Considerations for the Design of Precast Concrete Bearing Wall Buildings to Withstand Abnormal Loads

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Presents design guidelines for precast bearing wall buildings subject to abnormal loadings. Emphasizes horizontal, vertical, and peripheral ties sufficient to provide interaction between all building elements. Empirical design approaches, based on various reports, are suggested and some examples are provided in the Appendix.

CONTENTS

CHAPTER 1—OBJECT AND RANGE OF APPLICATION

1.1 Scope of report

This report provides general guidelines for the design of precast concrete bearing wall buildings and is applicable to structures other than one- or two-family buildings.

Except as otherwise indicated in this report, these structures should conform to the latest revision of the ACI Building Code Requirements for Reinforced Concrete, ACI 318-71,[1] or the ACI Building Code Requirements for Structural Plain Concrete, ACI 322-72.[30]

1.2 Types of construction

The following types of construction are governed by this report:

1. Precast wall panels combined with precast floors or roofs
2. Precast wall panels combined with cast-in-place floors or roofs.
3. Precast boxes or modified boxes
4. Combinations of the above.

1.3 Weight of concrete

Walls and floors may be constructed of normal weight concrete or structural lightweight concrete.

1.4 Types of panels

Wall panels may include the following types:

1. Solid concrete
2. Sandwich (insulated)
3. Ribbed
4. Hollow-core
5. Composite

1.5 Types of reinforcement

Structural elements may be reinforced with prestressing steel, welded wire fabric, reinforcing steel, or expanded metal reinforcement.

1.6 Special systems

Nothing in this report should be construed as precluding the use of any special system of large panel construction. The requirements of Section 1.4, ACI 318-71, will govern the approval of such special systems.

CHAPTER 2—DESIGN LOADS

2.1 Load requirements

Service loads, wind, and seismic requirements should be in accordance with the local applicable building code. Design loads are service loads multiplied by an appropriate load factor, determined in accordance with ACI 318-71. Since this report covers the treatment of abnormal loads, a ϕ factor of one should be assumed.

2.2 Lateral load

The structure should be designed for all lateral load requirements of the local applicable building code. As a minimum, the requirements of Section 2.2.1 should be satisfied.

2.2.1—A minimum total design force equal to 2 percent of the service dead load (weight of superstructure plus superimposed dead loads) should be applied as a lateral load at each floor over the height of the structure. The vertical design load is taken as the service dead load. Strength and stability of the structure should be investigated for this combination of loading.

2.3 Volume changes

Effects of forces due to restrained creep, shrinkage, temperature change, and differential settlement are to be considered.

2.4 Load distribution

The stiffness and the location of the lateral load resisting elements relative to the center of rigidity of the entire structure should be considered when distributing loads.

2.5 Design loads

Forces given in this report are design loads, and are not to be factored.

CHAPTER 3—STABILITY

3.1 Bracing elements

Large panel structures should be arranged with bracing elements that provide lateral load resistance in two perpendicular vertical planes.[10] Bracing elements may be loadbearing walls, non-loadbearing shear walls, or moment-resisting frames. Some possible arrangements are indicated in Chapter 5.

3.2 Ties

Vertical and horizontal tensile ties should be provided in walls, floors, and roofs of all structures over two stories in height (see Fig. 1). Ties are to be interconnected. Unless analysis indicates a larger value, the size and the spacing of these ties should be

Committee Statement

This report provides design guidance and minimum provisions which represent the best judgment of the Committee at the time the report was prepared.

The objective is to have precast concrete bearing wall buildings designed and built with a degree of continuity and ductility that will develop a reasonable resistance to the effects of abnormal loading.

It is not intended to represent that this report is the only procedure to reach the objectives; design engineers should be permitted design latitude provided they can show adequate performance.

Designers and manufacturers are invited to discuss and comment on the design considerations given in the report. Subsequently, it is intended to have the report up-dated and extended upon receipt of research data currently being developed, to consider the experience of designers familiar with this type of structure, and to formalize the recommendations in this document for building code adoption.

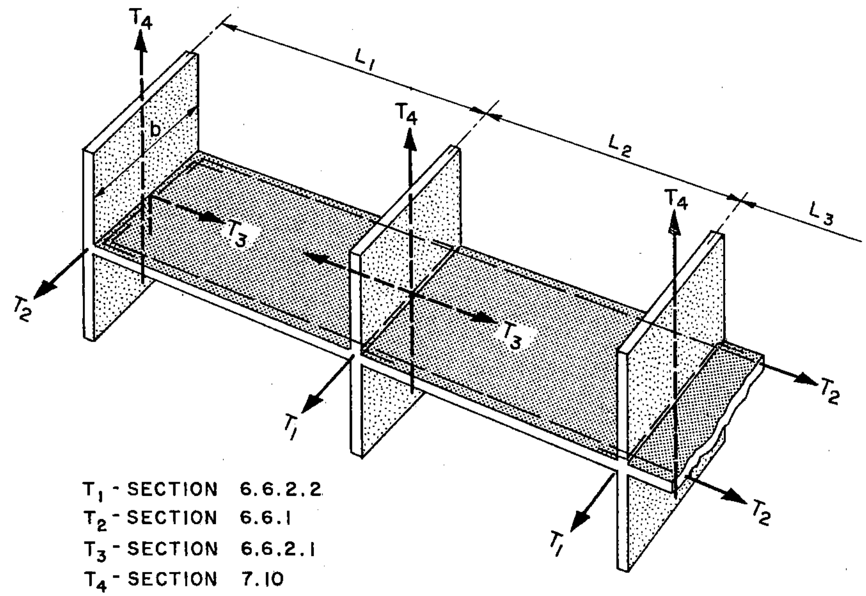


Fig. 1. Recommended tie forces in precast concrete bearing wall buildings.

determined in accordance with Chapters 6 and 7.

3.3 Continuity

Bracing elements should be continuous the full height of the structure to insure stability at all levels. Where discontinuities occur, reinforcement of local elements should insure transfer of strength across the discontinuity.

3.4 Load distribution

Where structural stability is provided by a combination of bracing walls and moment-resisting frames, consideration must be given to the possible redistribution of loads to the bracing walls resulting from the formation of plastic hinges in the frames at design loads.[11]

3.5 Erection

Stability during erection must be considered. The contract documents should require that the Contractor review his erection procedure with a Structural Engineer.

CHAPTER 4—FOUNDATIONS

4.1 Supporting elements

Loadbearing panels and shear walls may be supported by continuous footings, isolated piers, grade beams, or transfer girders. In determining stress distribution in both panels and supporting elements, consideration should be given to the relative flexibility of the loadbearing panels or shear walls, and of their supporting elements.

4.2 Long-term effects

The sequence of construction, differential settlement, and long-term effects of creep should be considered when establishing critical loads on the substructure.

4.3 Differential settlement

Foundations are sized and detailed to minimize differential settlement of the completed structure. Differential settlement and rotation should be considered in the design.

CHAPTER 5—STRUCTURAL SYSTEMS

5.1 Types of wall panel systems

Large panel buildings are differentiated by the general arrangement of loadbearing walls (see Fig. 2):

1. *Cross wall system*—Loadbearing walls are parallel to each other and perpendicular to the longitudinal axis of the structure.
2. *Spline wall system*—Loadbearing walls are parallel to each other and to the longitudinal axis of the structure.
3. *Mixed systems*—Combination of cross wall and spline wall systems.

5.2 Box systems

Box systems, made up of walls and floor slabs, may employ tubular elements cast as integral units or individual components which are then assembled with connections to insure integral behavior. Floors may be included within the tubular element, or they may be precast separately. Box systems are differentiated by their arrangement in the structure; they may be contiguous, or they may be separated by infill floor panels. Box systems may be arranged to form moment-resisting frames in one direction, and to act as shear walls in the perpendicular direction.

5.3 Diaphragm action

The floors of large panel structures usually serve as horizontal diaphragms, distributing applied lateral loads to the vertical bracing elements. Floors and walls may also be considered to act as moment-resisting frames.[3] In this case, particular care must be taken to insure the integral behavior of the joints, including consideration for long-term creep and shrinkage. Special consideration

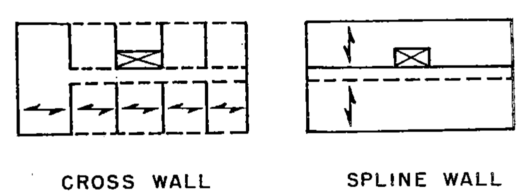


Fig. 2. Arrangement of loadbearing walls.

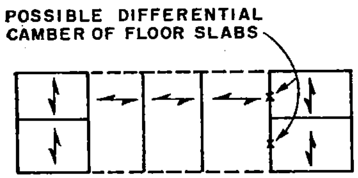


Fig. 3. Floor spans in different directions.

should be given to the interconnection of precast floor elements in Seismic Zones 2 and 3.[34]

5.4 Structural layout

In choosing the structural layout of panels, consideration should be given to arranging the direction of floor spans so that gravity loads of the floors resist uplift caused by lateral loading. Such an arrangement will require special attention to the possible differential camber of adjacent floor panels spanning perpendicular to each other (see Fig. 3).

5.5 Expansion joints

Expansion joints may be located as required by local climatic conditions, by the arrangement of stiffening elements, and as necessary to accomodate effects of shrinkage and creep. In general, they should not be spaced more than 180 ft unless special consideration is given to the effects of creep, shrinkage, and thermal conditions. Expansion joints should also be considered at abrupt changes in the structure.

