6 mm)
- Each additional 10 ft (3 m) .... ±1/16 in. (±1.5 mm) per 10 ft (3 m)

b = Total thickness or flange thickness .... −⅛ in., +¼ in. (−3 mm, +6 mm)

c = Rib thickness .... ±⅛ in. (±3 mm)

d = Rib to edge of flange .... ±⅛ in. (±3 mm)

e = Distance between ribs .... ±⅛ in. (±3 mm)

f = Angular variation of plane of side mold .... ±1/32 in. per 3 in. (±1 mm per 75 mm) of depth or ±1/16 in. (1.5 mm), whichever is greater

g = Variation from square or designated skew (difference in length of the two diagonal measurements) .... ±⅛ in. per 6 ft (±3 mm per 2 m) of diagonal or ±½ in. (±13 mm), whichever is greater*

h = Length and width of blockouts and openings within one unit .... ±¼ in. (±6 mm)

h_1 = Location and dimensions of blockouts hidden from view and used for HVAC and utility penetrations .... ±¾ in. (±19 mm)

h_2 = Some types of window and equipment frames require openings more accurately placed. When this is the case, the minimum practical tolerance should be defined with input from the producer.

i = Dimensions of haunches .... ±¼ in. (±6 mm)

*Applies both to panel and to major openings in the panel.

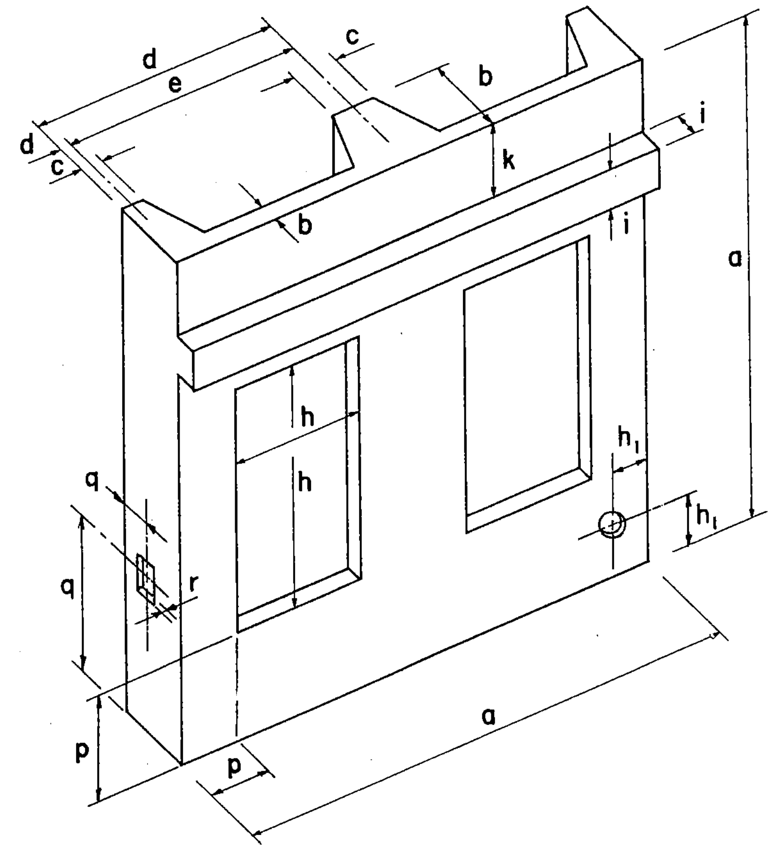

Fig. 2.15. Architectural panels.

j = Haunch bearing surface deviation from specified plane .... ±⅛ in. (±3 mm)

k = Difference in relative position of adjacent haunch bearing surfaces from specified relative position .... ±¼ in. (±6 mm)

l = Bowing .... L/360† max 1 in. (255 mm)

m = Differential bowing between adjacent panels of the same design .... ½ in. (13 mm)†

n = Local smoothness .... ¼ in. in 10 ft (6 mm in 3 m)†

†Refer to Chapter 2 for description of how bowing, warping, and smoothness variations are measured, and for dimensional limitations for which the tolerances apply.

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Does not apply to visually concealed surfaces.
(Refer to Fig. 2.5 for definition.)

o = Warping .... 1/16 in. per ft (1.5 mm per 300 mm) of distance from nearest adjacent corner†

p = Location of window opening within panel .... ±1/4 in. (±6 mm)

q = Position of plates .... ±1 in. (±25 mm)

r = Tipping and flushness of plates .... ±1/4 in. (±6 mm)

Position tolerances. For cast-in items measured from datum line location as shown on approved erection drawings:

Weld plates .... ±1 in. (±25 mm)

Inserts .... ±1/2 in. (±13 mm)

Handling devices .... ±3 in. (±75 mm)

Reinforcing steel and welded wire fabric ... ±1/4 in. (±6 mm) where position has structural implications or affects concrete cover, otherwise ±1/2 in. (±13 mm)

Tendons .... ±1/8 in. (±3 mm)

Flashing reglets .... ±1/4 in. (±6 mm)

Flashing reglets at edge of panel .... ±1/8 in. (±3 mm)

Reglets for glazing gaskets .... ±1/16 in. (±1.5 mm)

Groove width for glazing gaskets .... ±1/16 in. (±1.5 mm)

Electrical outlets, hose bibs, etc. .... ±1/2 in. (±13 mm)

Haunches .... ±1/4 in. (±6 mm)

†Refer to Chapter 2 for description of how bowing, warping, and smoothness variations are measured, and for dimensional limitations for which the tolerances apply.

2.16.11 PILING (HOLLOW AND SOLID) (SEE FIG. 2.16)

a = Length* .... ±1 in. (±25 mm)

b = Width or diameter .... ±3/8 in. (±9.5 mm)

c = Sweep (variation from straight line parallel to centerline of member) (considered to be a form tolerance) .... ±1/8 in. per 10 ft (±3 mm per 3 m)

d = Position of tendons .... ±1/4 in. (±6 mm)

*In most cases, controlling pile length to +6 in. (150 mm) −2 in. (50 mm) is functionally acceptable.

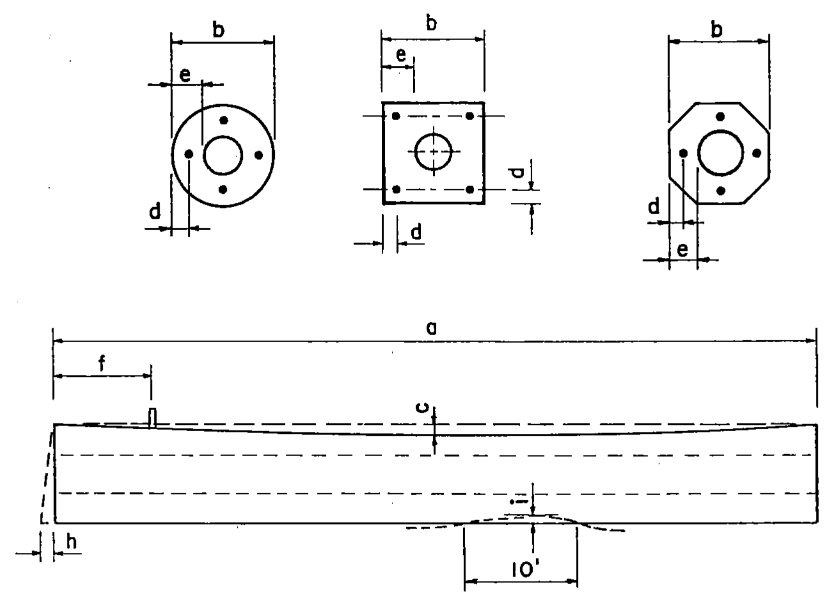

Fig. 2.16. Piling (hollow and solid).

e = Wall thickness .... −1/4 in., +1/2 in. (− 6 mm, + 13 mm)

f = Position of handling devices .... ±6 in. (±150 mm)

g = Position of steel driving tips .... ±1/2 in. (±13 m)

h = Variation from specified end squareness or skew .... ±1/4 in. per 12 in., ±1/2 in. max (±6 mm per 300 mm, ±13 mm max)

i = Local smoothness any surface .... 1/4 in. in 10 ft (6 mm in 3 m) (Refer to Fig. 2.5 for definition.)

j = Longitudinal spacing of spiral reinforcement .... ±3/4 in. (±19 mm)

2.16.12 TEE JOISTS OR KEYSTONE JOISTS (SEE FIG. 2.17)

a = Length* .... ±1 in. (±25 mm)

*For some types of combined precast and cast-in-place construction, it may be possible to achieve satisfactory and economical results with somewhat relaxed tolerances on these features. This situation should be discussed with the producer.

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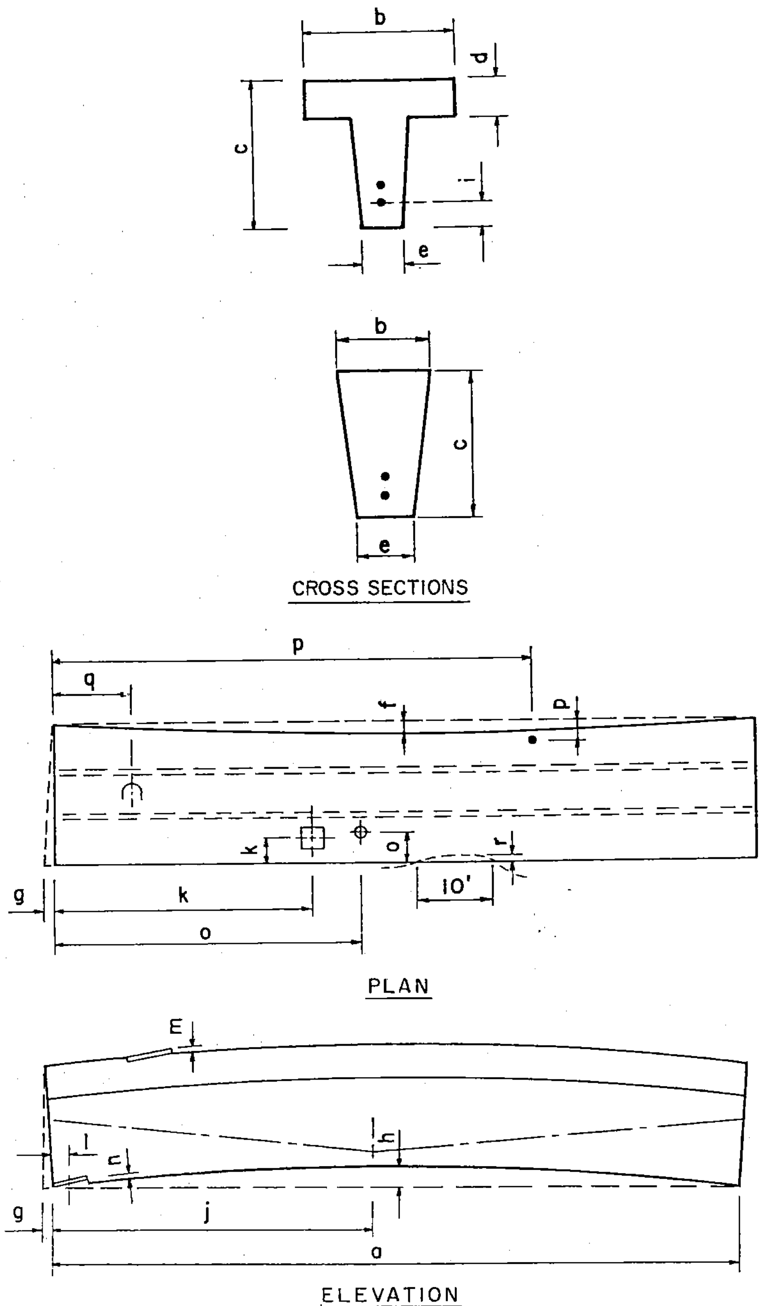

Fig. 2.17. Tee joists or keystone joists.

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b = Width .... ±¼ in. (±6 mm)

c = Depth .... ±¼ in. (±6 mm)

d = Flange thickness .... +¼ in., −⅛ in. (+6 mm, −3 mm)

e = Stem width .... ±⅛ in. (±3 mm)

f = Sweep (variation from straight line parallel to centerline of member)
  Up to 40 ft (12 m) member length .... ±⅜ in. (±9.5 mm)
  40 to 60 ft (12 to 18 m) member length .... ±⅝ in. (±16 mm)
  Greater than 60 ft (18 m) member length .... ±¾ in. (±19 mm)

g = Variation from specified end squareness or skew* .... ±¼ in. per 12 in., ±½ in. max (±6 mm per 300 mm, ±13 mm max)

h = Camber variation from design camber* .... ±¼ in. per 10 ft, ±¾ in. max† (±6 mm per 3 m, +19 mm max)

i = Position of tendons
  Individual .... ±¼ in. (±6 mm)
  Bundled .... ±½ in. (±13 mm)

j = Position of deflection points for deflected strands‡

k = Position of plates .... ±1 in. (±25 mm)

l = Position of bearing plates .... ±½ in. (±13 mm)

m = Tipping and flushness of plates .... ±¼ in. (±6 mm)

n = Tipping and flushness of bearing plates .... ±⅛ in. (±3 mm)

o = Position of sleeves cast in stems .... ±1 in. (±25 mm) in both horizontal and vertical plane

p = Position of inserts for structural connections .... ±½ in. (±13 mm)

q = Position of handling devices
  Parallel to length .... ±6 in. (±150 mm)
  Transverse to length .... ±1 in. (±25 mm)

r = Local smoothness any surface .... ¼ in. in 10 ft (6 mm in 3 m)
  Does not apply to top surface of joist left rough to receive a topping or to visually concealed surfaces. (Refer to Fig. 2.5 for definition.)

*For some types of combined precast and cast-in-place construction, it may be possible to achieve satisfactory and economical results with somewhat relaxed tolerances on these features. This situation should be discussed with the producer.

†For members with a span-to-depth ratio approaching or exceeding 30, the stated camber tolerance may not apply. If the application requires control of camber to this tolerance in beams of such span-to-depth ratios, special premium production measures may be required. This requirement should be discussed in detail with the producer.

‡The economical location of strand deflection points depends in large measure on the individual bed characteristics. Use of a large location tolerance for this item is often possible with little design consequence. Location tolerances on the order of ±20 in. (±510 mm) will provide benefits of economy.

2.16.13 STEP UNITS (SEE FIG. 2.18)

a = Length .... ±½ in. (±13 mm)
b = Width .... ±⅜ in. (±9.5 mm)
c = Thickness .... ±¼ in. (±6 mm)
d = Height .... ±½ in. (±13 mm)
e = Riser
    Height .... ±³⁄₁₆ in. (±4.5 mm)
    Maximum difference between two risers .... ±¼ in. (±6 mm)
f = Tread width .... ±¼ in. (±6 mm)
g = Position of miscellaneous inserts or plates .... ±1 in. (25 mm)
h = Variation from square or designated skew (difference in length of two diagonals) .... ±¼ in. per 6 ft (±½ in. max) (±6 mm per 1.8 m, ±13 mm max)
i = Position of inserts for structural connections .... ±⅜ in. (±9.5 mm)

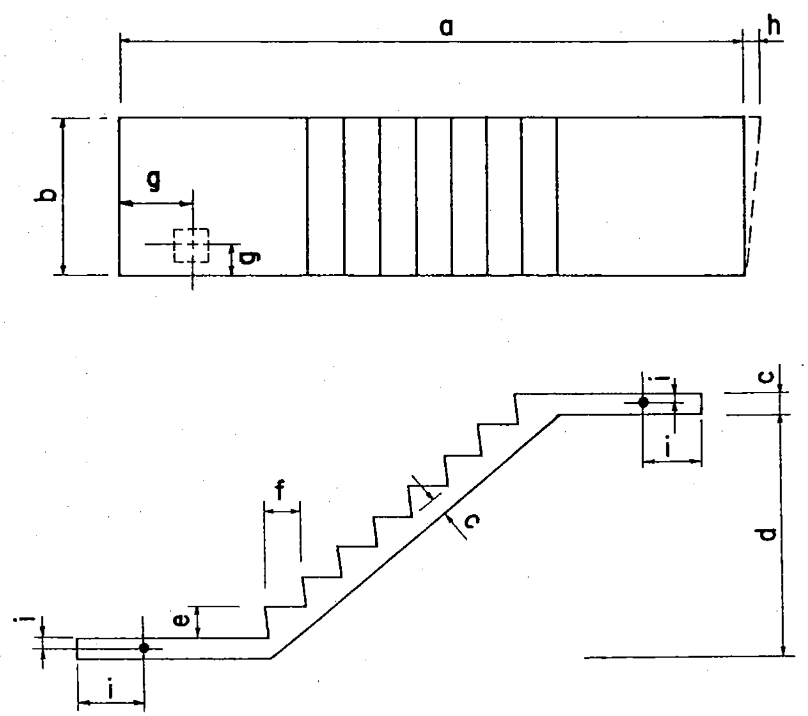

Fig. 2.18. Step units.