Generally, expansion joints separate the superstructure for its full height. Additional localized expansion joints may be required, as determined by the characteristics of the structure. Expansion joints in Seismic Zones 2 and 3 should be arranged to prevent hammering.

5.5.1—Special attention should be given to portions of buildings which may be subject to extreme differential temperature changes (rooms containing high temperature equipment, exterior portions of structure attached to interior portions, etc.).

5.5.2—Loadbearing elements at expansion joints should be connected to the fixed side floor or roof slab in accordance with the requirements for horizontal ties in floors and roofs, as given in Chapter 6.

CHAPTER 6—FLOORS AND ROOFS

6.1 Cast-in-place members

Cast-in-place floors and roofs should be designed and constructed in accordance with ACI 318-71.

6.2 Precast elements

Precast floor and roof elements may consist of solid slabs, hollow-core slabs or ribbed units, prestressed or conventionally reinforced. These elements are designed in accordance with ACI 318-71 and the following additional requirements:

6.2.1—If intended to provide diaphragm action, precast elements with continuous grouted keyways between the elements may be considered fully effective when floor ties are provided in accordance with this report. Mechanical connections may be used in lieu of, or in combination with, grouted keyways. A structural cast-in-place concrete topping, properly reinforced and bonded to the precast elements, may also be considered to act as a diaphragm in combination with the precast elements.

6.2.2—Precast floor and roof elements with minimal or no top reinforcing steel at the supports should have a sufficient bearing length to safely transfer applied loads by direct bearing, but in no case less than:

1. 2½ in. for solid and hollow-core slabs.
2. 3½ in. for ribbed slabs, including solid slabs with projecting nibs.

The length of bearing is measured from the face of the wall, except that, if the wall is chamfered more than ½ in., the bearing length is measured from the inside edge of the chamfer. The above minimum bearing lengths may be reduced if bearing surfaces are adequately armored. For this special condition, the usual manufacturing and erection tolerances may require modification, as determined by the Engineer.

6.2.3—Precast floor and roof elements, with sufficient top reinforcement at the supports to resist calculated support moments, may have reduced bearing lengths, provided the joint is designed to transfer floor loads to the supporting bearing panels. Permissible bearing length may be determined by analysis or tests. Consideration should be given to the possible need for bottom reinforcement across the joint to prevent separation due to creep and shrinkage, and the subsequent loss of contact of precast element and joint fill concrete.

6.3 Tolerances

Manufacturing and erection tolerances should be considered in determining the bearing lengths used in design. The contract documents should indicate the tolerance required by the Engineer of Record.

6.4 Restraint

For simple spans, the effect of a possible clamping action of wall panels on precast floor elements, and the resulting restraint against translation and rotation, should be considered. Continuity of precast floors and roofs may be considered if the following requirements are satisfied:

1. Negative reinforcement, as determined by elastic analysis, is spaced not greater than three times the floor or roof thickness.
2. Floor tie requirements of Section 6.6 are provided in addition to the negative reinforcement as determined by analysis.

6.5 Ties

Horizontal tensile ties should be provided in floors and roofs, or within the joints of precast elements, to satisfy all loading requirements, and to perform the following functions:

1. Resist forces which may be induced by erection eccentricities.
2. Transfer to floor diaphragms the horizontal reactions due to lateral loads on wall panels.
3. Transfer horizontal reactions due to applied lateral loads on shear walls.
4. Resist tensile forces due to restrained volume changes, differential settlement, and lateral loads.

6.6 Minimal tie requirements

The following minimal requirements for horizontal floor and roof ties (Fig. 1) must be satisfied. Tie capacity is based on yield stress.

6.6.1—At each floor and roof level a continuous peripheral tie, T_2, capable of resisting a design force sufficient to develop diaphragm action, but not less than 16,000 lb, should be provided within the depth of floor or roof element, or sufficiently close to these elements and connected thereto. The tie may be reinforcing steel in the grout joint or continuous reinforcing steel (including prestress) in an edge beam. Reinforced spandrels or walls adequately anchored to the floor or roof may also be considered.

If ties are interrupted by offsets in the structure, supplemental reinforcement properly spliced can be provided to insure effective continuous tie action. Where buildings are provided with expansion joints, each section must be treated as an entity. A precast floor or roof slab may be considered to be an edge beam, provided the continuous tie reinforcement is located within 4 ft of the perimeter of the structure.

6.6.2—At each floor and roof level, continuous longitudinal and transverse internal ties should be provided at approximately right angles across the building. Ties may be mechanically anchored or spliced to the peripheral reinforcement at each end. At openings in floors or roofs the total tie force must be maintained by supplemental reinforcement, properly anchored. Longitudinal and transverse ties should be in accordance with the following:

6.6.2.1—Longitudinal ties, T_3 (in the direction of the floor span), connecting floor or roof elements that abut over internal walls, or connecting external bearing walls with floor or roof, should be capable of resisting a design force of 2½ percent of the service load on the wall, but not less than 1500 lb per lin. ft of distance measured perpendicular to the span. These ties should be spaced no greater than 8 ft on centers. They may project from the precast floor or roof slabs, or be embedded within joints between the precast elements.

Reinforcement must be of sufficient length and with sufficient concrete cover on all sides to develop the specified design force. When so developed, the tie force beyond the end of this reinforcement may be considered to be provided by the normal reinforcing steel in the floor or roof slabs. Longitudinal ties at exterior bearing walls, or other bearing walls which may be loaded from one side only, can develop the specified forces by projecting wall reinforcement into the floor, or by providing a properly designed mechanical anchorage between floor and wall.

End walls may also be braced by structural walls or frames perpendicular to these end bearing walls. The bracing elements may comprise a stair tower, elevator tower, or similar components. When such bracing elements are used, connections between the bracing elements and the bearing walls should be sufficient to resist a design force of 2½ percent of the service load on the bearing wall but not less than 1500 lb per lin. ft of distance measured perpendicular to the span at each floor level.

6.6.2.2—Ties, T_1, transverse to the direction of the floor or roof span should be capable of resisting a design force not less than 1500 lb per lin. ft of distance measured parallel to the direction of floor or roof span. These ties may be encased in the floor or roof units, may be placed in a structural topping of thickness not less than 2 in., or may be concentrated at or in the transverse walls. Reinforcement located in the transverse walls may be placed within the depth of the floor, or at a distance from the floor not greater than the floor thickness.

6.6.2.3—Flexural and temperature reinforcement provided within floor or roof slabs may be considered to provide part or all of the above specified forces when properly detailed and anchored.

6.7 Shear keys

For precast floor or roof slabs with properly grouted shear keys or mechanical connections,[4] the line load from partitions parallel to the direction of the floor span may be assumed distributed in accordance with the lesser of the following:

1. Distribution as determined by structural analysis, considering flexural strength and shear strength in the lateral direction.

2. One-half the clear span length of the floor.

Note: All values recommended in this chapter may be varied in accordance with the procedures of Section 1.6.

CHAPTER 7—LOADBEARING WALLS

7.1 Design requirements

A wall is here defined as a vertical loadbearing element whose length is equal to or greater than four times its thickness.[4]

The vertical load-carrying capacity of walls may be determined in accordance with ACI 318-71, Chapter 10 or Chapter 14.

Loadbearing walls whose thickness is at least 1/25 of its unsupported height may be designed in accordance with the requirements of ACI 322-72.

7.2 Bracing

Loadbearing panels should be braced. This bracing may take the form of transverse walls directly connected to the loadbearing panel, or connection of the loadbearing panel through floor diaphragm action to transverse walls located remote from the panel.[2]

When applying the design procedures of ACI 318-71, the value of k may be determined in accordance with the following:

7.2.1—Unless analysis indicates that a smaller value is acceptable, the value of k should be taken as one for panels braced against sidesway.

For panels braced against sidesway, the effect of slenderness may be neglected when

$$kL/r < 34 - 12\,(M_1/M_2) \qquad (1)$$

where

k = effective length factor for compressive members

L = height of panel between supports

M_1 = value of smaller design end moment on panel, positive if bent in single curvature, negative if bent in double curvature

M_2 = value of larger design end moment on panel, always positive

r = radius of gyration

7.2.2—For panels braced against sidesway, and connected on all four edges, the value of k may be taken as:

$$k = 1 \text{ for } (L/b) < \tfrac{1}{2} \qquad (2)$$

$$k = 1\tfrac{1}{2} - (L/b) \text{ for } \tfrac{1}{2} \leq (L/b) \leq 1 \qquad (3)$$

$$k = \frac{1}{1 + (L/b)^2} \text{ for } (L/b) > 1 \qquad (4)$$

where b is the length of panel between supports.

7.2.3—For panels braced against sidesway, and connected on one vertical edge and at top and bottom, the value of k may be taken as:

$$k = 1 \text{ for } (L/b) < 1 \qquad (5)$$

$$k = 1 - 0.423[(L/b) - 1] \text{ for } 1 \leq (L/b) \leq 2 \qquad (6)$$

$$k = \frac{1}{[1 + 0.5(L/b)^2]^{1/2}} \text{ for } (L/b) > 2 \qquad (7)$$

7.2.4—For panels braced against sidesway, and connected at top and bottom only, the value of k should be taken as one.

7.3 Connections

In order to be considered as effectively braced along an edge, the loadbearing panel must be adequately attached to another component which is approximately at right angles to the bearing wall, and which has a length not less than one-fourth of its clear height between floors. Equivalent rigidity may be provided by other acceptable methods.