CHAPTER 3 — ERECTION TOLERANCES

This chapter discusses the tolerance principles and considerations related to the erection and acceptable matching of precast and prestressed concrete members when they are used for the entire structure or in combination with other structural systems.

To protect the project cost and schedule by minimizing erection problems, the dimensions and location of in-place structures should be checked prior to starting precast erection. After precast erection, and before other trades interface any material with the precast and prestressed concrete members, it should be verified that the precast elements are erected within recommended tolerances.

When establishing tolerances, the function of the precast concrete components should be considered. For example, members which are covered by finish materials may not need the close tolerances required for those that are exposed. Members used for an industrial building normally do not require tolerances as restrictive as those used for a visually sensitive commercial or residential application.

In general, the more restrictive the erection tolerances, the higher the cost of erection. For example, combining liberal product tolerances with restrictive erection tolerances may place an unrealistic burden on the erector. This can cancel any cost or time saving the designer expected to achieve by using larger product tolerances. It is therefore recommended that the designer review proposed tolerances with manufacturers and erectors prior to deciding on the final project tolerances.

3.1 Recommended Erection Tolerances

The recommended erection tolerance values are those to which the primary control surfaces of the member should be set. It is the positional dimensions of these primary control surfaces which are controlled during erection. The remaining positional dimensions of the features and secondary control surfaces of the member will be the result of the combination of the erection tolerances given here and the appropriate product tolerances given in Chapter 2.

The erection tolerances are given in three groups:

(1) Precast element to precast element
(2) Precast element to cast-in-place

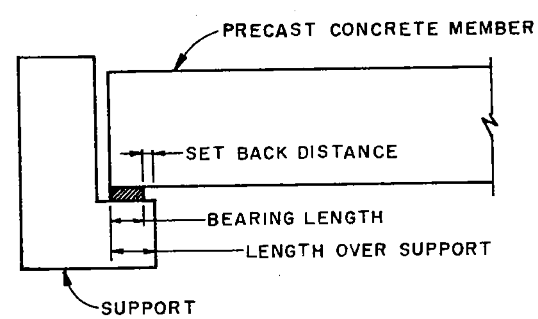

Fig. 3.1. Relationship between bearing length and length over the supports.

concrete or masonry
(3) Precast element to steel construction

Because erection is both equipment and site dependent, there may be good reason to considerably vary some of the recommended tolerances to account for unique project conditions. The effects of tolerances on the specific details in joints, at connections, and in other locations in the structure should be evaluated by the designer, since different details have varying amounts of sensitivity to tolerances.

Judgment should be used when applying these recommendations to unusual projects. To meet unusual tolerance requirements, the erection tolerances should be carefully reviewed by the designer and the involved contractors and adjusted, if necessary, to meet the project requirements.

Many of the erection tolerances address the tolerance on bearing length. The bearing and the length of the end of a member over the support are often not the same, as shown in Fig. 3.1. Bearing length is measured in the direction of the member span; bearing width is measured at 90 degrees to the direction of the member span. Acceptable bearing tolerance conditions which are required for safe erection should be shown on the erection drawings.

3.1.1 BEAMS AND SPANDRELS (SEE FIG. 3.2)

The following erection tolerances apply to beams and spandrels and particularly, precast element to precast element, precast element to cast-in-place concrete and masonry, and precast element to steel frame.

a = Plan location from building grid datum .... ±1 in (±25 mm)

a_1 = Plan from centerline of steel* .... ±1 in. (±25 mm)

b = Bearing elevation† from nominal elevation at support
- Maximum low .... ½ in. (13 mm)
- Maximum high .... ¼ in. (6 mm)

c = Maximum plumb variation over height of element
- Per 12 in. (300 mm) height .... ⅛ in. (3 mm)
- Maximum .... ½ in. (13 mm)

d = Maximum jog in alignment of matching edges
- Architectural exposed edges .... ¼ in. (6 mm)
- Visually noncritical edges .... ½ in. (13 mm)

e = Joint width
- Architectural exposed joints .... ±¼ in. (±6 mm)
- Hidden joints .... ±¾ in. (±19 mm)
- Exposed structural joint *not* visually critical ...... ±½ in. (±13 mm)

f = Bearing length‡ (span direction) .... ±¾ in. (±19 mm)

g = Bearing width‡ .... ±½ in. (±13 mm)

*For precast elements erected on a steel frame, this tolerance takes precedence over tolerance dimension "a."
†Or member top elevation where member is part of a frame without bearings.
‡This is a setting tolerance and should not be confused with structural performance requirements set by the architect/engineer.

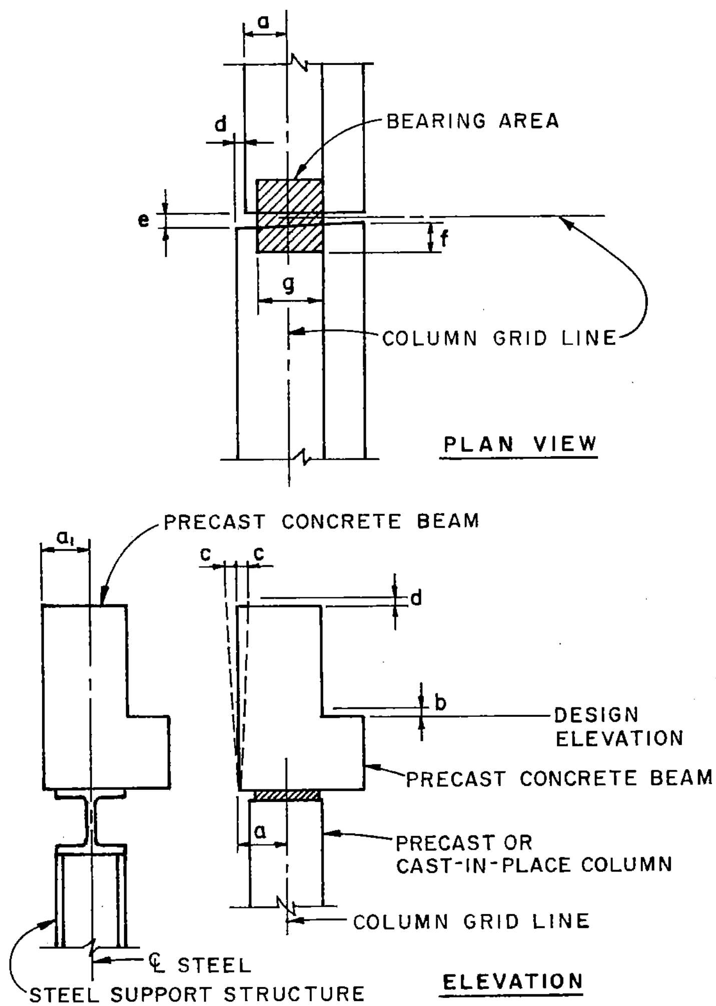

Fig. 3.2. Erection tolerances for beams and spandrels.

3.1.2 FLOOR AND ROOF MEMBERS (SEE FIG. 3.3)

The following erection tolerances apply to floor and roof members and particularly, precast element to precast element, precast element to cast-in-place concrete and masonry, and precast element to steel frame.

a = Plan location from building grid datum .... ±1 in. (±25 mm)

a_1 = Plan location from centerline of steel* .... ±1 in. (±25 mm)

*For precast elements erected on a steel frame, this tolerance takes precedence over tolerance on dimension "a."

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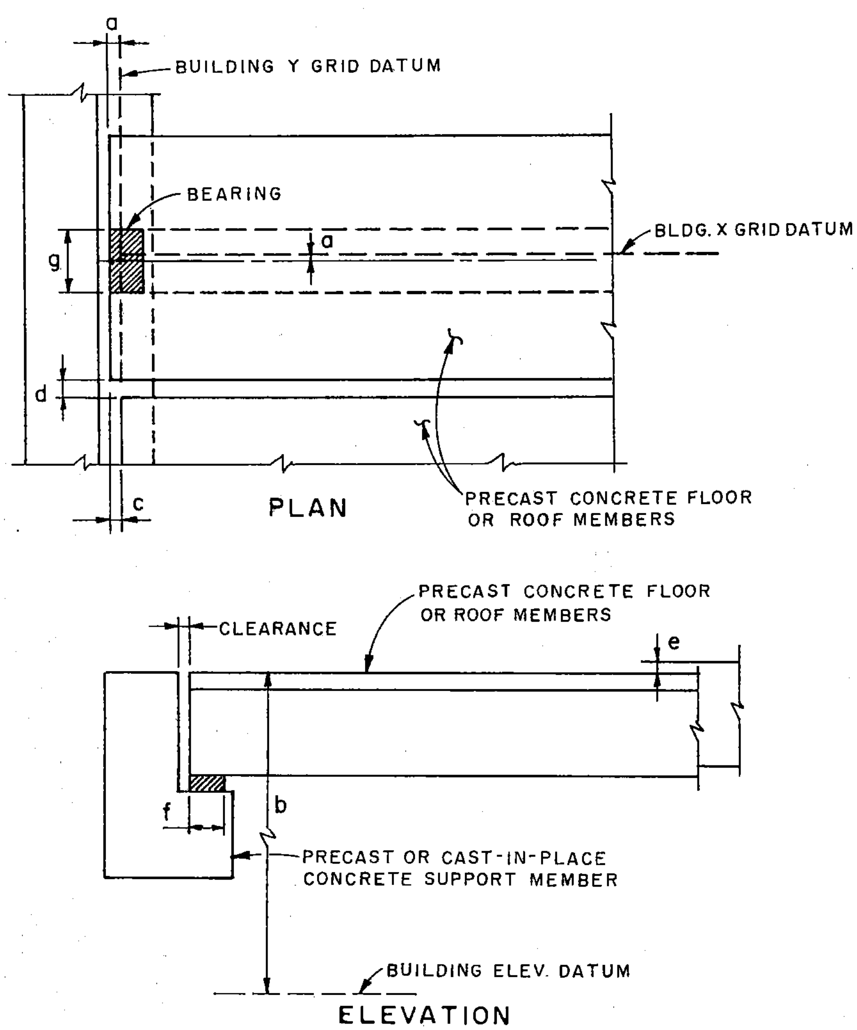

Fig. 3.3. Erection tolerances for floor and roof members.

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b = Top elevation from nominal top elevation at member ends
- Covered with topping .... ±¾ in. (±19 mm)
- Untopped floor .... ±¼ in. (±6 mm)
- Untopped roof .... ±¾ in. (±19 mm)

c = Maximum jog in alignment of matching edges (both topped and untopped construction) .... 1 in. (25 mm)

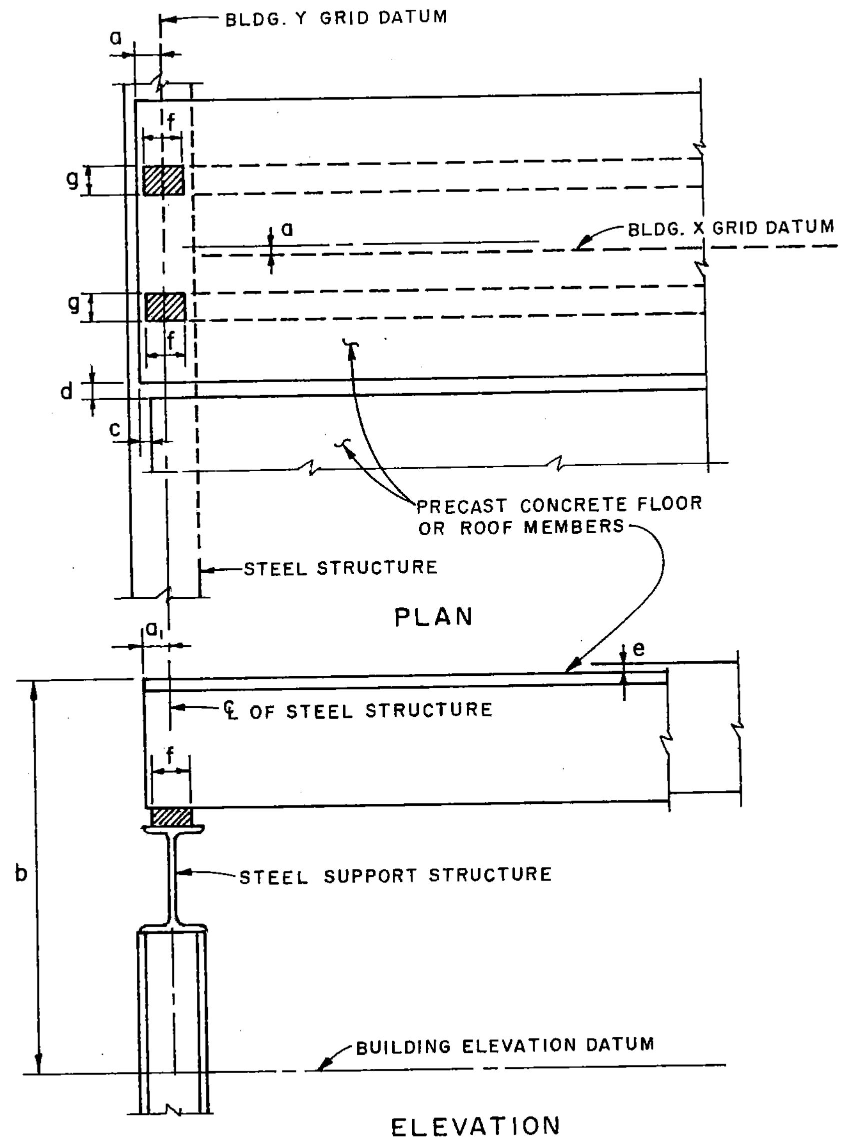

Fig. 3.3 (cont.). Erection tolerances for floor and roof members.

d = Joint width
- 0 to 40 ft (0 to 12 m) member length .... ±½ in. (±13 mm)
- 41 to 60 ft (12.5 to 18 m) member length .... ±¾ in. (±19 mm)
- 61 ft (18.5 m) plus .... ±1 in. (±25 mm)

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e = Differential top elevation as erected
  Covered with topping .... ¾ in. (19 mm)
  Untopped floor .... ¼ in. (6 mm)
  Untopped roof .... ¾ in. (19 mm)

f = Bearing length† (span direction) .... ±¾ in. (19 mm)

g = Bearing width† .... ±½ in. (±13 mm)

h = Differential bottom elevation of exposed hollow-core slabs‡ .... ¼ in. (6 mm)

†This is a setting tolerance and should not be confused with structural performance requirements set by the architect/engineer.

‡Untopped installations will require a larger tolerance.