Attachment between walls may be achieved by providing a mechanical connection (e.g., bolting or welding), or by constructing a reinforced concrete joint (Fig. 4). The connection should be able to resist the design force (factored dead load plus live load) determined by static analysis, plus a bracing force of 2½ percent of this vertical design force carried along the length of lateral support.

If loadbearing panels are pierced with significant openings, the portion between such openings should be considered as unbraced along the vertical edges bounded by the openings.

7.4 Effective thickness

The effective thickness of loadbearing walls for purpose of design may be determined in accordance with the following criteria:[20]

7.4.1—The effective thickness of solid concrete panels without a surface treatment is taken as the full thickness.

7.4.2—The effective thickness of solid concrete panels cast monolithically with a surface facing having a compressive strength equal to or greater than the concrete backup of the panel is taken as the total panel thickness less the average depth of the reveal of any exposed aggregate, when the average depth of the reveal exceeds 3 percent of the total panel thickness.

7.4.3.—The effective thickness of solid concrete panels that have a separate facing, not cast monolithically with the panel, is taken as the total panel thickness less the facing thickness, unless adequate means are provided to insure interaction.

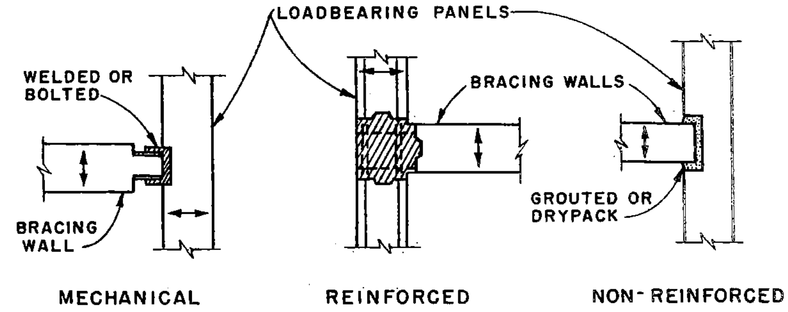


Fig. 4. Typical wall-to-wall connections (plan views).

7.4.4—The effective thickness of ribbed panels cast monolitically may be conservatively taken as three times the radius of gyration of the panel in the direction in which stability is being considered.

7.5 Flange width

The effective flange width of symmetrical ribbed panels should not exceed the least of:

1. One-fourth the height of the panel between supports.
2. Eight times the thickness of the flange on either side of the stem.
3. The center-to-center spacing of the stems.

The effective projecting flange width for unsymmetrical panels should not exceed the least of:

1. One-twelfth the height of the panel between supports.
2. Six times the thickness of the flange on either side of the stem.
3. One-half the clear distance to the next rib.

7.6 Hollow-core panels

Hollow-core panels should be designed on the same basis as solid panels, with due consideration given to the voids in determining section properties and strength.

7.7 Sandwich panels

Sandwich panels consisting of two layers of concrete separated by a non-structural insulation core should have the two layers sufficiently interconnected, preferably by mechanical shear ties (Fig. 5), so that both layers can be considered structurally effective. Shear stresses are not to be assumed resisted by the non-structural insulation core unless confirmed by test. Compressive and flexural stresses are assumed to be carried by the concrete only.

The spacing of ties should be such that the slenderness ratio of the concrete layer supporting load does not exceed the slenderness ratio of the

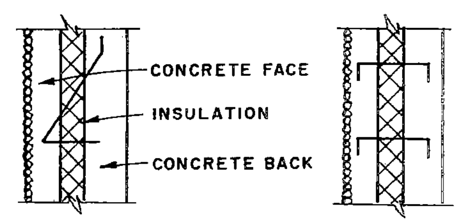


Fig. 5. Ties in sandwich panels.

overall panel, unless it can be demonstrated that a larger spacing will not affect the overall behavior of the panel.

Where the outer layer of concrete is not directly supported on the panel below, the ties must be designed to transfer the weight of this layer through the core to the loadbearing layer. Consideration must also be given to the effects of differential shortening between loaded and unloaded layers.

When vertical loads are applied to one layer only, it should be assumed that this layer resists the total applied load. When considering the effect of slenderness for this case, the overall thickness of the sandwich panel may be used provided ties and/or bond are sufficient to insure composite action.

7.8 Reinforcement

Wall panel reinforcing practice varies. Experience indicates that the following general guidelines are appropriate:

7.8.1—Casting and handling methods chosen to minimize the flexural tensile stresses are more effective than reinforcing steel in the control of cracks. The calculated tensile stress on the effective concrete section should preferably not exceed $5\sqrt{f'_{ci}}$, where f'_{ci} is the concrete compressive strength at time of stripping.

7.8.2—Areas of abrupt change in the cross section of the panel should be reinforced. For panels less than 6 in. thick, provide at least one No. 4 bar around openings. For panels 6 in. and thicker, provide at least two No. 4 bars.

Bars should be continuous around openings, or extend 2 ft past openings on all sides which is sufficient to develop the bar. Other sizes and arrangement of reinforcement, which has been demonstrated as sufficient to prevent cracking, may be used.

7.8.3—Loadbearing walls should be reinforced with an amount of steel of at least $0.001bL$, both vertically and horizontally, unless designed in accordance with ACI 322-72. Spacing of reinforcement shall not exceed 30 in. for interior walls or 18 in. for exterior walls.

7.8.4—At points of significant local bearing stress, panels should be reinforced against splitting. Design may be in accordance with Part 6 of the *PCI Design Handbook*.[33]

7.9 Concentrated loads

Concentrated loads on wall panels should conform to the following:

7.9.1—Concentrated loads may be assumed to be distributed uniformly within a zone as indicated in Fig. 6.

7.9.2—Bearing stresses under concentrated loads should not exceed the values determined by the methods of ACI 318-71, Section 10.14, or ACI 322-72, Section 7.10, whichever document is used for the design of the panels.

7.9.3—For uniform bearing on unreinforced concrete, without provision for confinement reinforcing, the design provisions of the *PCI Design Handbook*, Section 6.1.6, will govern.

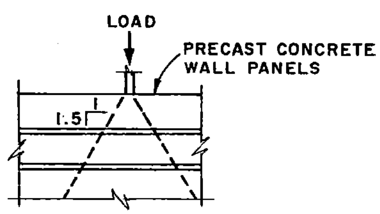


Fig. 6. Distribution of concentrated loads on walls.

7.9.4—For uniform bearing on concrete with confinement reinforcing, the design provision of the *PCI Design Handbook*, Section 6.1.9, will govern.

7.9.5—Where loads are distributed across vertical joints in panels, design of the joint must provide for the transfer of these loads.

7.10 Buildings over two stories

In buildings over two stories in height, continuous vertical ties (T_4 in Fig. 1) should be provided from foundation to roof in all loadbearing walls and structural shear walls. The capacity of these ties, based on yield stress, must be sufficient to resist any calculated tensile force (uplift, shear-friction), but should, as a minimum, provide a design capacity of 3000 lb per lin. ft of wall. A minimum of two ties should be provided per wall panel. Where panels have significant openings, consideration must be given to the possible need for additional vertical ties between such openings.

7.11 Design eccentricities

Loadbearing walls which are braced laterally may be designed assuming no moment transfer to the floors, unless the connection between wall and floor has been designed and reinforced to transmit moments.

In establishing design eccentricities, consideration should be given to:

1. Erection tolerances
2. Manufacturing tolerances
3. Joint details
4. Lateral forces, such as wind and seismic

The minimum total eccentricity is 0.6 in. (see ACI 318-71[1]).

7.11.1—The resultant eccentricity of applied loads, with respect to the neutral axis of the panel at the point under consideration, may be determined using Reference 3:

$$e_a \text{ (or } e_b) = \frac{P_1 e_1 + P_2 e_2}{P_1 + P_2} \qquad (8)$$

where

P_1 = design loads from wall above, considering level "a" or level "b"
e_1 = eccentricity of P_1
P_2 = design loads from floor at level under consideration
e_2 = eccentricity of P_2
e_a = eccentricity at top of panel
e_b = eccentricity at bottom of panel

7.11.2—When employing the design procedures of ACI 322-72, the design value of eccentricity, e, shall be taken as the average of e_a and e_b (see Fig. 7).

7.11.3—The value of e_1 is a function of the manufacturing and erection tolerances, and the location of the bearing surface of the upper

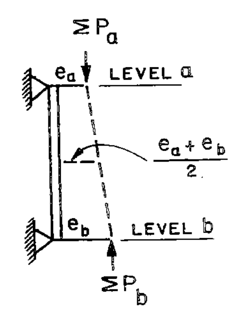


Fig. 7. Average eccentricity on a wall panel.

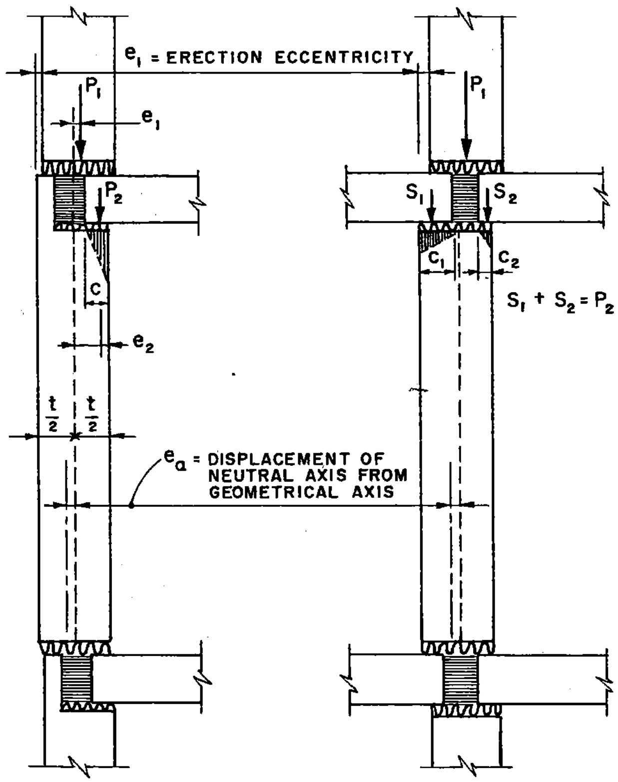


Fig. 8. Development of eccentricities in wall panels.

wall panel on the joint (see Fig. 8).

7.11.4—The value of e_2 may be determined by statics:

For wall panels loaded on both sides

$$e_2 = \frac{S_1(t/2 - c_1/3) - S_2(t/2 - c_2/3)}{S_1 + S_2} \tag{9}$$

For wall panels loaded on one side

$$e_2 = t/2 - c/3 \tag{10}$$

where

S_1, S_2 = floor loads

c_1, c_2 = bearing length of loads S_1 and S_2

t = thickness of panel

7.12 Shear transfer

Wall panels which are intended to act as beams are designed in accordance with the methods of ACI 318-71. Where more than one-story height of wall is intended to act integrally, provision must be given to the transfer of horizontal shears at the joints by means of positive interconnection of panels.

7.13 Transfer girders

The design of wall panels supported by transfer girders should consider the effective stiffness of the connected elements. One design method is given in Appendix 1.

CHAPTER 8—SHEAR WALLS

8.1 Ductility

Large panel structures constructed in accordance with this report should be considered nonductile, unless they are continuously reinforced for their full height, in accordance with Reference 16, or are post-tensioned to insure ductile behavior.

8.2 Lateral resistance

The resistance to lateral load in large panel structures is usually provided by shear walls, which may also be loadbearing walls. Lateral resistance may be obtained by the combined action of shear walls and moment-resisting frames, or by connecting walls and floors to form moment-resisting frames. In the latter case, special precautions should be taken to insure the integrity of the joint, in accordance with Section 5.3.

8.3 Bracing

Shear walls may be considered as braced at each floor when the floor and walls are tied in accordance with Section 6.6. When shear walls extend beyond the edge of the floor (Fig. 9), the projecting portion may be considered as braced provided that:

1. The floor is tied to the wall in accordance with this report.
2. The projecting portion considered as braced does not exceed twelve times its thickness.

8.4 Transverse walls

Shear walls may be assumed to act together with transverse walls to form a monolithic flanged section when properly connected to transmit joint stresses (unit shearing stress, VQ/I) plus the effects of differential shortening.

Where interior and exterior walls intersect, consideration must be given to the effect of temperature differential.

The effective length of flange wall on each side of a shear wall (Fig. 10) is taken as the least of the following, unless a more accurate analysis, including the effect of shear lag, is made:

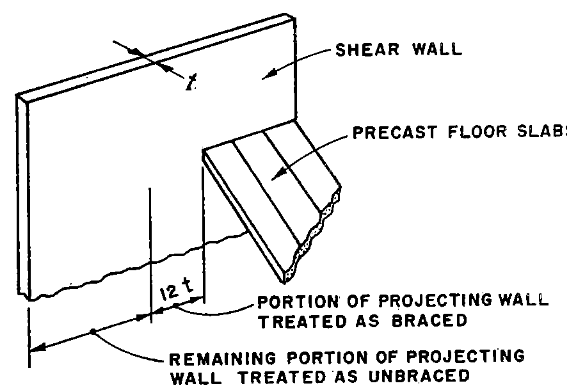


Fig. 9. Bracing beyond floor edges.

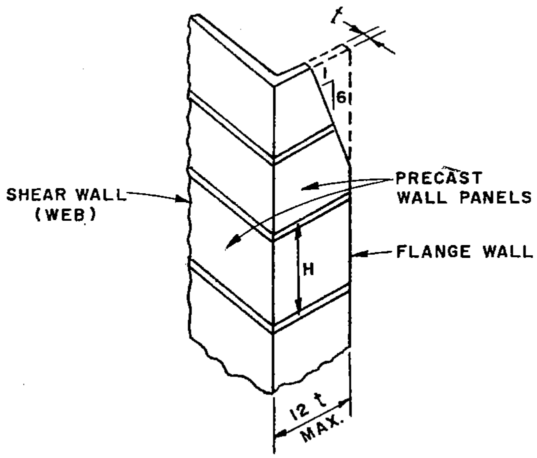


Fig. 10. Effective limits for flange walls.

1. The distance from the shear wall to any significant opening in the flange wall.
2. One-sixth the distance from the level under consideration to the top of the flange wall.
3. Twelve times the thickness of the flange wall.

8.5 Openings

Shear walls with large openings should be designed as coupled walls, considering the stiffness of the connecting beam.[15,21]

8.5.1—Walls so coupled should be in the same plane. The analysis should also consider the support conditions at the base of the wall.

8.5.2—Beams designed to couple shear walls should be reinforced for flexure and shear. The total design shear in the beam should be taken by closed stirrups, with no allowance for the shear resistance of plain concrete. Distribution of flexural reinforcement should be in accordance with ACI 318-71, Section 10.6.

8.5.3—The effect of the floor in contributing to the stiffness of the beam may be considered when the floor is cast-in-place or of composite construction and properly connected to the shear wall and to the beam; otherwise it should be neglected.

8.5.4—Proper coupling of in-plane walls will significantly reduce their deflection. When a cast-in-place connection between walls is provided, the effect of reduced rigidity due to any cracking of the connection should be considered.

8.6 Flexural strength

The flexural strength of shear walls[17] may be determined in accordance with the following:

8.6.1—Vertical reinforcing steel concentrated at the ends of the wall. For this condition, the usual theory of flexure is applicable.

8.6.2—Vertical reinforcing steel distributed along the length of the wall. For this condition, a strain compatibility analysis is required to determine flexural strength. In lieu of detailed analysis, the following equation may be used:

$$M_u = (\phi/2)A_s f_y L_w \left[1 + \frac{N_u}{A_s f_y}\right] \times \left[1 - \frac{c}{L_w}\right] \qquad (11)$$

where

M_u = design resisting moment of the wall
A_s = total area of vertical reinforcement
f_y = specified yield strength of reinforcement
L_w = horizontal length of shear wall
N_u = design axial load, positive if compression
c = distance from extreme compression fiber to neutral axis
ϕ = capacity reduction factor, here taken as one

The value of c/L_w may be determined from the following:

$$\frac{c}{L_w} = \frac{\dfrac{\rho_v f_y}{f'_c} + \dfrac{N_u}{L_w t f'_c}}{\dfrac{2\rho_v f_y}{f'_c} + 0.5\beta_1} \qquad (12)$$

where

t = thickness of wall
β_1 = factor defining rectangular stress block (ACI 318-71, Section 10.2.7)
$\rho_v = A_s/(L_w t)$
f'_c = specified 28-day concrete compressive strength

8.7 Shear strength

The shear strength of walls may be determined in accordance with ACI 318-71, Section 11.16.

8.8 Uplifts

Depending on their location, shear walls may also be loadbearing walls.

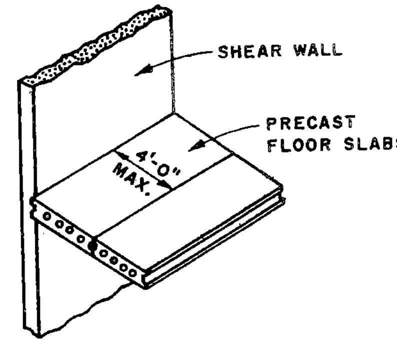


Fig. 11. Effective floor load on parallel shear wall.

When shear walls are not bearing walls, uplifts may occur under lateral loading. It is often advantageous to "load" these walls to counteract the calculated uplifts. One method is to post-tension the walls. Other methods are to tie the parallel floor span edge to the shear wall, or to tie the shear wall to a loadbearing wall.

For slabs spanning parallel to the wall and connected thereto, it may be assumed conservatively that a width of floor not in excess of 4 ft or one slab width, whichever is less (Fig. 11), contributes load to that part of the shear wall exhibiting uplift, provided the connection between wall and floor is sufficient to transfer such loads.

8.9 Stiffness

Unless a stability analysis includes the effects of lateral deflection (P-Δ effect), the stiffness of all lateral load-resisting elements within a structure (in the direction considered) must be such that the following criteria are satisfied:[6]

$$H\sqrt{P/(EI)} \leq 0.6 \text{ for } n > 4 \qquad (13)$$

$$H\sqrt{P/(EI)} \leq 0.2 + 0.1\,n \quad \text{for } 1 < n \leq 4 \qquad (14)$$

where

H = total height of structure
n = number of stories in structure
P = sum of all vertical service loads on structure
EI = sum of bending stiffness of all restraining elements in direction considered, assuming the sections are uncracked

The units in Eqs. (13) and (14) must be consistent.