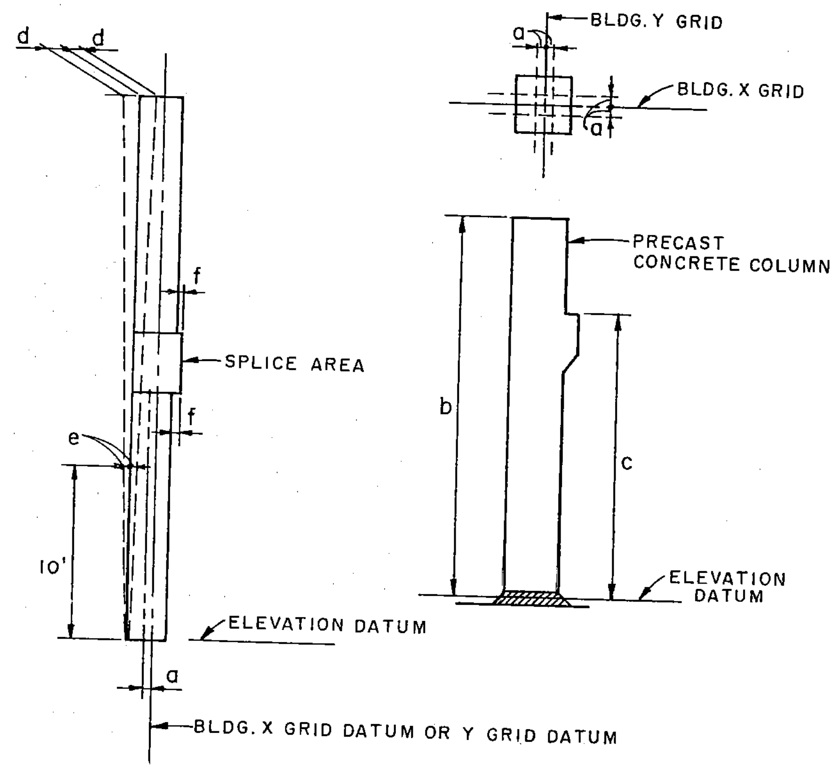

Fig. 3.4. Erection tolerances for columns.

3.1.3 COLUMNS (SEE FIG. 3.4)

The following erection tolerances apply to columns and particularly, precast element to precast element.

a = Plan location from building grid datum
  Structural applications .... ±½ in. (±13 mm)
  Architectural applications .... ±⅜ in. (±9.5 mm)

b = Top elevation from nominal top elevation
  Maximum low .... ½ in. (13 mm)
  Maximum high .... ¼ in. (6 mm)

c = Bearing haunch elevation from nominal elevation
  Maximum low .... ½ in. (13 mm)
  Maximum high .... ¼ in. (6 mm)

d = Maximum plumb variation height of element [element in structure of maximum height of 100 ft (30 m)] .... 1 in. (25 mm)

e = Plumb in any 10 ft (3 m) of element height .... ¼ in. (6 mm)

f = Maximum jog in alignment of matching edges
  Architectural exposed edges .... ¼ in. (6 mm)
  Visually noncritical edges .... ½ in. (13 mm)

3.1.4 STRUCTURAL WALL PANELS (SEE FIGS. 3.5 AND 3.6)

These erection tolerances apply to structural wall panels, particularly precast element to precast element, precast element to cast-in-place concrete and masonry, and precast element to steel frame.

a = Plan location from building grid datum* .... ±½ in. (±13 mm)

a_1 = Plan location from centerline of steel† .... ±½ in. (±13 mm)

b = Top elevation from nominal top elevation
  Exposed individual panel .... ±½ in. (±13 mm)
  Nonexposed individual panel .... ±¾ in. (±19 mm)
  Exposed relative to adjacent panel .... ½ in. (13 mm)
  Nonexposed relative to adjacent panel .... ¾ in. (19 mm)

c = Bearing elevation from nominal elevation
  Maximum low .... ½ in. (13 mm)
  Maximum high .... ¼ in. (6 mm)

d = Maximum plumb variation over height of structure or 100 ft (30 m) whichever is less* .... 1 in. (25 mm)

*For precast buildings in excess of 100 ft (30 m) tall, tolerances "a" and "d" can increase at the rate of ⅛ in. (3 mm) per story over 100 ft (30 m) to a maximum of 2 in. (50 mm).

†For precast elements erected on a steel frame, this tolerance takes precedence over tolerance on dimension "a."

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e = Plumb in any 10 ft (3 m) of element height .... ¼ in. (6 mm)

f = Maximum jog in alignment of matching edges .... ½ in. (13 mm)

g = Joint width (governs over joint taper) .... ±⅜ in. (±9.5 mm)

h = Joint taper over length of panel .... ½ in. (13 mm)

h_{10} = Joint taper over 10 ft (3 m) length ....⅜ in. (9.5 mm)

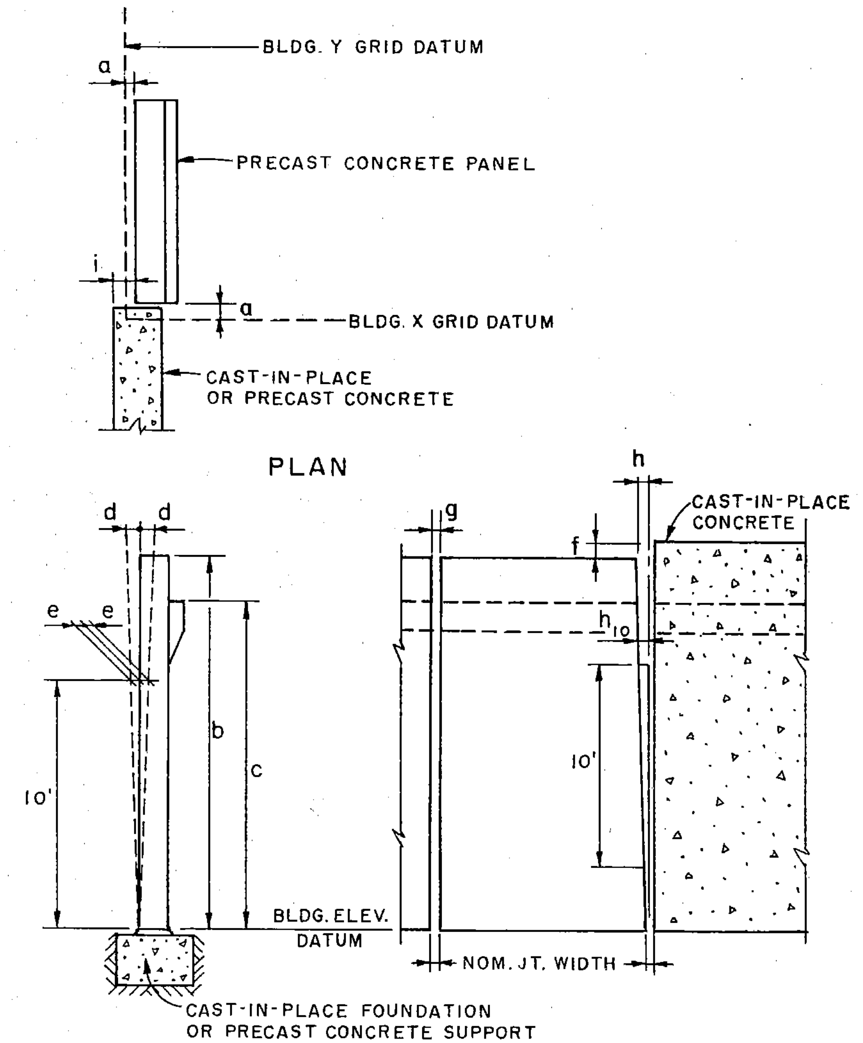

Fig. 3.5. Erection tolerances for structural wall panels.

i = Maximum jog in alignment of matching faces
Exposed .... ⅜ in. (9.5 mm)
Nonexposed .... ¾ in. (19 mm)

j = Differential bowing, as erected, between adjacent members of the same design‡ .... ½ in. (13 mm)

‡Refer to Chapter 2 (Warping, Bowing, and Local Smoothness of Panels) for description of bowing tolerance.

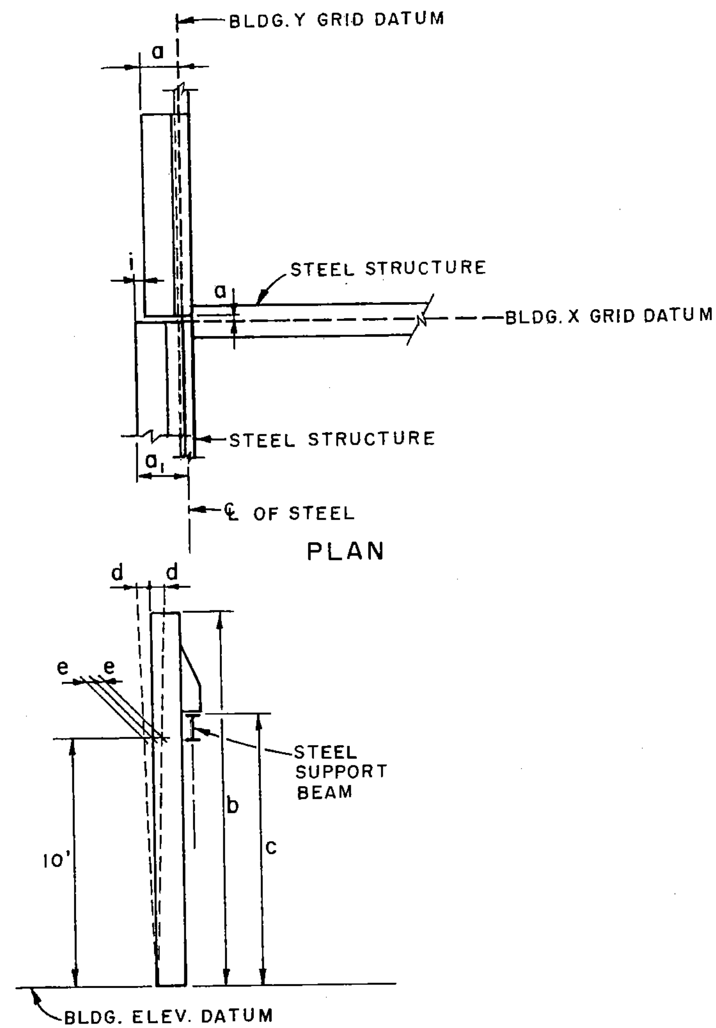

Fig. 3.6. Erection tolerances for wall panels.

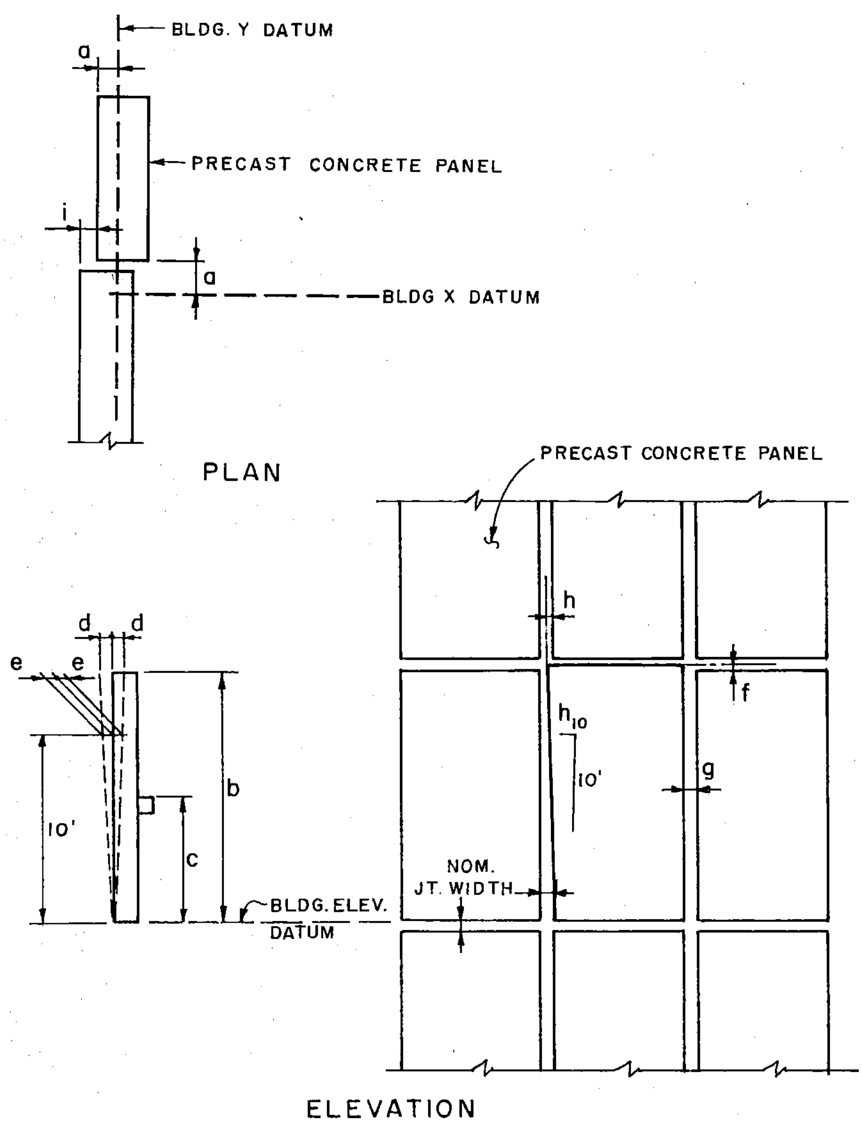

Fig. 3.6 (cont.). Erection tolerances for wall panels.

3.1.5 ARCHITECTURAL WALL PANELS (SEE FIG. 3.6)

These erection tolerances apply to architectural wall panels, particularly precast element to precast element, precast element to cast-in-place concrete and masonry, and precast element to steel.

- a = Plan location from building grid datum* .... ±½ in. (±13 mm)
- a_1 = Plan location from centerline of steel† .... ±½ in. (±13 mm)
- b = Top elevation from nominal top elevation
  - Exposed individual panel .... ±¼ in. (±6 mm)
  - Nonexposed individual panel .... ±½ in. (±13 mm)
  - Exposed relative to adjacent panel .... ¼ in. (6 mm)
  - Nonexposed relative to adjacent panel .... ½ in. (13 mm)
- c = Support elevation from nominal elevation
  - Maximum low .... ½ in. (13 mm)
  - Maximum high .... ¼ in. (6 mm)
- d = Maximum plumb variation over height of structure or 100 ft (30 m) whichever is less* ....1 in. (25 mm)
- e = Plumb in any 10 ft (3 m) of element height .... ¼ in. (6 mm)
- f = Maximum jog in alignment of matching edges .... ¼ in (6 mm)
- g = Joint width (governs over joint taper) .... ±¼ in. (±6 mm)
- h = Joint taper maximum .... ⅜ in. (9 mm)
- h_{10} = Joint taper over 10 ft (3 m) .... ¼ in. (6 mm)
- i = Maximum jog in alignment of matching faces .... ¼ in. (6 mm)
- j = Differential bowing or camber as erected between adjacent members of the same design‡ .... ¼ in. (6 mm)

*For precast buildings in excess of 100 ft (30 m) tall, tolerances "a" and "d" can increase at the rate of ⅛ in. (3 mm) per story over 100 ft (30 m) to a maximum of 2 in. (50 mm).

†For precast elements erected on a steel frame, this tolerance takes precedence over tolerance on dimension "a."

‡Refer to Chapter 2 (Warping, Bowing, and Local Smoothness of Panels) for description of bowing tolerance.

3.2 Mixed Building System

Mixed building systems subject erection tolerances to even more variables. A mixed system uses precast and prestressed concrete with other materials, usually cast-in-place concrete or steel.

Each industry has its own specified erection tolerances which apply when its products are used exclusively. Because the tolerances of different materials are not necessarily compatible, the compatibility of each industry's erection tolerances must be verified.

Clearance Example 3 shows one problem that can occur when erection tolerances are chosen for each system without considering the project as a whole.

3.3 Connections

The manner in which precast concrete members are connected together or to members fabricated of other materials must be considered when specifying erection tolerances. The designer must consider how the erector will physically construct the connections. Space must be provided so that adequate material and equipment can be placed in the connection and actually utilized in the intended manner under the most adverse combination of tolerances.

3.4 Clearances

Clearance is the space provided between adjacent members and is one of the most important factors to consider in erection. Clearance should provide a buffer area where erection and production tolerance variations can be absorbed.

Exposed joint clearance determination for architectural panels is an especially important consideration. In the architectural panel, the joint width must accommodate variations in the panel dimensions and the erection tolerances for the panel, and must still provide both a good visual line and sufficient width to allow for effective sealing. Generally, the larger the panel the wider the theoretical joint should be in order to accommodate realistic tolerances in straightness of panel edge, in edge taper, and in panel width.

Tolerances in overall building width and length are normally accommodated in panel joints, making the overall building size tolerance an important joint consideration. When all factors are combined and considered, for most situations the minimum theoretical architectural panel joint width should not be less than ¾ in. (19 mm).

The following items should be addressed when determining the appropriate clearance to provide in the design:

(1) Product Tolerance

The product tolerance of the element or system (if it is an interfacing situation) and the possible maximum and minimum variations in the size of the member must be considered when determining the clearance.