8.10 Lateral deflection

Lateral deflection (drift) under service loads should not exceed $H/600$.

The differential lateral deflection between adjacent walls resisting lateral loads must be limited so as not to cause distress of any member attached to these walls. Otherwise, these members must be designed so that they can tolerate the differential movement.

Any reduction in wall stiffness due to the characteristic behavior of joints within that wall (both horizontal and vertical joints) should be considered in calculating lateral deflection (see Chapter 9).

CHAPTER 9—JOINERY

9.1 Load transfer

Properly designed and constructed joints are an important factor in assuring load transfer without distress, and in achieving stability of large panel structures. Joints represent discontinuities and may be the location of significant stress concentrations. The importance of correct detailing practice, with consideration given to the quality and conditions of site work, is emphasized.

9.2 Vertical joints

Vertical joints act primarily to transfer vertical shear forces between elements. In addition, the joint may be required to resist flexure and shear in a direction perpendicular to the plane of the walls. Factors affecting the ability of vertical joints to transfer load include:

1. Shape of the joint.
2. Reinforcement within the joint and across it.
3. Quality of concrete within the joint and in the adjacent wall panels.
4. Condition of the contact surfaces.
5. Curing conditions of the joint material.
6. Presence of reinforcing ties, within the floors, which pass across the joint.
7. Strength of mechanical connections or castellations (vertically spaced keys) within the joint.

9.3 Types of vertical joints

With respect to the behavior of precast walls subject to lateral load, vertical joints may be classified[23] as rigid, semi-rigid, or open (see Fig. 12).

9.3.1 Rigid—The shear deformation of the joint is of the same order of magnitude as the shear deformation of the panel. A rigid joint will provide for full interaction of the connected panels up to design load.

9.3.2 Semi-rigid—The shear deformation of the joint is significantly more than the shear deformation of the panel. A semi-rigid joint will generally provide full interaction until such load (less than the design load) is reached that the joint will deform at a faster rate than the adjacent panels.

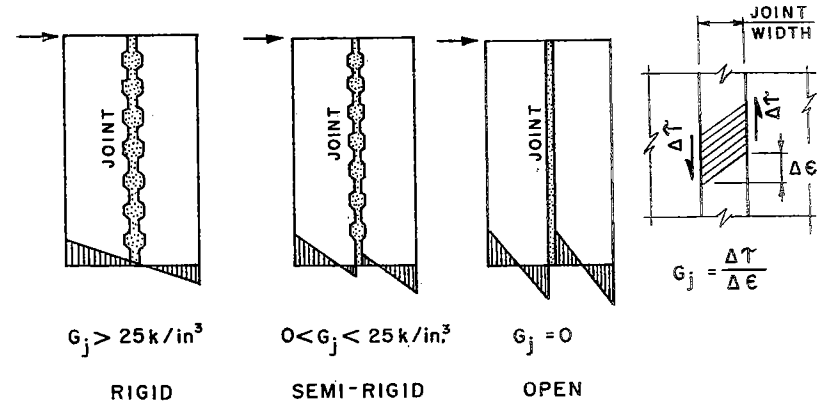
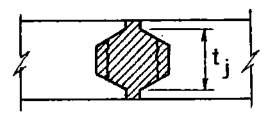


Fig. 12. Classification of vertical joints.

9.3.3 Open—No shear forces can be transferred.

9.3.4—Representative values of the modulus of rigidity[24] for a specific castellated keyed joint are as follows:

Actual Stress Level, τ	Modulus of Rigidity, G_j kg/cm³	Modulus of Rigidity, G_j kips/in.³
$0.9\tau_u$	100	3.6
$0.8\tau_u$	200	7.2
$0.5\tau_u$	1000	36.1
$0.3\tau_u$	5000	180.6



$$\tau = V/(t_j H) \qquad (15)$$

where

V = loaded shear on vertical panel joint
H = height of panel
τ = loaded joint stress
τ_u = ultimate capacity of joint as determined by test

Tables, such as shown above, will indicate the amount of load transfer across the vertical joint.[14]

9.3.5—Joints constructed of cast-in-place concrete may be classified as smooth or castellated. Fig. 13 shows some representative shapes of these joints.

9.4 Vertical joint behavior

Tests indicate the following general behavior characteristics of concreted vertical joints in walls when subjected to shearing forces along the length of joint:

9.4.1—Joints without horizontal reinforcing steel across the joint will generally fail due to bond destruction and subsequent slippage at random values of shear stress.

9.4.2—Joints with horizontal reinforcing steel exhibit a considerably

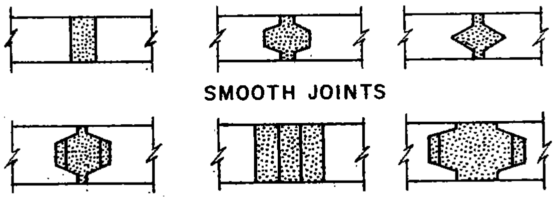


Fig. 13. Typical vertical joints.

greater load capacity, and a smaller dispersion of test results.

9.4.3—The arrangement of horizontal reinforcing steel has no significant effect on joint behavior. Reinforcement may be uniformly spaced along the length of the joint, or concentrated at discrete locations (e.g., within the floor zones).

9.4.4—Vertical reinforcing steel within the joint may contribute to strength, particularly when the horizontal reinforcement within the joint is in the form of interconnected loops, or mechanically connected.

9.5 Shear connections

Design recommendations for joints subjected to shearing forces along the length of the vertical joint are as follows:

9.5.1—Joints should be reinforced unless the adjacent panels are designed to act separately.

9.5.2—As an acceptable alternate to the use of cast-in-place reinforced concrete joints, connections may be accomplished by mechanical methods, i.e., by bolting or welding.

9.5.3—Connections (reinforced concrete or mechanical) may be designed in accordance with the shear-friction theory.

For smooth joints, the shear-friction coefficient, μ, may be taken as 0.7 when determining the reinforcement.

For castellated joints, the amount of reinforcement required will depend on the shape and spacing of the keys (Fig. 14).

The general shear-friction equation can be used:

$$A_s = \frac{V_u}{\phi f_y \mu} \qquad (16)$$

where

A_s = required horizontal reinforcement

V_u = design shear on joint

μ = 1.4

Based on the concepts of shear-friction, F_2 will become zero prior to development of the maximum value of F_1. Therefore all the tension is generated at the key[18,19].

9.5.4—Where ductility is an important design consideration (e.g., in Seismic Zones 2 and 3) vertical joints should have castellations (keys) spaced the full height of the joint. Mechanical connections uniformly spaced may also be used.

9.5.5—The contribution of the floor tie reinforcement that crosses the vertical joint may be included in the determination of total joint resistance.

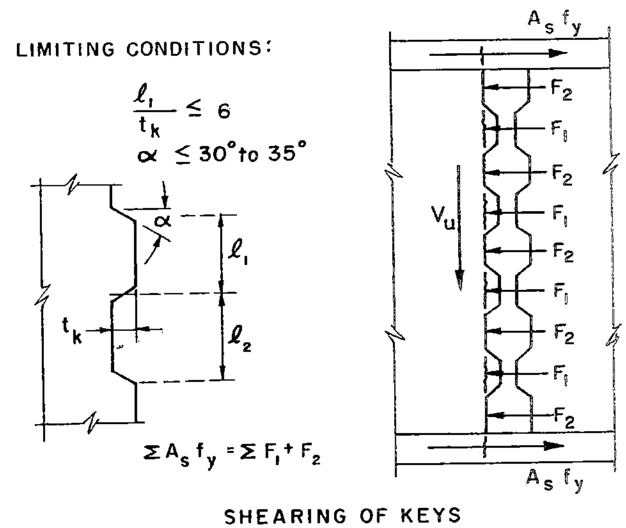


Fig. 14. Shearing of castellations in a vertical joint.

9.5.6—Consideration should be given to shrinkage and to differential shortening between adjacent panels.

9.6 Horizontal joints

Horizontal joints are a particularly critical element in the performance of large panel structures. A correctly detailed joint not only will perform as intended once the structure is complete, but it will also permit:

1. Construction to proceed with a minimum of restriction on erection methods, and a minimum of delay.
2. Visual examination of the connection as it is accomplished.
3. Ease of placement of any reinforcement and concrete within and across the joint.

Continuous chases or conduits should not be permitted within that portion of a joint that is intended to transmit vertical load.

9.7 Types of horizontal joints

General types of horizontal joints (Fig. 15) are as follows:

1. Closed or platform joint
2. Wedge joint[32]
3. Open joint

The three types of bearing conditions are as follows:

1. Concrete on concrete (partial bearing)
2. Concrete on grout (total bearing)
3. Concrete on bearing pads

9.8 Loads

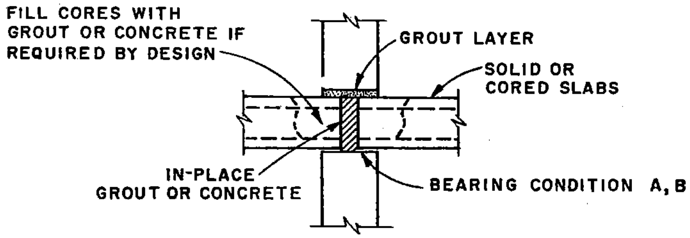
Various loads which may be applied to a horizontal joint are as shown in Fig. 16. The symbols in the sketch are as follows:

P = design loads applied from wall above

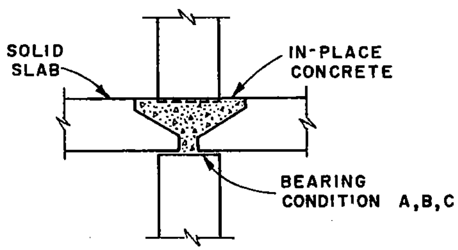
V_w = horizontal shear from wall above

M = end moments from floor loads

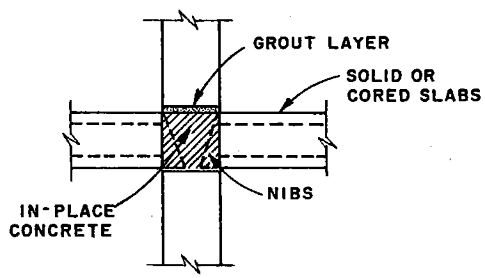
V_f = vertical shear from floor loads



CLOSED JOINT



WEDGE JOINT



OPEN JOINT

Fig. 15. Types of horizontal joints.