(2) Type of Member

The type of member is partially accounted for when the product tolerances are considered. Additionally, an exposed-to-view member requiring small erection tolerances requires more clearance for adjustments than does a nonexposed member with a more liberal erection tolerance. Similarly, a corner member should have a large enough clearance provided so it can be adjusted to line up with both of the adjacent panels.

(3) Size of the Member

Large members are more difficult to handle than smaller ones. Therefore, a large member being erected by a crane requires more clearance than the small member that can be hand erected or adjusted.

(4) Location of Member

The location of the member in the structure may also be critical. With multistory units for example, floor members are lowered from the top down and require a greater clearance than a roof member.

(5) Member Movement

Clearance design should consider movements caused by temperature expansion and contraction, creep, shrinkage, deflection, and rotation. The clearance between a vertical member and a horizontal member should allow for some movement in the horizontal member to prevent the vertical member from being pushed or pulled out of its original alignment. This is especially critical on exposed structures such as parking decks, where temperature ranges are large.

The effects of support member deflection on panel movement must be considered in determining the clearance between cladding panels that are supported by structural members.

(6) Function of Member

The function of a member also may affect clearance. For example, allowances must be provided for end rotation of heavily loaded beams; likewise, a minimum amount of joint width is needed to assure it can be reliably sealed when the member must provide protection against the elements.

(7) Erection Tolerance

The erection tolerances are important in clearance determination. If the clearance is too small erection may be slow and costly because of fit-up problems and the possibility of rework.

Of these factors, product tolerances and member movement are the most significant to consider. As shown in some of the examples that follow, it may not always be practical to account for all possible factors in the clearance provided.

3.5 Procedure for Determining Clearance

The following is a systematic approach for making a trial selection of a clearance value and then testing that selection to ensure that it will allow practical erection to occur.

Step 1 — Determine the maximum size of the members involved (basic or nominal dimension plus additive tolerances). This should include not only the precast and prestressed members, but also other materials.

Step 2 — Add to the maximum member size the minimum space required for member movement.

Step 3 — Check to see if this clearance allows the member to be erected within the erection and interfacing tolerances, such as plumbness, face alignment, etc. If the member interfaces with other structural systems, such as a steel frame or a cast-in-place concrete frame, check to see if the clearance provides for the erection and member tolerances of the interfacing system. Adjust the clearance as required to meet all of the needs.

Step 4 — Check to see if the member can physically be erected with the clearance determined above. Consider the size and location of members in the structure and how connections will be made. Adjust the clearance as required.

Step 5 — Review the clearance to see if increasing its dimensions will allow easier, more economical erection without adversely affecting aesthetics. Adjust the clearance as required.

Step 6 — Review structural considerations such as types of connections involved, sizes required, bearing area requirements, and other structural issues.

Step 7 — Check design to ensure adequacy in the event that minimum member size should occur. Adjust clearance as required for minimum bearing and other structural considerations.

Step 8 — Select the final clearance which will satisfy all of the conditions considered.

3.6 Clearance Examples

The following examples are given to show a thought process only. Judgment situations were created to emphasize that engineering judgment must be included as part of the clearance determination process. Therefore, the solutions shown are not the only correct ones for the situations described.

EXAMPLE 1

Given: A double-tee roof member 60 ft long (see Chapter 2, ±1 in. length tolerance) bearing on ribbed wall members 25 ft high, maximum plan variance ±½ in., variation from plumb ¼ in. per 10 ft, haunch depth 6 in. beyond face of panel, long-term roof movement −¼ in. Refer to Fig. 3.7. (Note: 1 ft = 0.305 m, 1 in. = 25.4 mm.)

Find: The minimum acceptable clearance.

Procedure:

Step 1 — Determine Maximum Member Sizes
(Refer to Product Tolerances)

Maximum tee length .... +1 in.
Wall thickness .... +¼ in.

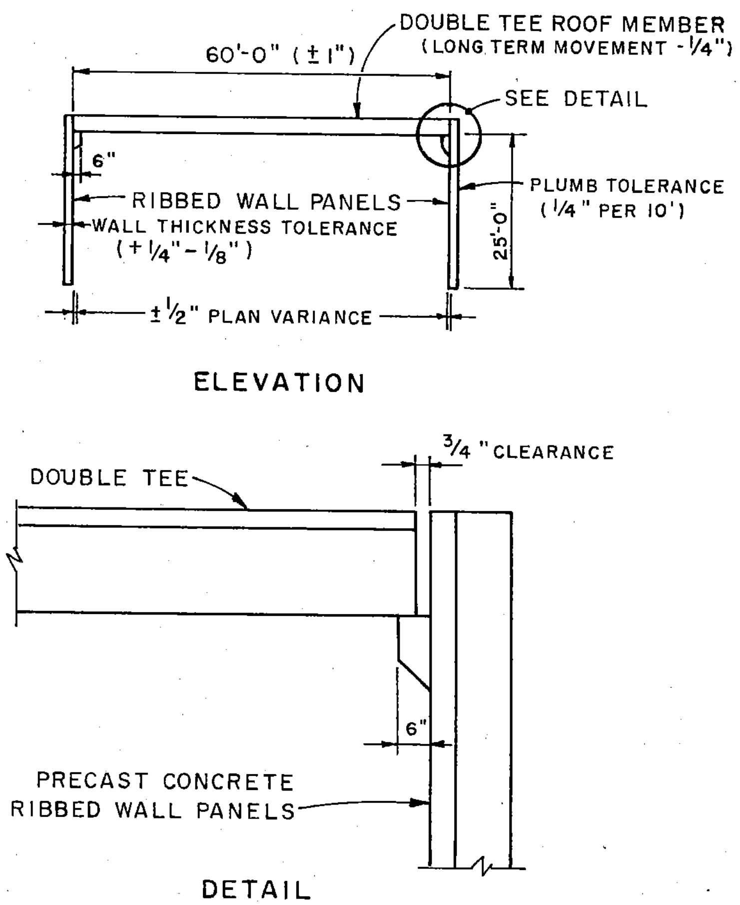

Fig. 3.7. Clearance Example 1.

Initial clearance
chosen .... ¾ in. per end

Step 2 — Member Movement

The long-term shrinkage and creep movement will increase the clearance so this movement can be neglected in the initial clearance determination, although it must be considered structurally.

Required clearance adjustment as
a result of member movement ... 0
Clearance chosen .... ¾ in.

Step 3 — Other Erection Tolerances

If the wall panel is set inward toward the building interior ½ in. and erected plumb, the clearance should be increased by ½ in. If the panel is erected out of plumb outward ½ in., no clearance adjustment is needed.

Clearance adjustment required
for erection tolerances 0
Clearance chosen .... ¾ in.

Step 4 — Erection Considerations

If all members are fabricated perfectly, then the joint clearance is ¾ in. at either end (1½ in. total). This is ample space for erection. If all members are at maximum size variance, maximum inward plan variance, and maximum inward variance from plumb, then the total clearance is zero. This situation is undesirable as it would likely require some rework during erection.

A judgment should be made as to the likelihood of maximum product tolerances all occurring in one location. If the likelihood is low, the ¾ in. clearance needs no adjustment but, if the likelihood is high, the engineer might increase the clearance to 1 in. In this instance, the likelihood has been judged low; therefore, no adjustment has been made.

Clearance chosen .... ¾ in.

Step 5 — Economy

In single-story construction, increasing the clearance beyond ¾ in. is not likely to speed up erection as long as product tolerances remain within allowables. No adjustment is required for economic considerations.

Step 6 — Review Structural Considerations

Allowing a setback from the edge of the corbel, assumed in this instance to have been set by the engineer at 1¼ in. plus the clearance, the bearing is 4 in. and there should be space to allow member movement. The engineer judges this to be acceptable from a structural and architectural viewpoint and no adjustment is required for structural considerations.

Step 7 — Check for Minimum Member Sizes
(Refer to Product Tolerances)

Tee length .. −1 in. (½ in. each end)
Wall thickness .... −⅛ in.
Bearing haunch .... No change
Clearance chosen .... ¾ in.

Minimum bearing, without setback 4⅝ in. is satisfactory in this instance.

Note: Wall plumbness would also be considered in an actual application.

Step 8 — Final Solution

Minimum clearance
used .... ¾ in. per end
(satisfies all conditions considered)

Note: For simplicity in this example, end rotation, flange skew, and global skew tolerances have not been considered. In an actual situation, these factors should also be taken into account.

EXAMPLE 2

Given: Bearing wall panel, 18 ft high erected on cast-in-place concrete footing. Minimum space for proper grouting in this case is judged to be ½ in. For simplicity, it is assumed the top of panel elevation to be set at exactly the basic elevation. Refer to Fig. 3.8. (Note: 1 ft = 0.305 m, 1 in. = 25.4 mm.)

Find: The minimum acceptable clearance.

Procedure:

Step 1 — Determine Maximum Member Sizes
(Refer to Product Tolerances)

Panel height .... +½ in.
Footing top elevation .... +½ in.
Initial clearance chosen .... 1 in.

Step 2 — Member Movement

Bottom of member will be fixed once member is grouted. No adjustment required for member movement.

Step 3 — Other Erection Tolerances

Not applicable because of the assumption that the top of the wall panel is being held at the nominal elevation.

Step 4 — Erection Considerations

Ease of erection is not influenced by the size of this clearance.

Step 5 — Economy

Varying the clearance above 1 in. will make the grouting operation more costly.

Step 6 — Structural Considerations

Clearance chosen .... 1 in.
Minimum grout bed .... ½ in.
Based on this, clearance
should be adjusted to ...... 1½ in.

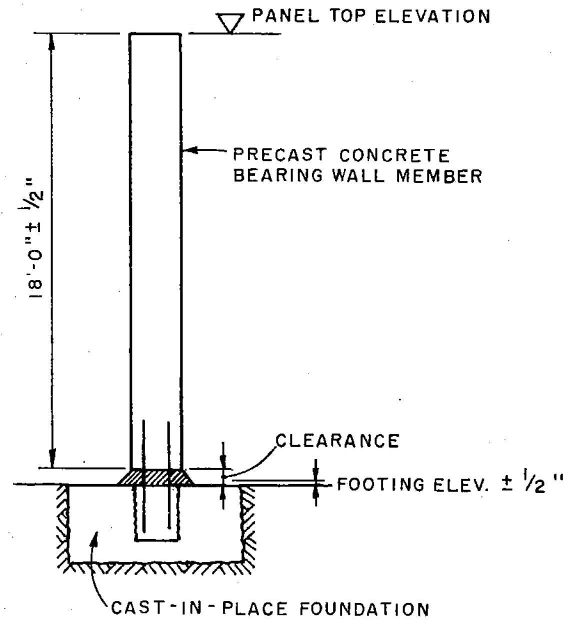

Fig. 3.8. Clearance Example 2.

Step 7 — Check Minimum Member Sizes
(Refer to Product Tolerances)

Previous clearance from Step 6 .... 1½ in.
Panel length .... ½ in. short
Footing top elevation ..... ½ in. low
Max clearance calculated 2½ in.

A judgment condition now exists. A 1½ in. standard grout bed with 2½ in. possibly is expensive. As a general rule, for normal contracting conditions, it is desirable to provide at least 1½ in. of clearance for a situation such as this. If special attention to detail is agreed upon with the contractors involved, one might reduce the clearance to 1 in.

Step 8 — Final Solution

Minimum clearance used ...... 1 in.
(satisfies all of the conditions considered)

Note: Alert contractor and erection crews to instances which may require isolated rework in order to provide minimum grout space.

EXAMPLE 3

Given: A 36-story steel frame structure, precast concrete cladding, steel tolerances per AISC, member movement negligible. In this example, precast tolerance for variation in plan is ±¼ in. Refer to Fig. 3.9. (Note: 1 ft = 0.305 m, 1 in. = 25.4 mm.)

Find: Whether or not the panels can be erected plumb and determine the minimum acceptable clearance at the 36th story.

Procedure:

Step 1 — Product Tolerances
(Refer to Product Tolerances)

Precast cladding thickness .... +¼ in., −⅛ in.

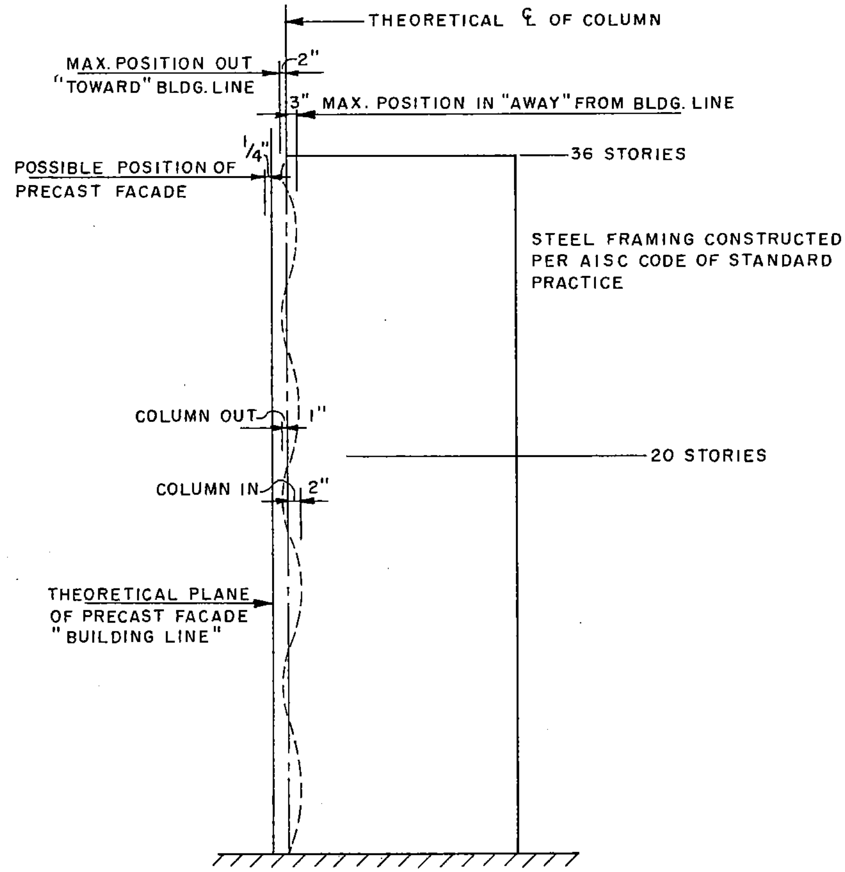

Fig. 3.9. Clearance Example 3.

Steel width .... +¼ in., −3/16 in.
Steel sweep (varies) .... ¼ in. assumption
Initial clearance chosen .... ¾ in.

Step 2 — Member Movement

For simplification, assume this can be neglected in this example.

Step 3 — Other Erection Tolerances

Steel variation in plan, maximum .... 2 in.
Initial clearance .... ¾ in.
Clearance chosen .... 2¾ in.

Step 4 — Erection Considerations

No adjustment required for erection considerations.

Step 5 — Economy

Clearance chosen .... 2¾ in.
Increasing clearance will not increase economy. No adjustment for economic considerations.

Step 6 — Structural Considerations

Clearance chosen .... 2¾ in.
Expensive connection but possible. No adjustment.

Step 7 — Check Minimum Member Sizes at 36th Story
(Refer to Product Tolerances)

Initial clearance	2¾ in.
Precast thickness	−⅛ in.
Steel width	−3/16 in.
Steel sweep	−¼ in.
Steel variation in plan minimum	−3 in.
Clearances calculated	6 5/16 in.

Step 8 — Final Solution

Minimum clearance used .... 2¾ in.

Note: When the minimum condition exists, the resulting clearance of over 6 in. produces an extremely expensive connection for the precast concrete. In addition, it produces a high torsional force that must be considered in the design of any steel horizontal supporting members. All together, this can substantially increase the cost of the steel perimeter members.

The 6 in. clearance is judged not practical, although the 2¾ in. minimal initial clearance is still needed. Therefore, the original erection tolerances need to be adjusted. Either the precast member should be allowed to follow the steel frame or the frame tolerances need to be made more stringent. The most economical and recommended solution likely will be for the precast member to follow the steel frame.