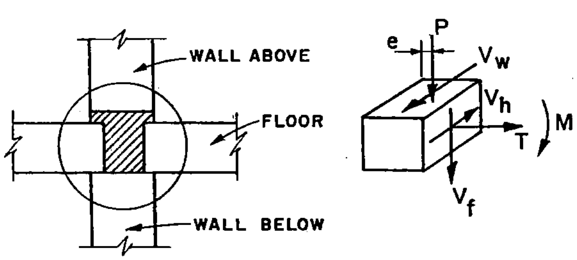


Fig. 16. Loads on horizontal joint.

V_h = horizontal shear in plane of floor due to diaphragm action

T = horizontal thrust in plane of floor (tension or compression) due to diaphragm action and restraint force due to creep, shrinkage, and thermal change

Resistance to the forces in the plane of the floor (M, T, V) is provided in the usual manner in accordance with flexure and shear theories. At any wall where the floor reinforcing steel does not continue across the joint, e.g., at an end wall, care must be taken to insure that these in-plane forces are properly transferred to the wall, or eliminated.

9.9 Vertical load capacity

Three methods for establishing the vertical load carrying capacity of horizontal joints are treated in Appendix 2.

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APPENDIX 1—DESIGN OF TRANSFER GIRDERS

A.1.1 General

Transfer girders supporting loadbearing walls and shear walls may be designed by this method[12] provided that the ratio $H/L > 2$.

The distribution of the wall loads at the level of the supporting girder will not be uniform, but will tend to concentrate near the columns since the lower parts of the wall tend to act in arch action.

Analysis indicates that the load distribution on the beam, and the magnitude of the tensile force in the beam (arch tie) are dependent primarily on the following characteristics of the system:

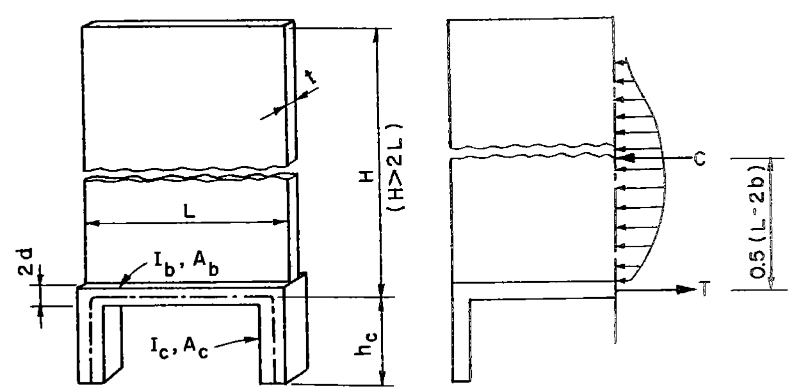
1. Span and flexural stiffness of frame and wall.
2. Effective width of column.
3. Position and size of wall openings.

A typical wall frame is shown in Fig. A1.

A.1.2 Tie Force

The tie force in the girder may be determined from:

$$T = W\left[\frac{1}{4} - \frac{b}{L}\left(1 - \frac{cb}{L}\right)\right] \qquad \text{(A1)}$$



H = wall height
L = wall length
$2d$ = beam depth
I_b = moment of inertia of girder
A_b = cross-sectional area of girder
I_c = moment of inertia of column
A_c = cross-sectional area of column
h_c = height of column

Fig. A1. Arrangement of loadbearing wall on transfer girder.

where

T = tie force

W = total vertical load at bottom of wall

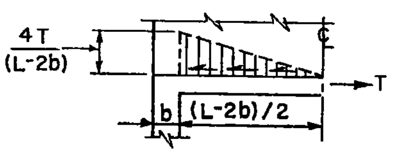
b = modified width of column, taken as 0.75 of the column width for an exterior column and 0.50 of the column width for an exterior column

c = a factor reflecting the stress distribution of the wall loads on the beam. This factor is expressed as the ratio of the (actual vertical stress of wall loads at beam support) to the (average vertical stress of wall loads).

The beam reinforcement determined from this tie force should be properly anchored at the ends of the beam.

A.1.3 Horizontal shear

The horizontal shear between base of wall and girder may be assumed to vary linearly from zero at the midspan of the girder in accordance with the sketch shown below.



Shear reinforcement should be provided between wall and girder, and may be determined in accordance with the shear-friction theory.

A.1.4 Vertical shear

The maximum vertical shear in the girder may be determined from:

$$V = (W/2)[1 - (cb/L)] \qquad \text{(A2)}$$

A.1.5 Bending moments

The bending moments in the girder at midspan and at the support may be determined from:

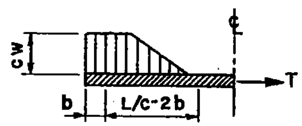
$$M_{sup} = \frac{wL}{12}\left[c\left(\frac{f}{L}\right)^2\left(2 - \frac{f/L}{1-2b/L}\right) + \frac{5c''}{8}\left(1-\frac{2b}{L}\right)^2\right] + \frac{2Td}{3} \qquad \text{(A3)}$$

$$M_{midspan} = \frac{wL}{12}\left[\frac{c(f/L)^3}{1-2b/L} + \frac{3c''}{8}\left(1-\frac{2b}{L}\right)^2\right] - \frac{Td}{3} \qquad \text{(A4)}$$

where w is the average vertical stress of wall loads.

If $\left(\frac{1}{C} - \frac{2b}{L}\right) \leq \left(\frac{1}{2} - \frac{b}{L}\right)$

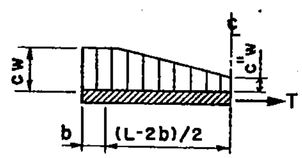
then $c'' = 0$ and $f/L = 1/c - 2b/L$.



If $\left(\frac{1}{c} - \frac{2b}{L}\right) > \left(\frac{1}{2} - \frac{b}{L}\right)$

$$c'' = \frac{2 - c(1 + 2b/L)}{1 - 2b/L}$$

and $f/L = \frac{1}{2} - b/L$.



The value, c, can be taken from Fig. A2, where the abscissa gives the range of the relative stiffness parameter, C_1, on a logarithmic scale. Note that:

$$C_1 = [(L - 2b)^3\, t\, E_w]/(I_b\, E_b)$$

where E_w and E_b are the modulus of elasticity of the wall and transfer girder, respectively.

A.1.6 Other loads

In addition to the above, the girder frame should be designed for any floor loads directly applied, for wind loads, for seismic loads when applicable, and for erection loads applied prior to effective composite action between wall and girder-frame.

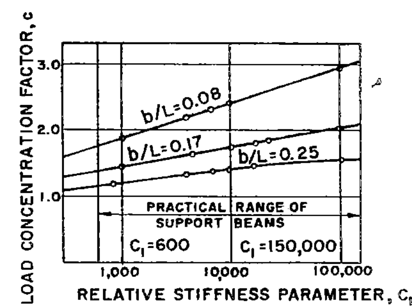


Fig. A2. Variation of load concentration factor, c.

APPENDIX 2—LOAD CAPACITY OF HORIZONTAL JOINTS

A.2.1 General

A comprehensive design approach, including the effect of the variables discussed in Section 9.8, has not as yet been developed for all the different joint types. However, several methods for determining the load capacity of horizontal joints in large panel structures have been proposed. This Appendix presents three design procedures that apply to one common type of joint.

A.2.2 Method 1—Based on strengths of joint components

In this method (see Reference 33), the applied design load on the joint is distributed to the various components of the joint in accordance with the stress-deformation and strength characteristics of the components. Then the sum of the strengths of the components is compared to the strength of the wall as determined by ACI 318-71, Chapter 14. A "confinement factor" is permitted on the cylinder strength of the joint concrete, reflecting the confining nature of the joint.

This method applies particularly to a typical detail (see Fig. A3), where prestressed hollow-core slabs are continuously supported and extend a minimum of 2½ in. over the bearing walls. The space between ends of slabs is filled with concrete, not necessarily of the same strength as the walls. The leveling joint between tops of slabs and bottom of wall above is solidly packed with grout of equal or greater strength than the joint concrete.

Stress transfer is complex at any joint; therefore it is necessary to clarify and isolate forces into manageable units. The floor slabs rest on bearing pads of dissimilar, "softer materials." Thus stresses can be calculated on the basis of stress-deformation characteristics of bearing pads and joint concrete.