Another solution which has proven both practical and economical is to specify the more stringent AISC elevator column erection tolerances for steel columns in the building facade to receive the precast panels. This type of solution should be settled as part of the design and specification process, or at least prior to finalization of the fabrication erection process.

CHAPTER 4—INTERFACING TOLERANCES AND TYPICAL DETAILS

The purpose of this chapter is to help the designer deal with the problem of designing for interfacing tolerances. A comprehensive discussion of interfacing tolerances is presented and a number of typical details and examples are given as illustrations.

With interfacing tolerances, it is important to note that the tolerances may be very system dependent. For example, windows of one type may have a quite different interface tolerance requirement than windows of another type. If material or component substitutions are made after the initial design is complete, the interfacing design must be reviewed for the new system to assure compatibility of tolerances.

4.1 Interfacing Requirements

Following is a partial checklist to determine interfacing requirements.

(1) Structural Requirements

Does the interface perform a structural function in the structure (e.g., load transfer)? Does the behavior of the structure require that the interfacing system be isolated from primary or secondary structure loads? How are building motions, dimensional changes, and vibrations taken into account structurally, and how do they collectively affect interfacing tolerances?

(2) Volume Change

Does either the primary structure or the interfacing system undergo mutually incompatible volume changes?

(3) Weather/Corrosion

Is the interface exposed to weather? If so, what dimensional requirements result from the need to provide protection from moisture and the elements? How do the proposed precast concrete details enhance or detract from the ability of the structure to remain serviceable and durable over time?

(4) Waterproofing

What are the waterproofing requirements of the roofing details, exterior penetrations, and drainage schemes, and how do they apply to the interface between the precast concrete and the other materials?

(5) Drainage

Where are the areas to be drained and how does the drainage requirement affect the interface between precast concrete and other materials? What are the consequences of dimensional tolerance to the drainage system?

(6) Architectural Requirements

Which portions of the structure exterior and interior are exposed to view? What are the architectural treatments proposed for the various interfaces? How do these treatments relate to interfacing tolerance requirements?

(7) Dimensional Considerations

How closely can the dimensions of the other materials be controlled? What are the dimensional considerations relating to the proper function of the interfacing systems?

(8) Vibration Considerations

Does the mechanical subsystem have vibration considerations which must be accounted for in the interface between it and the precast concrete?

(9) Fire-Rating Requirements

Does the need for fire resistance of the system impose any requirements on the nature of the interface such as maximum allowable gaps?

(10) Acoustical Considerations

Does the acoustic environment place any special requirements on the interface between precast concrete and interfacing systems?

(11) Economics

Has the most economical interfacing design alternative been used? Has the cost trade-off between in-plant work and field work been considered? Does the chosen interface design alternative place any unusual demands on either the precast or the interfacing system?

(12) Manufacturing/Erection Considerations

Is the interfacing method chosen practical from a precast concrete manufacturing viewpoint? Is the time required to manufacture the interface consistent with factory productions? Can the interfacing parts of the structure be erected together safely and economically?

4.2 Interface Design Approach

The following design approach is one suggested method of organizing the task of systematically designing the interface between two tolerance systems.

Step 1—Define the interface between the two systems. Graphically define the interface to show its shape, location, and any split of contractual responsibility. Show the material furnished by the different contracting parties. For example, one might indicate the precast panel furnished by the precaster, the window furnished and installed by the general contractor, and the sealant between the window and the precast concrete furnished and installed by the general contractor.

Step 2—Define the functional requirements of each interfacing system. For example, the building drain line must have a flow line which allows adequate drainage. This will place functional limits on where the line must penetrate panels. Another example is that the tee beam has prestressing strands in the stems, making this a difficult location for drain line penetrations.

Step 3—Define the dimensional tolerances of each interfacing system. For example, determine from the manufacturer's specifications what the external tolerances on the door jamb are. Determine from the precast/prestressed concrete product tolerances what the tolerance on a large panel door opening will be. For the door installation, determine what the floor surface tolerance will be in the area of the door.

Step 4—Select the operational clearance required. For example, determine the magnitude of operational clearances which are needed to align the door to function properly. Then, make nominal dimensional choices which include an allowance for necessary clearances.

Step 5—Check compatibility of the interface tolerances. Starting with the least precise system, check the minimum and maximum tolerance conditions and compare against the minimum and maximum dimensions of the interfacing system. If interferences result, then alter the nominal dimension of the appropriate system. For example, it is usually more economical to make a larger window opening than to specify a prefabricated window system with either nonstandard sizes or tolerances.

Step 6—Establish assembly and installation procedures for the interfacing systems to assure compatibility. Show in the installation procedure the preferred adjustments to accommodate the tolerances of the interfacing systems. Specify such items as minimum allowable bearing areas, minimum and maximum joint gaps, and other features which will vary in dimension as a result of the interface tolerances. Consideration should be given here to a number of economic trade-off considerations such as in-plant work versus field work, and minor fit-up rework versus tighter tolerances.

Step 7—Check the final project specifications as they relate to interfacing. Be especially aware of possible subsystem substitutions made during the final bidding and procurement activities.

4.3 Characteristics of the Interface

The following list of questions will help to define the nature of the interface:

1. What specifically is to be interfaced?

2. How does the interface function?

3. Is there provision for adjustment upon installation?

4. How much adjustment can occur without rework?

5. What are the consequences of an interface tolerance mismatch?
- (a) Rework requirements (labor and material)
- (b) Rejection limits

6. What are the high material cost elements of the interface?

7. What are the high labor cost elements of the interface?

8. What are the normal tolerances associated with the system to be interfaced?

9. Are the system interface tolerances simple planar tolerances or are they more complex and three dimensional?

10. Do all of the different products of this type have the same interface tolerance requirements?

11. Do you as the designer of the precast system have control over all aspects of the interfaces involved? If not, what actions do you need to take to accommodate this fact?

Armed with the answers to these questions, one is prepared to undertake the detailed design of the interface.

Different types of systems may have substantially different interfacing requirements. For example, mechanical piping for on-site fabrication in primarily straight runs may have different interfacing tolerance requirements than do complex prefabricated mechanical piping systems which have bends occurring at penetration locations.

Listed below are some common characteristics and considerations which are typical of most systems:

(1) Windows and Doors
- (a) No load transfer through window element
- (b) Compatible with air and moisture sealant system
- (c) Open/close characteristics (swing or slide)
- (d) Compatibility with door locking mechanisms

(2) Mechanical Equipment
- (a) Duct clearances for complex prefabricated ductwork
- (b) Large-diameter prefabricated pipe clearance requirements
- (c) Deflection from forces associated with large-diameter piping and valves
- (d) Expansion/contraction allowances for hot/cold piping
- (e) Vibration isolation/transfer considerations
- (f) Acoustical shielding considerations
- (g) Hazardous gas/fluids containment requirements

(3) Electrical Equipment
- (a) Multiple mating conduit runs
- (b) Prefabricated cable trays
- (c) Embedded conduits and outlet boxes
- (d) Corrosion considerations related to DC power
- (e) Special insert placement requirements for electrical isolation
- (f) Location requirements for embedded grounding cables
- (g) Shielding clearance requirements for special "clean" electrical lines

(4) Elevators and Escalators
- (a) Elevator guide location requirements
- (b) Electrical conduit location requirements
- (c) Elevator door mechanism clearances
- (d) Special insert placement requirements

(5) Architectural Cladding
- (a) Joint tolerance for caulking system
- (b) Flashing and reglet fit-up (Lining up reglets from panel to panel is very difficult and often costly. Surface-mounted flashing should be considered.)
- (c) Expansion and contraction provisions for dissimilar materials
- (d) Effects of differential thermal gradients

(6) Structural Steel and Miscellaneous Steel
- (a) Details to prevent rust staining of concrete
- (b) Details to minimize potential for corrosion at field connections between steel and precast concrete
- (c) Coordination of structural steel expansion/contraction provisions with those of the precast system
- (d) Special provisions for weld plates or other attachment features for steel structures

(7) Masonry
- (a) Coordination of masonry expansion/contraction provisions with those of the precast system
- (b) Detailing to assure desired contact bearing between masonry and precast units
- (c) Detailing to assure desired transfer of load between masonry shear wall and precast frame

(8) Roofing
- (a) Roof camber, both upon erection and long term, as it relates to roof drain placement
- (b) Fit-up of prefabricated flashing
- (c) Dimensional effects of added material during reroofing
- (d) Coordination of structural control joint locations with roofing system expansion/contraction provisions
- (e) Location of embedded HVAC unit supports

(9) Waterproofing

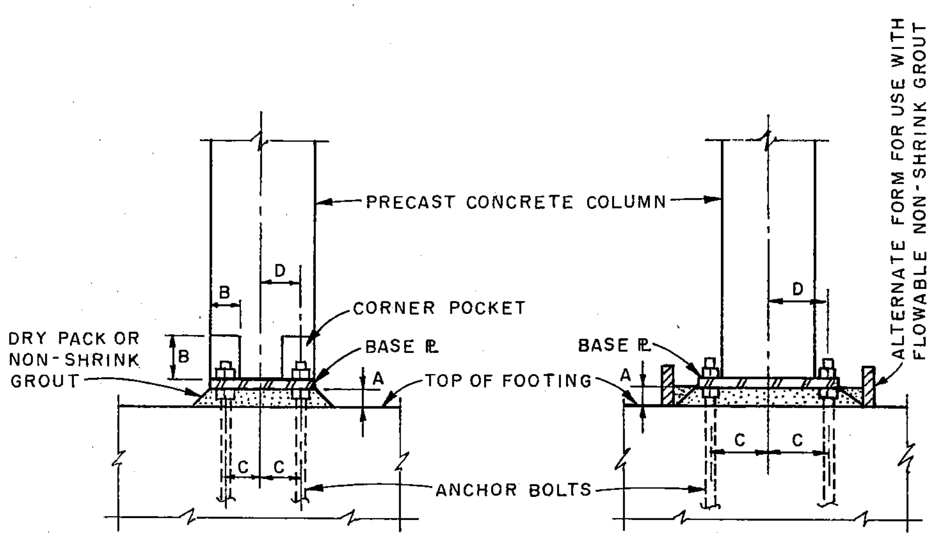

Fig. 4.1. Column to footing connection.

(a) Location and dimensions of flashing reglets
(b) Location and shape of window gasket grooves
(c) Coordination of waterproofing system requirements with structural system expansion provisions
(d) Special details around special penetrations

(10) Interior Finishes — Floors, Walls, and Ceilings

(a) Joints between plank members for direct carpet overlay
(b) Visual appearance of joints for exposed ceilings
(c) Fit-up details to assure good appearance of interior corners
(d) Appearance of cast-in-place to precast concrete interfaces

(11) Interior Walls and Partitions

(a) Clearance for prefabricated cabinetry
(b) Interfacing of mating embedded conduit runs

4.4 Typical Details

The following pages illustrate assemblies often used in precast concrete structures. In some instances, precast to precast details are shown; however, many of the details are also applicable to interfacing with other materials as well.

Detailing suggestions are given with each detail. This report is primarily concerned with tolerance related considerations; therefore, structural design and aesthetic aspects, while of great importance, are not generally emphasized.

Note that in all details showing weld plates, anchors are not shown to avoid confusion of lines; they, of course, also should be properly designed.

Detailing Suggestions for Column to Footing Connection (Fig. 4.1)

1. Provide clearance "A" to accommodate tolerances required of column length and footing elevation.
2. Provide pocket dimension "B" large enough for easy operation of manual or power wrench to tighten bolts.
3. Control clearance "C" by using templates to set footing anchor bolts.
4. Control dimension "D" by providing tighter tolerances on location and size of receiving holes in baseplate.
5. Do not use grout thickness less than 2 in. that is typically needed to accommodate bottom leveling nuts and for proper tamping of dry pack grout.

Detailing Suggestions for Column to Footing Connection (Fig. 4.2)

1. Provide clearance "A" large enough to accommodate tolerances required of column length and footing elevation.
2. Provide minimum 2 in. (50 mm) clearance (dimension "B") to accommodate proper flow of grout.
3. Control dimension "C" by using template for projection of column reinforcement.
4. Use oversize conduit at least two times the diameter of the reinforcing bar bundle to accommodate placement tolerances for conduit sleeve and column reinforcing bars.

Detailing Suggestions for Column to Footing Connection (Fig. 4.3)

1. Use matching template for column reinforcement and anchor bolt placement to control critical dimension "C".
2. Provide clearance "A" large enough to accommodate tolerances required of column length and footing elevation. Also, this clearance must be adequate for proper grouting of reinforcing bar sleeves.
3. Consult the manufacturer of reinforcing bar sleeves for proper dimensions "B" and "D" and for their tolerances.
4. Do not execute such assemblies without seeking recommendations from the manufacturer of reinforcing bar splice sleeve.

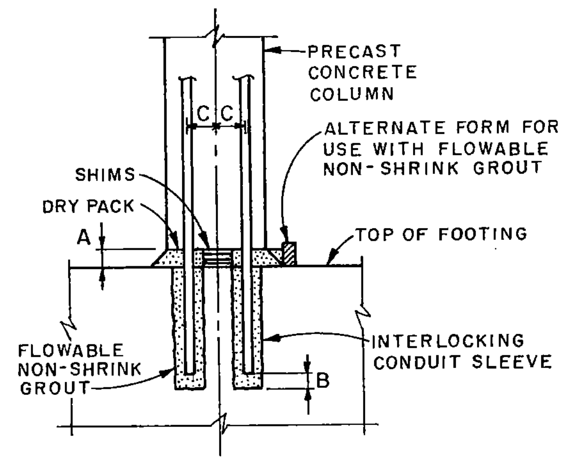

Fig. 4.2. Column to footing connection.

Detailing Suggestions for Wall Panel to Footing Connection (Fig. 4.4)

1. Provide clearance "A" to accommodate tolerances required of panel length and footing elevation.
2. Locate erection shims under centroid of the panel (dimension "B") for wall plumbness.
3. Provide adequate projection "C" to accommodate tolerances required for plate placement.
4. Use inside of wall as control surface when required for reference.
5. Do not use wall centroid as a control surface since tolerances of wall depth cause it to be a poor reference line.

Detailing Suggestions for Column to Column Connection (Fig. 4.5)

1. Provide adequate clearance "A" to accommodate the tolerances required for column length.
2. Provide pocket dimension "B" large enough for easy operation of manual or power wrench to tighten bolts.
3. Control dimension "C" by using templates for setting anchor bolts in the bottom column.
4. Do not use grout thickness less than 2 in. (50 mm) in order to accommodate leveling nuts.

Detailing Suggestions for Composite Beam to Support Double Tees (Fig. 4.6)

1. Consider the layout of the double tee stems while determining the spacings required for beam shear reinforcement. This is to assure that the shear reinforcement will not conflict with the stems of the tees.
2. Provide adequate cutback flange to assure that forming to the beam can be accomplished with minimal tolerance problems.
3. Do not provide top bars with too large a diameter as these bars often must thread through the clearance between the top of tee stems and the shear reinforcement.

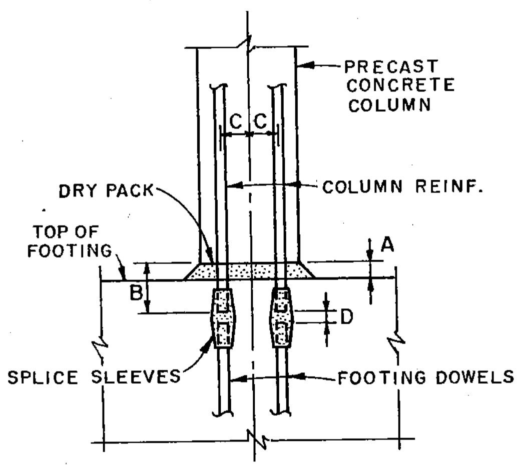

Fig. 4.3. Column to footing connection.

Detailing Suggestions for Beam to Column Connection (Fig. 4.7)

1. Use oversize conduits to accommodate tolerances required of insert placement. Refer to product tolerances for insert placement tolerances.
2. Provide clearance "A" to accommodate tolerances required of beam length and its end squareness.

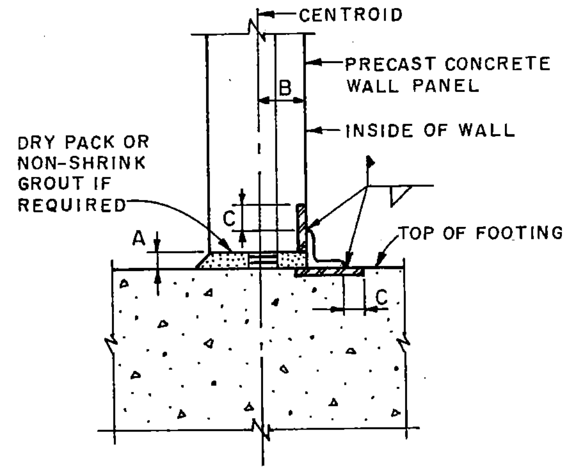

Fig. 4.4. Wall panel to footing connection.

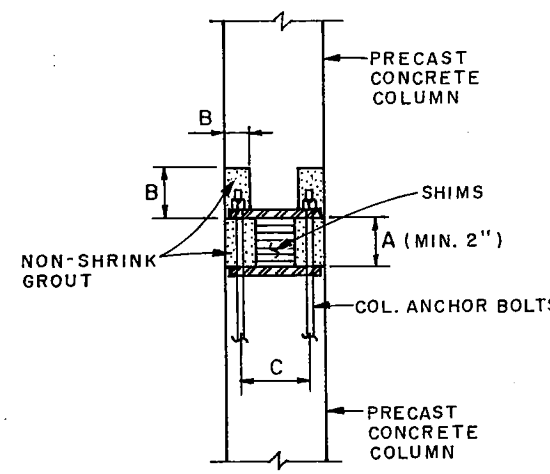

Fig. 4.5. Column to column connection.

3. Control dimension "B" by using a jig or template.
4. Prevent rotation of beam due to unbalanced loads.
5. Do not vary elevation for top of the column; instead, vary grout thickness at bottom to accommodate tolerances of column length.

Detailing Suggestions for Exterior Spandrel Beam to Double Tee Connection (Fig. 4.8)

1. Allow adequate offset "A" in order to accommodate tolerances required of the beam depth and the double tee bearing elevation. This is to ensure that the bottom of the tee will not be lower than the bottom of the beam; otherwise, the random bottom line of tees will form an unpleasant appearance as viewed from the exterior. A 1 in. (25 mm) offset usually provides a satisfactory condition.
2. This same type of offset "A" should be allowed for the surface of a suspended ceiling.
3. Provide adequate clearance "B" to accommodate tolerances.

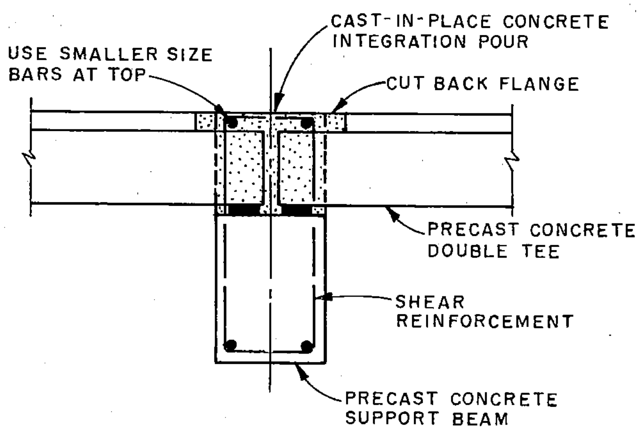

Fig. 4.6. Composite beam to support double tees.

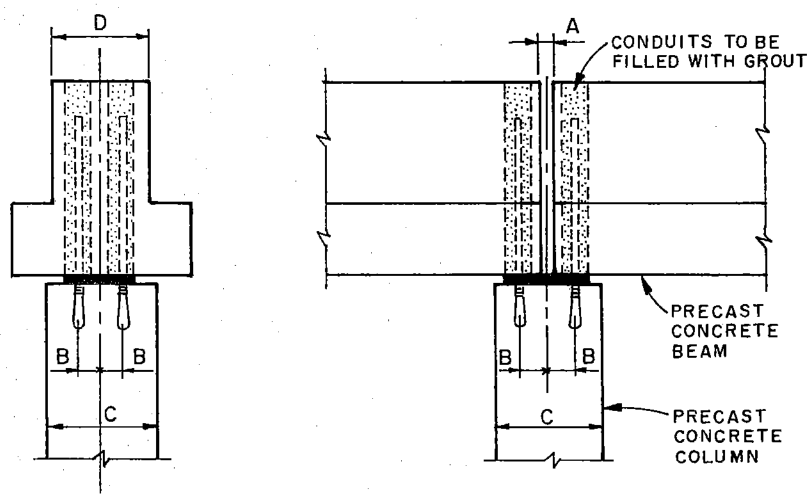

Fig. 4.7. Beam to column connection.

Detailing Suggestions for Load Bearing Wall Panel to Tee Connection (Fig. 4.9)

1. Maintain the top of haunch elevation. Tolerances for the wall length should be absorbed at the bottom connection by varying the grout thickness.
2. Provide adequate clearance "A" to accommodate tolerances required of the roof tee.
3. Provide adequate projection "B" to accommodate tolerances required of plate placement and for proper welding.

Detailing Suggestions for Wall Panel to Tee Connection (Fig. 4.10)

1. Provide adequate insert clearance "A" to accommodate anticipated camber growth of tee, including the camber tolerances.
2. Provide adequate clearance "B" to accommodate anticipated deflection of tee.
3. Use flashing to accommodate differential elevations at top of panels.
4. Take measures to assure that fingertight bolt will not work loose during the life of the structure.
5. Provide adequate clearance gap "C" to accommodate tolerances.
6. Do not tighten bolts more than just "finger tightening," per the AISC Code, since tightened bolts will prevent double tee movement in the vertical direction.

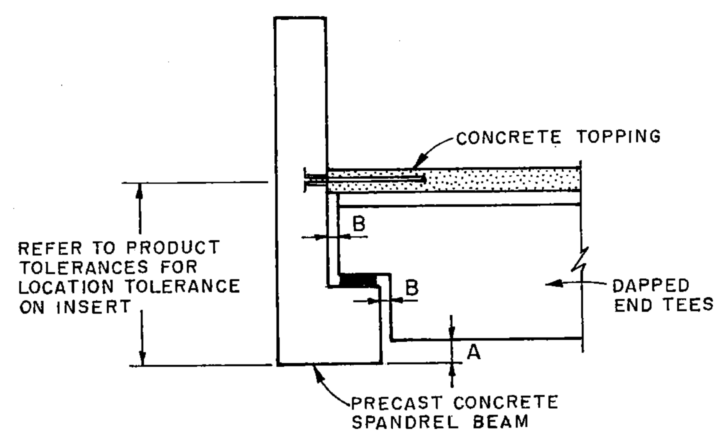

Fig. 4.8. Exterior spandrel beam to double tee connection.

Detailing Suggestions for Fascia Beam to Tee Connection (Fig. 4.11)

1. Provide adequate topping thickness "A" to accommodate the tolerances required for insert placement and add additional concrete at ends.
2. Consider the camber of the tee when deciding the location of the insert. Consideration should be given to using a slotted connection here.
3. Provide adequate clearance gap "C" to accommodate tolerances.
4. Do not have bottom of the beam and bottom of the tee at the same elevation. Allow adequate clearance "B" to accommodate tolerances required of beam height and double tee depth.

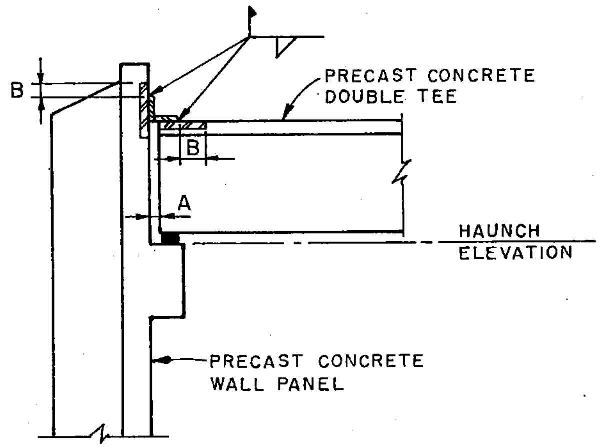

Fig. 4.9. Load bearing wall panel to tee connection.

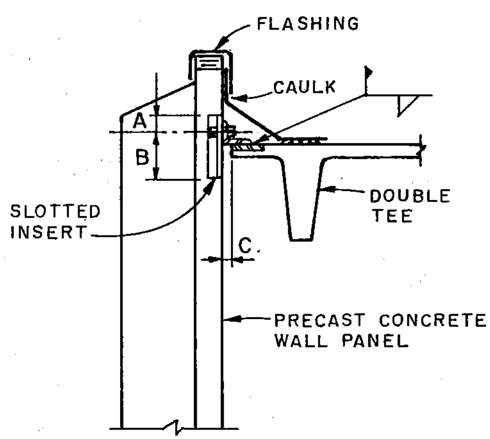

Fig. 4.10. Wall panel to tee connection.

Detailing Suggestions for Scupper Blockout in Wall Panels (Fig. 4.12)

1. Provide clearance "A" to accommodate tolerances required of tee depth and roof insulation placement.
2. Provide blockout dimension "B" of 6 in. (150 mm) minimum in order to minimize the plugging of these openings.
3. Place scupper away from panel joints to keep water from running into these joints.
4. Top of collection box should be at the same elevation or slightly lower than the bottom of the scupper to assure that tolerances allow positive drainage of the roof under all conditions.
5. Provide adequate clearance gap "C" to accommodate tolerances.

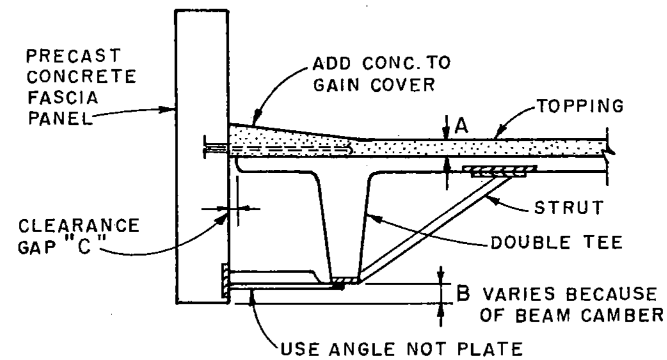

Fig. 4.11. Fascia beam to tee connection.

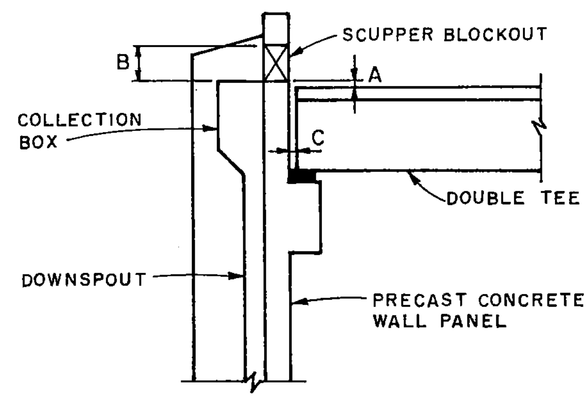

Fig. 4.12. Scupper blockout in wall panels.

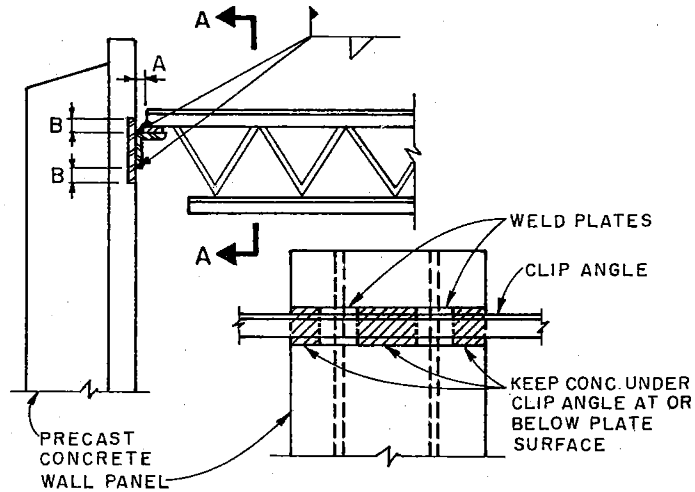

Fig. 4.13. Wall panel to bar joist connection.

Detailing Suggestions for Wall Panel to Bar Joist Connection (Fig 4.13)

1. Provide adequate plate projection "B" to accommodate tolerances required of plate location and for proper welding.
2. Provide adequate clearance "A" to accommodate tolerances required of bar joist span and panel vertical alignment.
3. Do not allow adjacent concrete to build up to a level higher than the weld plate: it may create tolerance problems for welding of the continuous angle.

Detailing Suggestions for Masonry Wall to Tee Connection (Fig. 4.14)

1. Provide adequate clearance "A" to accommodate tolerances required of vertical masonry alignment and tee lengths. Maximum dimension for "A" should be based on ease of forming and fire resistance considerations.
2. Set tolerances in a way that assures precast tee has some clearance "B" from the face of the masonry wall.
3. Locate bearing plate to assure beam does not contact masonry shell at "C" when tolerances are considered.

Detailing Suggestions for Typical Hollow-Core Joint (Fig. 4.15)

1. Level the hollow-core slabs at the joint location to minimize the camber differential for direct acoustical treatment on the ceiling.
2. Use heavy padding if the top of the hollow-core slabs receives direct carpet application.
3. Do not release the leveling devices at the joints until the grout is set.

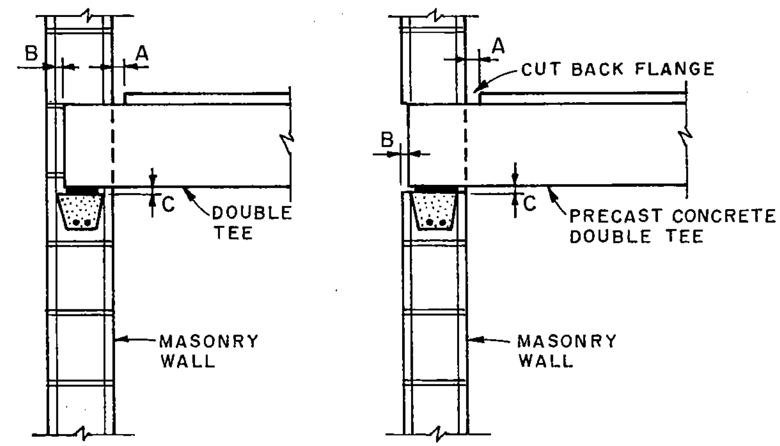

Fig. 4.14. Masonry wall to tee connection.

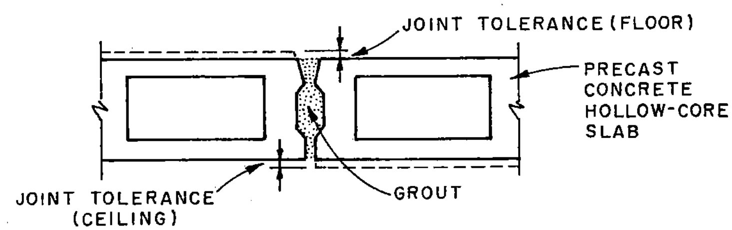

Fig. 4.15. Typical hollow-core joint.

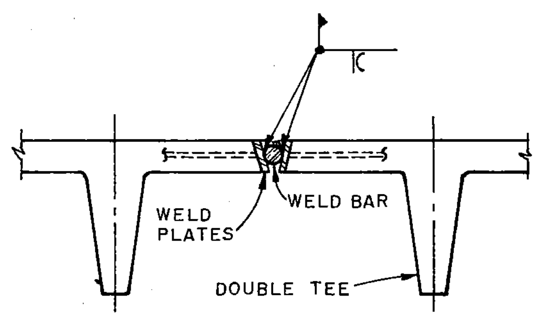

Fig. 4.16. Flange to flange connection.