Stress-deformation relations are shown in Fig. A4 and the ratio of elastic moduli is given as:

$$\alpha = \left[\frac{P_{n+1} - P_g}{P_g}\right]\left[\frac{t_g\,\Delta_g}{2t_p\,\Delta_p}\right] \qquad \text{(A5)}$$

Average values for α for specific joint materials are given in Table A1. From this table:

$$P_g = \frac{P_{n+1}}{1 + 2\alpha(t_p/t_g)} \qquad \text{(A6)}$$

The corresponding stress in the grout is:

$$f_c\,(\text{grout}) = P_g/(12t_g) \leq \beta f'_c \qquad \text{(A7)}$$

The factor, β, considers the confinement of the joint concrete and

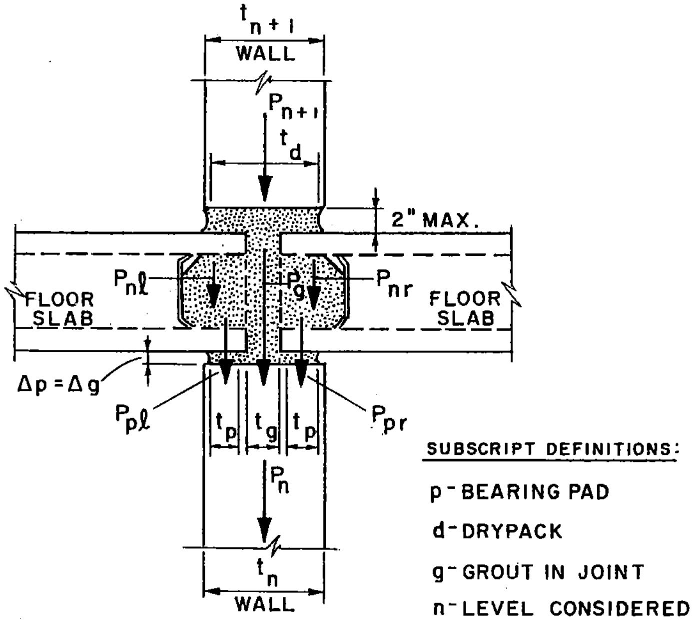
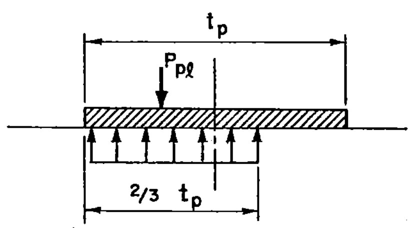


Fig. A3. Typical joint detail.

should not be taken greater than two.

The stress in the bearing pad, assuming the loads and stress distribution as shown, is



$$f_c\,(\text{pad}) = P_{pl}/[(12)(\tfrac{2}{3}t_p)] \leq f_p \quad \text{(A8)}$$

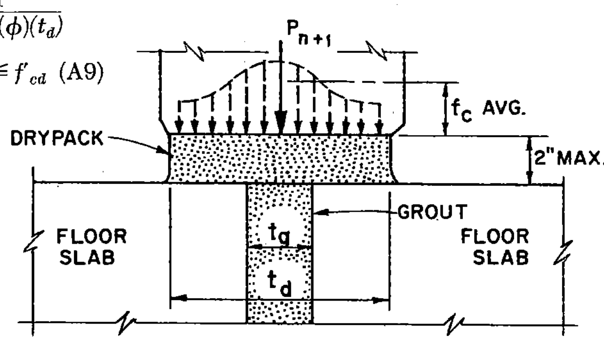
where f_p denotes a maximum recommended stress value.

The stress in the drypack grout under the wall above, assuming the loads and stress distribution as shown, is:

$$f_c\,(\text{drypack}) = \frac{P_{n+1}}{(12)(0.85)(\phi)(t_d)}$$

$$= \frac{P_{n+1}}{7.14(t_d)} \leq f'_{cd} \quad \text{(A9)}$$

where ϕ is 0.70 (from ACI 318-71).



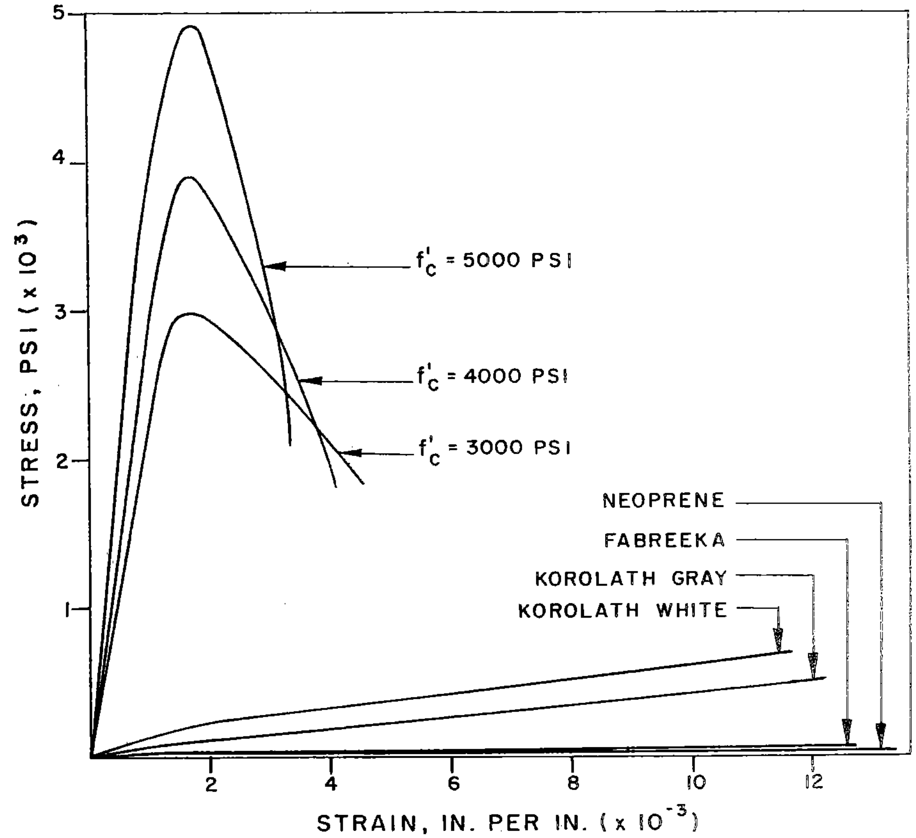


Fig. A4. Strength-deformation characteristic for grout and common bearing pads (see Reference 33).

Table A1. Average values for some common bearing pad materials (see Reference 33).

Type	f'_c (Grout)		
	3000 psi	4000 psi	5000 psi
Korolath White	0.0624	0.0541	0.0484
Korolath Gray	0.0544	0.0472	0.0422
Neoprene 50	0.0042	0.0036	0.0033
Neoprene 70	0.0063	0.0055	0.0049
Fabreeka	0.0029	0.0025	0.0023

Note: Neoprene - values based on shape factor of 7, for $f_p \leq 500$ psi.

Fabreeka - values based on $f_p \leq 500$ psi.

Example: Determine forces and stresses in horizontal joint (see Fig. A5)

Typical apartment house structure
Number of stories: 18
Live load reduction factor considered.
Spacing of bearing walls: 36 ft center to center.
Width of walls: 8 in.
Bearing pads: 2 in. wide, ⅛ in. thick, gray Korolath.
Loads: live load = 40 psf (use 50 percent reduction)

Partitions plus misc.	= 20 psf
Floor slabs	= 60 psf
Dead load	= 80 psf

$$w_u = 1.4 \times 0.08 + 1.7(1 - 0.5)\,0.04 = 0.146 \text{ ksf}$$

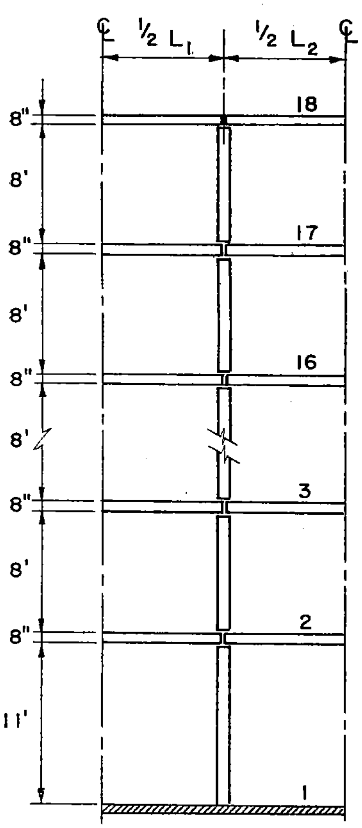


Fig. A5. Typical interior bay.

$$P_{u2} = [18 - 1]\,[(36 \times 0.146) + (1.4 \times 0.15 \times 8 \times 0.66)] = 108.4 \text{ kips per ft}$$

$$P_{u1} = 108.4 + (36 \times 0.146) + (1.4 \times 0.15 \times 11 \times 0.66) = 116 \text{ kips per ft}$$

Check concrete strength required for walls on first and second floors:

First floor
$f'_c = P_{u1}/c_w = 116/30.6 = 3800$ psi
Second floor
$f'_c = P_{u2}/c_w = 108.4/33.6 = 3300$ psi
Therefore, use an $f'_c = 4000$ psi.