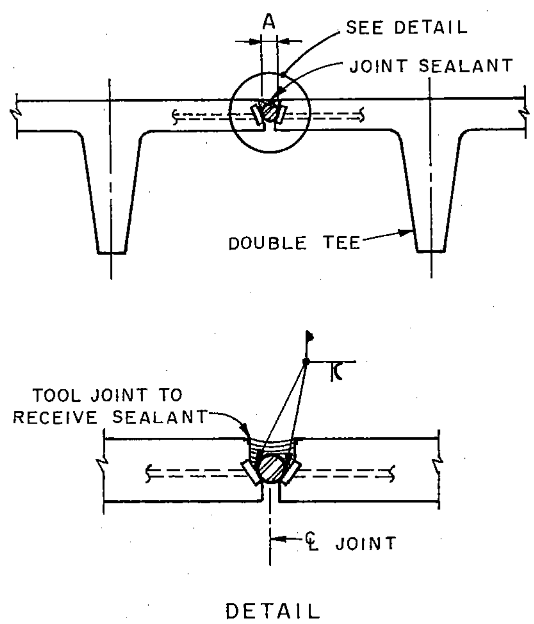

Fig. 4.17. Typical untopped double tee joints for parking structure.

Detailing Suggestions for Flange to Flange Connection (Fig. 4.16)

1. Provide weld plates at an angle to accommodate tolerances required for their placement and to maintain a proper fillet weld.
2. Provide weld bar size that is larger than the nominal double tee joint to assure its firm support under expected tolerance variations.

Detailing Suggestions for Typical Untopped Double Tee Joints for Parking Structure (Fig. 4.17)

1. Tool the edges of tees to prevent any damage to concrete and/or to sealant due to camber differential in tees.
2. Specify joint tolerances for the untopped tees which are compatible with the sealant system specified. These tolerances are normally more stringent than standard tee joint tolerances.
3. Provide adequate clearance gap "A" to accommodate tolerances.
4. Do not weld the tees until the bearing elevation is adjusted in such a manner that minimum camber differential is achieved.

Detailing Suggestions for Conduits in Double Tees (Fig. 4.18)

1. Provide conduits above the level of top strand or establish special tolerances for conduit placement due to draped strands.
2. Consider the tolerances required of shear reinforcement when locating conduits.

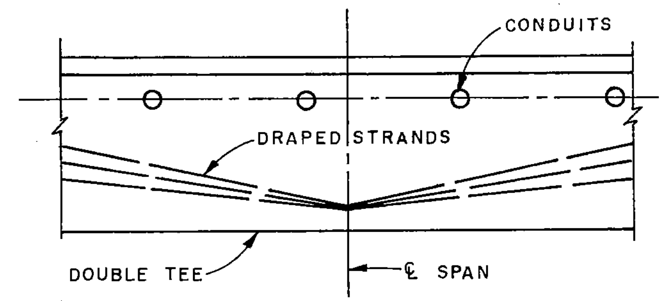

Fig. 4.18. Conduits in double tees.

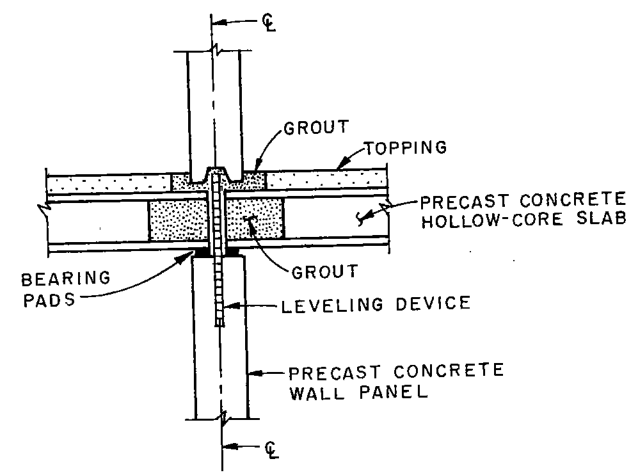

Fig. 4.19. Typical bearing wall panel system joint.

3. Do not provide conduits without assuring positive conduit positional support system during casting of the member.

Detailing Suggestions for Typical Bearing Wall Panel System Joint (Fig. 4.19)

1. Require tighter tolerances for the slab length than are normally expected of these members due to very small bearing width usually available in panel system framing such structures.
2. Provide leveling devices for the wall plumbness for each floor to assure that the tolerances required for vertical plumbness are not exceeded. Again, the main reason is the availability of only small bearing area.

Detailing Suggestions for Staggered Architectural Wall Panels (Fig. 4.20)

1. Check for excessive thermal bowing

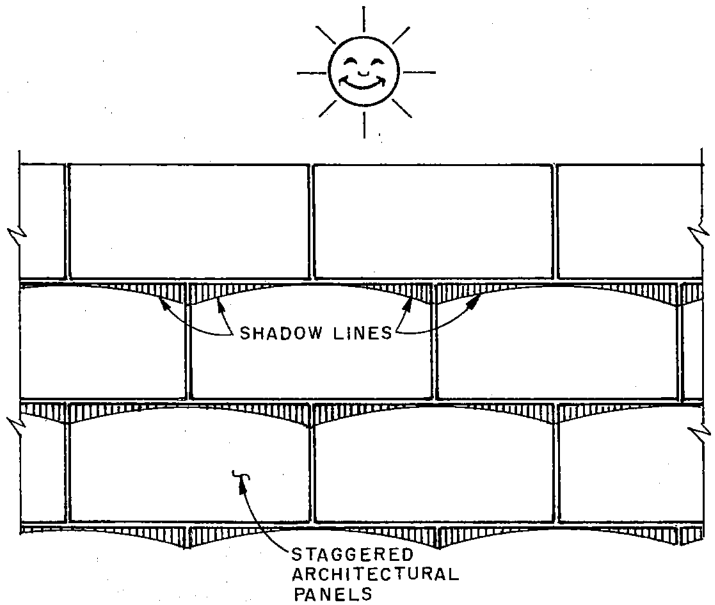

Fig. 4.20. Staggered architectural wall panels.

of panels and set panel tolerances to avoid unpleasant shadow marks, as shown.

2. Consider joint configuration and joint tolerances to minimize shadow effects.

Detailing Suggestions for Typical Architectural Panel Joints (Fig. 4.21)

1. Allow either a chamfered or reveal joint, since these types of joints can accommodate the tolerances required of panel thickness and the shadows formed within these joints will minimize any adverse effects on the aesthetic appearance of the joinery system.
2. As a general rule, the minimum design joint width should not be less than ¾ in. (19 mm). When tolerances are applied to joints designed narrower than this, they may become too narrow to allow effective caulking.
3. Avoid use of butt joint as the tolerances required of panel thickness will form shadow lines directly over the panels rather than within the joints. This will impair the aesthetic appearance of the panels.

Detailing Suggestions for Precast Column Located Near Previously Constructed Wall Corner (Fig. 4.22)

1. Allow pocket in the walls on either of locations as shown, to execute bolted connections marked "X" or provide adequate clearance "A" for easy access to complete this connection.

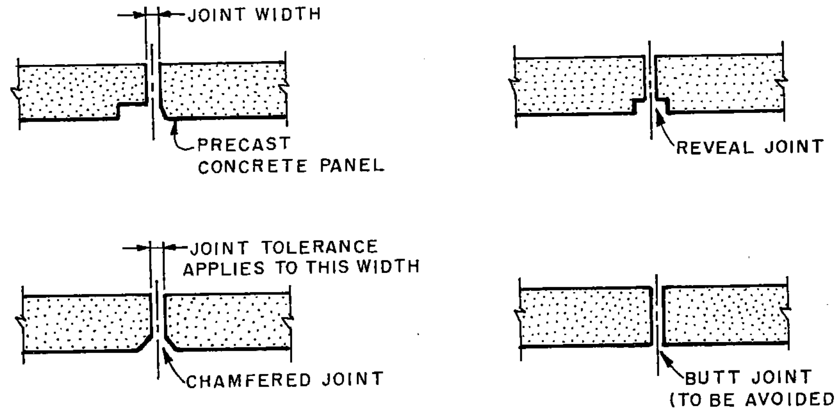

Fig. 4.21. Typical architectural panel joints.

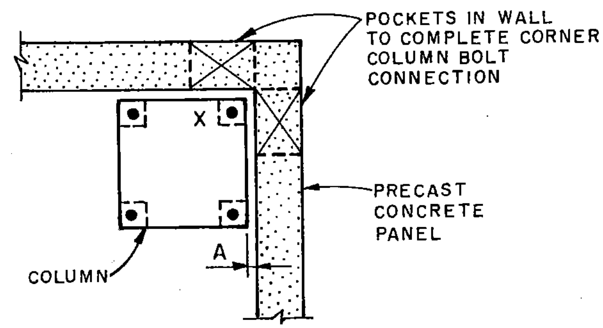

Fig. 4.22. Precast column located near previously constructed wall corner.

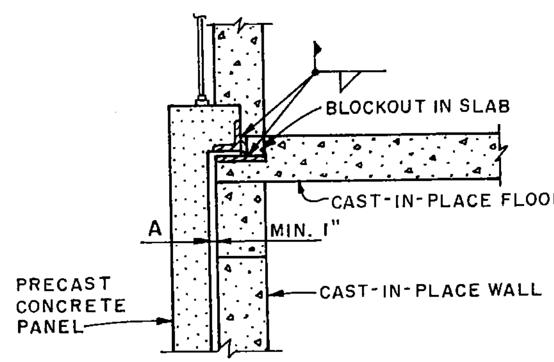

Fig. 4.23. Fascia to cast-in-place slab.

2. Do not assume that the bolted connection marked "X" will be accomplished without adequate provisions made either in the way of clearance for easy access or by other means.

Detailing Suggestions for Fascia to Cast-in-Place Slab (Fig. 4.23)

1. Provide a minimum 1 in. (25 mm) clearance "A" to accommodate tolerances required of the panel thickness and the cast-in-place floor slab. If the minimum 1 in. (25 mm) clearance is used, care must be taken to assure that the cast-in-place and precast tolerances allow a noninterference fit.
2. Keep slab reinforcement away from the blockout required to complete the connection.
3. Provide blockout in the slab with dimensions and tolerances that permit fit of fascia panel.
4. Do not support precast panel at more than two points. If the panel is supported at more than two points, the relative movement of the slab with respect to the panel may redistribute bearing loads in an adverse way.

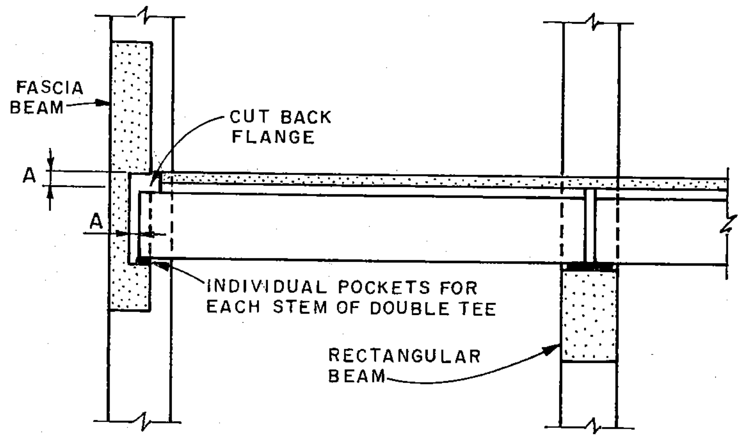

Fig. 4.24. Typical fascia to tee connection.

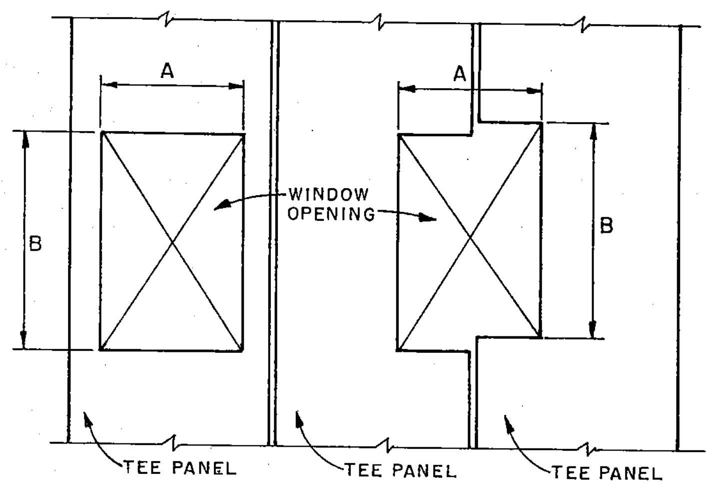

Fig. 4.25. Window opening in tee walls.

Detailing Suggestions for Typical Fascia to Tee Connection (Fig. 4.24)

1. Allow adequate clearance "A" at the fascia end to assure that the tee stems can easily be slid into the pocket provided in the fascia beam.
2. Provide cutback flange wide enough to assure that the flange tees have an adequate clearance from the fascia beam. This way, the flange does not hit the fascia beam when the tee is being held at an angle and is slid into the pockets provided in fascia beams. The maximum dimension of this clearance should consider ease of forming and fire resistance considerations.

Detailing Suggestions for Window Opening in Tee Walls (Fig. 4.25)

1. Consider tolerances required of both window width and caulking between the window jambs and wall panel to determine blockout dimension "A".
2. Consider tolerances required of window height and caulking between window head and wall panel to determine blockout dimension "B".

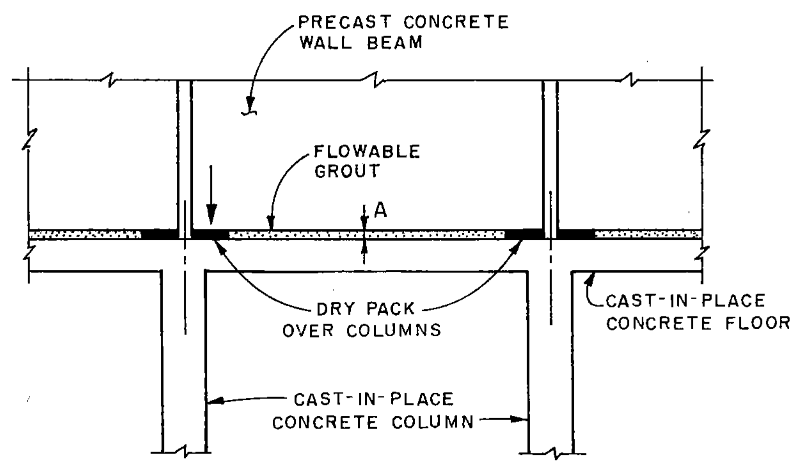

Fig. 4.26. Precast concrete wall beam to column connection.

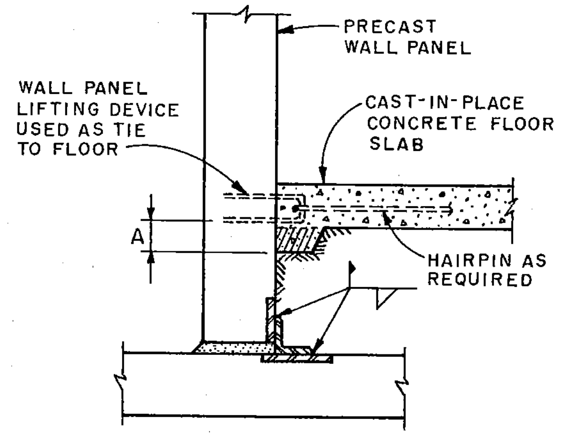

Fig. 4.27. Floor to precast wall connection.

3. Do not locate window blockouts across a wall panel joint (right), since the tolerance required for panel plumbness, panel width, and the joint may cause problems to the proper fit of window units.

Detailing Suggestions for Precast Concrete Wall Beam to Column Connection (Fig. 4.26)

1. Provide adequate clearance "A" to accommodate tolerance required of floor placement in order to avoid load transfer to this floor or eccentric loading to the columns.
2. Do not provide flowable grout over the columns unless special provisions are made to assure positive contact.

Detailing Suggestions for Floor to Precast Wall Connection (Fig. 4.27)

1. Use a tolerance of ±½ in. (±13 mm) from design nominal for location of lifting devices when used as a tie to floor.
2. Use added concrete as shown by cross-hatched area, to accommodate tolerances required to place lifting devices as well as to provide adequate concrete cover "A".
3. Do not use normal tolerances (Chapter 2) for location of the lifting devices if used as a tie to the slab. Such a connection will require tighter tolerances for placement of lifting device.