Check the grout capacity and strength at Level 2:
Use $f'_c = 3000$ psi; $\alpha = 0.0544$

$$P_g = \frac{108.4}{1 + 2 \times 0.0544(2/2)} = 97.8 \text{ kips per ft}$$

$$f_c\,(\text{grout}) = 97{,}800/(12 \times 2) = 4075 \text{ psi } (< 2 f'_c)$$

Check bearing pad stress:

$$P_{pl} = P_{pr} = (108.4 - 97.8)/2 + (36/2)\,0.146 = 7.93 \text{ kips per ft}$$

$$f_c\,(\text{pad}) = \frac{7930}{0.667 \times 2 \times 12} = 534 \text{ psi}$$

Note that this stress is less than the manufacturer's recommended ultimate stress.

Check drypack stress:

$$f_c(\text{drypack}) = 108.4/(7.14 \times 7) = 2400 \text{ psi}$$

Use $f'_c = 3000$ psi.

It should be noted from the example that, in addition to the floor loads of the level under consideration, only minor forces from the wall above are transferred through the bearing pads. This particular feature reinforces the validity of the assumptions made for the analysis of the structure according to the "hinged joint" pattern and literally recenters loads at each floor level.

Table A2. Factors c_w for calculating bearing capacity of concrete walls (ACI 318-71)

$P_u = (c_w)\,(f'_c)$ ksi

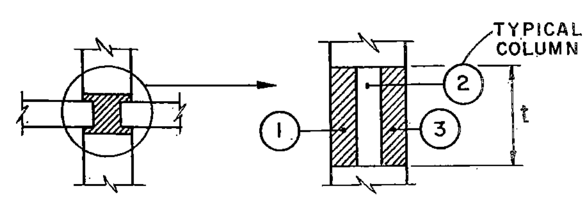
Height h_w, ft	Thickness, t, in. 12	11	10	9	8	7	6	5	4
7	53.7	48.9	44.0	39.3	34.4	29.4	24.3	19.0	13.3
7.5	53.5	48.7	43.7	39.0	34.0	29.0	23.8	18.4	12.6
8	53.2	48.4	43.4	38.6	33.6	28.5	23.3	17.8	11.8
8.5	52.9	48.0	43.0	38.2	33.2	28.0	22.7	17.1	10.9
9	52.6	47.7	42.7	37.8	32.7	27.5	22.1	16.3	10.0
9.5	52.3	47.4	42.3	37.4	32.2	27.0	21.4	15.6	9.1
10	51.9	47.0	41.9	36.9	31.7	26.4	20.8	14.8	
10.5	51.6	46.6	41.5	36.5	31.2	25.8	20.0	13.9	
11	51.2	46.2	41.0	36.0	30.6	25.1	19.3	13.0	
12	50.4	45.4	40.1	34.9	29.4	23.8	17.7	11.1	
13	49.5	44.4	39.0	33.8	28.2	22.3	16.0		
14	48.6	43.4	37.9	32.5	26.7	20.7	14.1		
15	47.6	42.3	36.7	31.2	25.2	18.9			
16	46.5	41.1	35.4	29.7	23.6	17.1			
17	45.4	39.9	34.1	28.2	21.9	15.2			
18	44.2	38.5	32.6	26.6	20.1				
19	42.9	37.1	31.1	24.9	18.2				
20	41.6								

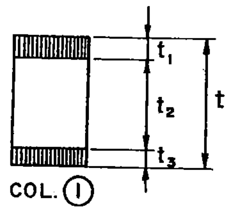
Note: c_w factors below double line apply to precast walls only; all other factors applicable to all concrete walls.

A.2.3 Method 2—Elastic analysis of joint

This method (see Reference 4) assumes that the elastic response of each component of the joint is known. It will permit a determination of the stress distribution, under service load conditions, on the various components of the joint.

The joint is divided into a series of discrete vertical "columns" as shown in the sketch. The amount of load that each "column" supports will be a func-





tion of the stiffness (spring constant) of that particular "column." Each of the layers of the "columns" may have a different thickness, and may be of different materials. The equivalent spring constant, K, of any "column" may be determined by:

$$K = \frac{t}{\Sigma\,(t_i/E_i)} \qquad \text{(A10)}$$

where

t = total height of column

t_i = thickness of an individual layer

E_i = modulus of elasticity of that layer

The vertical stress to which each "column" of the floor joint is subjected can be determined, once the spring constant of each "column" has been established by:

$$f_i = (Pe/S)\,(K_i/K) \qquad \text{(A11)}$$

where

f_i = stress on a particular "column"

P = total vertical service load applied to the joint

e = eccentricity of the force P, measured from the same axis as used to determine S

S = statical moment of a part of the reduced cross section loaded with an equivalent rectangular stress distribution, calculated in reference to one of the edges of the joint.

K_i = spring constant of the particular "column" being investigated

K = spring constant of the reference "column"

Example: Determine distribution of stress in a horizontal joint (see sketch below)

Step 1—Determine the spring constant for each "column"

Column 1:

$$K_1 = \frac{10.5}{\dfrac{1.5}{2000} + \dfrac{8}{4000} + \dfrac{1.0}{2000}} = 3231$$

$K_1/K = 3231/3231 = 1.00$

Column 2:

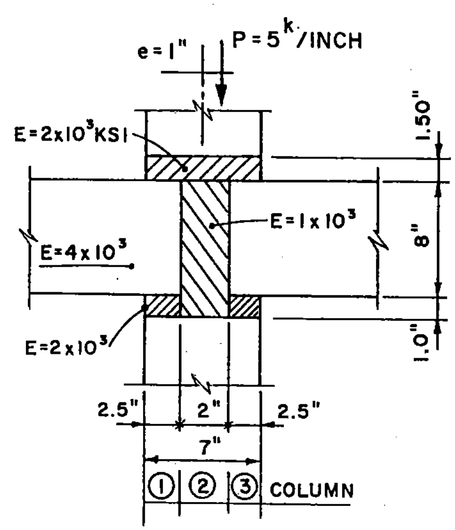
$$K_2 = \frac{10.5}{\dfrac{1.5}{2000} + \dfrac{9.0}{1000}} = 1077$$

$K_2/K = 1077/3231 = 0.333$

Column 3:

$$K_3 = \frac{10.5}{\dfrac{1.5}{2000} + \dfrac{8}{4000} + \dfrac{1.0}{2000}} = 3231$$

$K_3/K = 3231/3231 = 1.00$



Design example

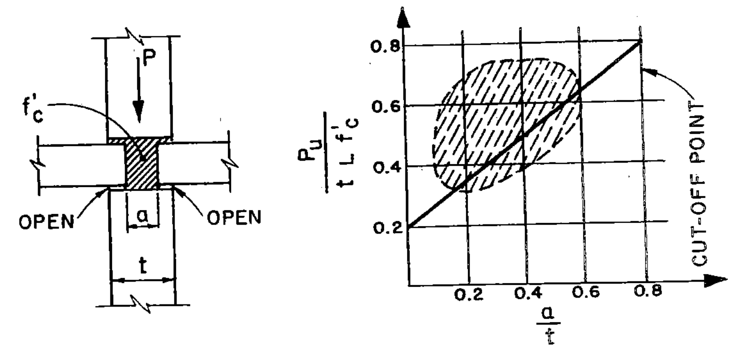
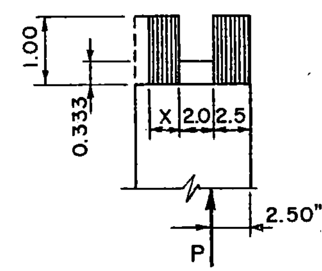


Fig. A6. Test results on a typical horizontal joint.

Step 2—Determine the dimensions of the equivalent stress blocks (see sketch)



$1.00 \times 2.50 = 2.500 \times 1.25 = 3.125$
$0.333 \times 2.00 = 0.666 \times 3.50 = 2.331$
$1.00\,(x) = (x)\,(4.5 + x/2) =$
$4.5x + x^2/2$

The above equations reduce to:
$2.50\,(x + 3.166) = 5.456 +$
$4.5x + x^2/2$ from which $x = 0.986$ in.

Step 3—Determine S

$S = x^2/2 + 4.5x + 5.456$
$S = (0.986)^2/2 + 4.5 \times 0.986 + 5.456$
$= 10.379$

Step 4—Determine stresses in each "column"

Column 1:
$f = [(5 \times 2.5)/10.379]\,[(1/1)]$
$= 1204$ psi

Column 2:
$f = [(5 \times 2.5)/10.379]\,[(0.333/1)]$
$= 400$ psi

Column 3:
$f = [(5 \times 2.5)/10.379]\,[(1/1)]$
$= 1204$ psi

Check

$\Sigma V = (1.204 \times 2.5) + (0.400 \times 2) + (1.204 \times 0.986)$
$= 5.0$ kips (ok)

A.2.4 Method 3—Tests on a typical horizontal joint

Tests by the Danish Structural Research Center, and by others, for the type of joint shown in Fig. A6, have been plotted, and the results generally fall within the shaded zone of the curve as indicated. Provided that the strength of the grout within the joint is about the same as the strength of the concrete in the wall, the following empirical relation is suggested for determining the design strength of the joint:

$$P_u = tLf'_c\,[0.2 + 0.75\,(a/t)] \qquad \text{(A12)}$$

where

a = width of grout bed

f'_c = compressive strength of grout

L = length of wall

P_u = design strength of the joint per unit length

Expressions such as Eq. (A12), based on tests applied to one type of joint, may not be applicable to other joint types or different materials or loading conditions.