Detailing Suggestions for Insulated Slender Wall Panels—Corner Detail (Fig. 4.28)

1. Consider the opening of the panel joint at corner caused by thermal bowing of these panels in different directions to assure that any joint opening does not exceed the tolerances required for performance of the caulking compound.
2. Provide a corner connection to resist thermal forces where the panel joint movement exceeds the movement which can be accommodated by the joint sealant.
3. Allow adequate clearance "A" between the columns and the panels so that the tolerances required for the embedded hardware in the precast panels can be easily accommodated.
4. Do not locate columns too close to the panels as it will be hard to execute the corner connection.

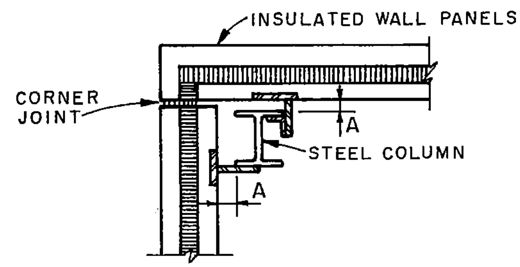

Fig. 4.28. Insulated slender wall panels — corner detail.

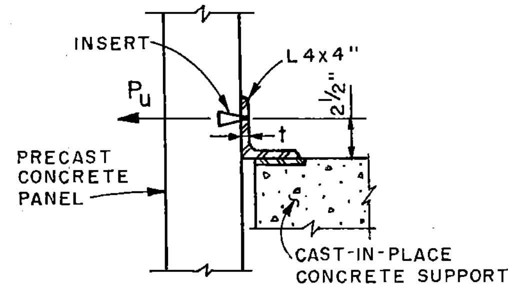

Fig. 4.29. Theoretical tolerances; all dimensions nominal.

4.5 Detailed Examples

A detailed numerical approach to designing details for tolerance considerations is illustrated in this section. Examples which show the consequences of certain tolerance conditions are given.

4.5.1 Clip Angle Used As Lateral Restraint

(1) Discussion

If all the dimensions are at their basic values, as shown in Fig. 4.29:

P_u = 2 kips (8.9 kn)
e = 2½ in. (63 kn)
Angle length = 6 in. (152 mm)
f_y = 36 ksi (248 MPa)
Required angle thickness = ⅜ in. (0.321 in., 8 mm)

However, a vertical slot, as shown in Fig. 4.30, is needed to allow for the following tolerances:

(a) Panel erected ¼ in. (6 mm) higher than nominal (see Chapter 3)
(b) Insert located ½ in. (13 mm) higher than nominal (see Chapter 2)
(c) Beam (steel or cast-in-place) located ⅜ in. (9 mm) lower than nominal (reference AISC or ACI)

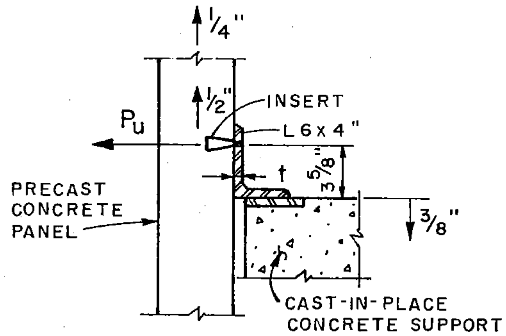

Fig. 4.30. Possible condition with all tolerances adversely combined.

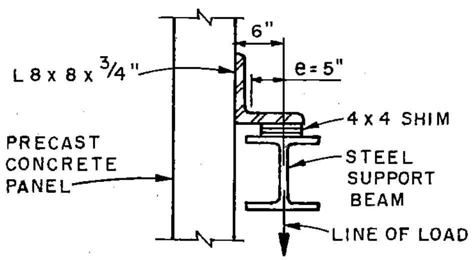

Fig. 4.31. Theoretical tolerances; all dimensions nominal.

Then:

$$e = 2\tfrac{1}{2} + \tfrac{1}{4} + \tfrac{1}{2} + \tfrac{3}{8}$$
$$= 3\tfrac{5}{8} \text{ in. } (92 \text{ mm})$$

Increase angle to 6 x 4 in. (152 x 102 mm)

Required angle thickness = $\tfrac{7}{16}$ in. (0.386 in., 10 mm)

This results in a 45 percent increase in clip angle weight.

(2) Conclusion

Do not assume that all dimensions are the basic dimensions. Use judgment in establishing design parameters, balancing the cost impact of allowing all tolerances to work against each other rather than specifying tighter tolerances.

It may be more realistic to assume that the most probable tolerance condition lies between the nominal dimensions and the extreme. If this assumption is made, the cost impact may be less; however, some field adjustments should be anticipated.

4.5.2 Clip Angle Supporting a Precast Concrete Panel

(1) Discussion

In this case, the designer has decided that the line of load is desired to be at centerline of the supporting beam to avoid torsion in the beam. If all dimensions are basic or nominal (Fig. 4.31), no torsion results from the connection and the angle is designed with an eccentricity equal to 5 in. (127 mm).

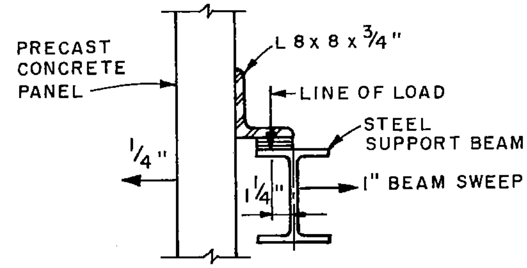

Fig. 4.32. Possible Condition No. 1.

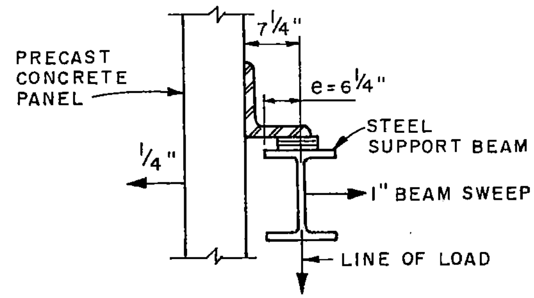

Fig. 4.33. Possible Condition No. 2.

Condition No. 1 (Fig. 4.32) may exist because beam sweep is 1 in. (25 mm) away from the panel (AISC Code of Standard Practice) and the panel is located ¼ in. (6 mm) away from its nominal position (see Chapter 3). To keep eccentricity in the angle at 5 in. (127 mm), the shim must be shifted to the left as shown. This results in torsion in the support beam, which may not be accounted for in its design.

If Condition No. 2 (Fig. 4.33) exists, the eccentricity in the angle increases to 6¼ in. (159 mm), resulting in an angle of increased thickness.

(2) Conclusion

The conclusions for this example are the same as for the previous example.

4.5.3 Precast Corbel With Steel/Steel Bearing

(1) Discussion

If all dimensions are nominal (Fig. 4.34), a corbel designed for an eccentricity equal to 4½ in. (114 mm) would be adequate.

If the situation shown in Fig. 4.35 exists due to the slope of the haunch bearing (see Chapter 2), or if the situation shown in Fig. 4.37 exists due to tipping of the bearing plate (see Chapter 2), the corbel must be designed for an eccentricity of 8 in. (203 mm), an increase of 78 percent.

(2) Conclusion

It is very unlikely that the nominal zero tolerance condition shown in Fig. 4.34 will occur. Therefore, the corbel

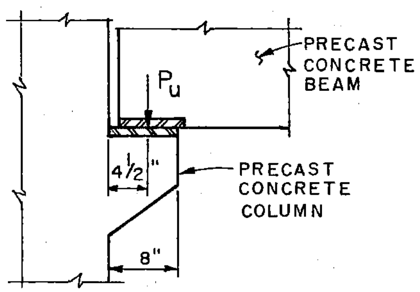

Fig. 4.34. Theoretical tolerances; all dimensions nominal.

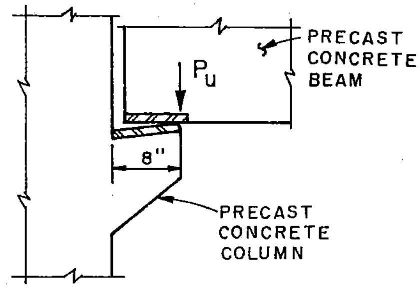

Fig. 4.35. Haunch plate tipped; maximum haunch eccentricity.

should be designed for the most adverse combination of tolerances.

It should also be noted that corbels designed for ultimate loads will support beams experiencing ultimate deflections, which may result in a condition similar to Fig. 4.37. This will occur even when all of the feature dimensions were originally at the nominal values. The use of bearing pads is encouraged to ensure better load distribution.

4.5.4 Effects of Beam Camber

Prestressed floor and roof members usually camber as a result of eccentric prestress force. The camber is a function of the design of the product and is often not "built in" to desired levels.

(1) Discussion

If the effect of camber is neglected, the situation shown in Fig. 4.38 may be the expected condition.

In actual fact, however, the real condition may resemble Fig. 4.39, where a long-span member may have several inches of camber. The floor topping in the example shown has been finished level, without regard to the cambered position of the beam flange surface. The reduced topping thickness at the midspan location may cause problems if not anticipated in design, or excess topping may be required if the design midspan topping thickness is to be maintained and the top of topping elevation is raised. This condition also leads to variations in topping dead load which may differ from design assumptions.

(2) Conclusion

The dimensional effects of design camber, especially of long span members, should be evaluated as part of the design process.

4.5.5 Effects of Camber Variation on Top Flange Connections

(1) Discussion

Camber variation between adjacent beam members can create significant dimensional discontinuities which may make the completion of important connections difficult.

In the section shown in Fig. 4.40, a step of ¾ in. (19 mm) between the flanges is possible, even if both of the long span roof or floor members are within the differential camber tolerance. Since welded diaphragm connections between members of this type are common, the designer should consider how the connection will be made under conditions of adverse tolerance.

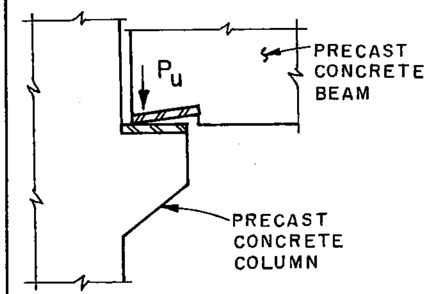

Fig. 4.36. Beam bearing tipped; minimum haunch eccentricity.

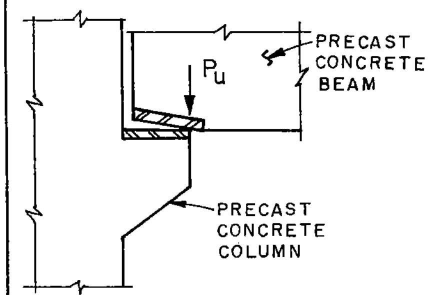

Fig. 4.37. Beam bearing tipped; minimum haunch eccentricity.

(2) Conclusion

This condition can also lead to excessive topping on one member while the adjacent member may receive too little topping.

Specifications for erection should address the maximum allowable difference in adjacent member top elevations if this is a design consideration for either connection effectiveness or topping thickness.

4.5.6 Deflection of Supporting Elements

(1) Discussion

If support member deflection is neglected when a series of small architectural panels are supported on a long span beam, the designer may expect that the final condition will be as shown in Fig. 4.41.

In actual fact, if the supporting beam is very flexible, the final condition may be as shown in Fig. 4.42. The support beam will deflect in increments as each panel is erected, resulting in an in-plane rotation of the panels previously erected. This rotation can result in joint widths as illustrated in Fig. 4.42.

Loading from other sources may also cause a problem. For example, if precast

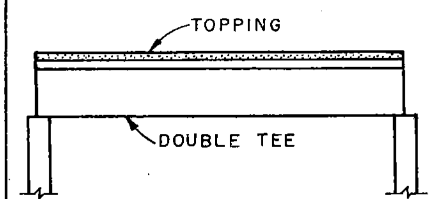

Fig. 4.38. Expected condition caused by neglect of camber effect.

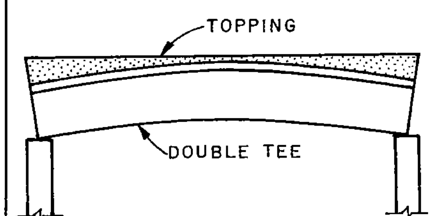

Fig. 4.39. Possible resulting condition.

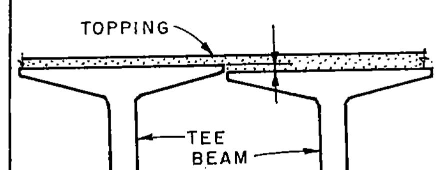

Fig. 4.40. Beam flange joint considerations.

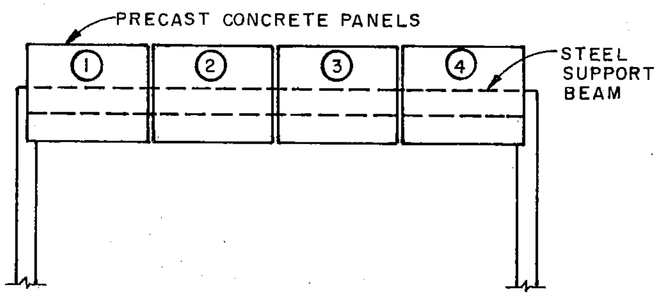

Fig. 4.41. Expected condition caused by neglect of support members deflection.

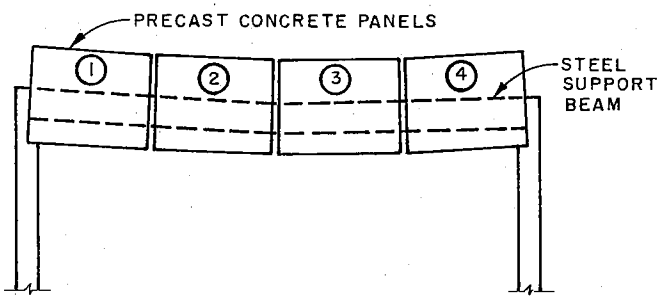

Fig. 4.42. Possible resulting condition.

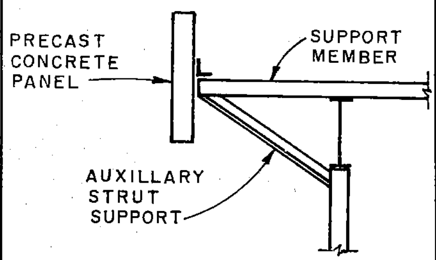

Fig. 4.43. Auxiliary strut support.

concrete is erected prior to floor slab construction, the weight of the floor may deflect the support beam and cause a problem similar to that shown in Fig. 4.42.

(2) Conclusion

This effect can be avoided by either determining the beam intermediate and final deflections, and setting the precast concrete panels out of alignment so the additional deflection will bring them into alignment, or by making adjustments after all of the panels are erected.

4.5.7. Panel Supported by a Cantilever

(1) Discussion

Panels supported by cantilever construction require extremely careful consideration. Often the best way to solve this problem is to use a support scheme that does not rely on cantilever action, such as is shown in Fig. 4.43.

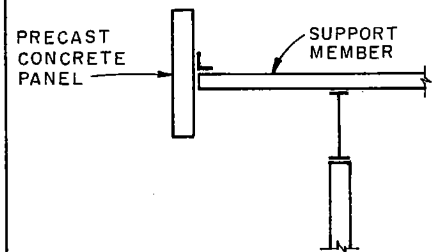

Fig. 4.44. Expected final condition.

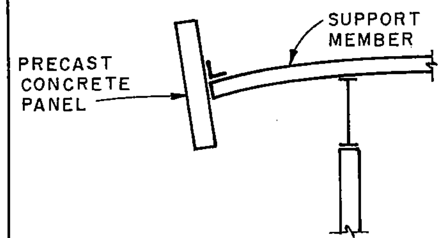

Fig. 4.45. Possible resulting condition.

If the detail in Fig. 4.44 is used, the deflection of the cantilever will not only result in vertical movement of the panel but also in rotation, as shown in Fig. 4.45.

(2) Conclusion

Of particular note is the condition when panels supported on cantilevers are adjacent to panels supported in a different manner. This may result in joint tapers and jogs in alignment.

The possibility of increased deflection and rotation of the panel over time, resulting from creep of the supporting cantilever, must also be considered.

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NOTE: Discussion of this report is invited. Please submit your comments to PCI Headquarters by September 1, 1985.