DESIGN OF LOAD BEARING WALL PANELS

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Although many high rise buildings throughout North America use precast concrete wall panels, unfortunately, their use is restricted generally to curtain wall panels instead of taking advantage of the panel's structural capabilities.

For a small increase in wall panel cost, these curtain wall panels can be used as both load bearing walls and shear walls. The slight increase in load bearing wall panel cost due to erection and connection requirements can be offset by the elimination of interior frames or interior shear walls.

In recent years, tremendous strides have been made regarding precast concrete structural engineering technology. The development of knowledge concerning connections and wall panel design makes it possible for the engineer to realistically consider the use of precast wall panels as load bearing or structural units.

This third paper in the series on design considerations for a precast prestressed concrete apartment building deals with the load bearing panel design for the 23-story structure.

PRELIMINARY DESIGN CONSIDERATIONS

The design process can be greatly simplified by reviewing the major items that will control the final solutions. Once these particulars are studied relative to the overall solutions, the other secondary phases of the design become apparent.

Order of Solution

The initial order of solution can be divided into the following categories:

1. Determine or estimate the design gravity loads to the wall panels.

2. Determine or estimate the design lateral loads to the wall panels.

3. Estimate initially which panels receive the greatest loading due to the combined or singular action of gravity and lateral loads.

4. Estimate, based on the above, whether gravity or lateral loads will control the panel dimensions and shapes.

5. The preliminary size and shape of the wall panel will be strongly affected

by the general influence of the connections and their locations, panel mullion to mullion connections, and general erection requirements.

6. The preliminary size and shape of the wall panel should be reviewed relative to all production requirements for stripping, general handling, storage, shipping or transportation, and architectural finish. In short, the panel size and shape should be practical.

7. Preliminary development of connection details must consider questions such as:

(a) If the details are practical, do they lend themselves to standardization?

(b) Can the details be plant produced within required tolerances assuring proper quality and strength?

(c) Can the details be field constructed within required tolerances assuring proper quality and strength?

(d) Once details are selected initially can they be designed by an accepted rational method and will volume change deformations (creep, shrinkage, and temperature) influence the detail or its design.

8. Review the creep, shrinkage, and temperature behavior of the building as a whole regarding how the structural behavior of the panels is influenced, and how non-structural items such as partitions, glass, and caulking are affected.

Following the above initial review and study, a realistic preliminary design can be made. The preliminary design will often serve as the final design considering the usual compromises and revisions which are normal to any building design.

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DESIGN PROCEDURE

The purpose of this presentation is to illustrate the design approach as discussed previously in "Preliminary Design Considerations." Rather than presenting designs for each wall panel component, a typical 10-ft high x 12-ft wide panel located between the first and second floors will be reviewed in depth. The designs for the other wall panels of the building would be completed in a similar manner.

The text portion of this paper will discuss the general concepts of the design presented by the calculations. Calculations in detail have been prepared for the typical panel as well as for the probable controlling design conditions.

Tabel 1 summarizes the major design section headings together with their corresponding calculation sheet numbers (see next page).

TABLE 1. CALCULATION SHEET INDEX

Design Section	Calculation Sheet
Loads and design criteria	1/2 A to 2/2 A
Gravity loads to panels	1/3 B to 3/3 B
Lateral wind loads to panels	1/3 B1 to 3/3 B1
Determination of mullion size	1/5 C to 5/5 C
Preliminary W1 panel shape	1/2 C1 to 2/2 C1
Design of W1 panel floor support connections	1/4 C2 to 4/4 C2
Connections design for lateral loads	1/11 C3 to 11/11 C3
Typical panel connections	1/5 C4 to 5/5 C4
Determine interaction at ultimate—center mullion	1/7 D to 7/7 D
Mullion column slenderness design	1/3 D1 to 3/3 D1
Mullion creep review	1/5 F to 5/5 F
Stripping design of W1 panel	1/10 S to 10/10 S

GRAVITY LOADS

The gravity loads imposed to the load bearing wall panels are presented in the calculations on Sheets A. Live loads are those required by the Uniform Building Code. Roof loadings are conservatively taken as equal to the floor loading for this preliminary design phase.

Dead and Live Loads

Aside from the wall panel dead load, the floor dead loads are those which are typical to any building design. An excellent rule of thumb is to estimate that window type architectural panels, made of normal weight concrete, weigh approximately 85 psf. This would reduce to about 65 psf if the panels are made of lightweight concrete.

The design live loads can and should be reduced in accordance with the Uniform Building Code. The lower story wall panels receive the greatest live load reduction, and for typical panels between the first and second floors, a live load reduction of 60 percent is allowed.

Floor Load Distribution

The wall panel support of building floors alternates for odd and even floors. The floor framing directions and panels receiving the loads are presented in the calculations on Sheets B.

Three basic types of gravity loadings are imposed to the wall panels. Some panels at a given floor level are loaded by that floor uniformly over their width, others receive no floor loadings, and panels such as Panels W1A and W1B in the calculations on Sheets B are also loaded by a concentrated beam reaction.

Generally, it is desirable to minimize the weight of the superstructure as it could influence foundations or other components of a classical framed structure. As is discussed later, the uniform loads to the building perimeter are one of the important advantages for high rise load bearing panel structures where the bearing walls also serve as shear walls.

Controlling Panel Loads

Architectural requirements of the building exterior normally require the building elevations to have wall panels of all the same appearance. Therefore, it initially becomes apparent that the wall panels receiving the greatest gravity loads should be determined.

A study of the typical building floor plan in the calculations on Sheets B indicates that either Panels W1A or W1B receive the greatest gravity loads due to the beam reactions. The manner by which Panels W1A or W1B resist the gravity loads is somewhat indeterminate.

The loads to the typical panel shown on Sheet 3/3 B can be resisted by the mullions only, or distributed across the panel width by the panel acting as a deep Vierendeel truss action. Most probably, the actual panel load carrying distribution is somewhere in between.

Since the panel shape and connections dictate the practicality of panel load distribution, it is necessary to consider the center panel mullion as carrying most of the load when selecting connections and mullion cross sections. The side mullions will receive loadings greater than those computed in the calculations on Sheets B due to the truss behavior. Selecting side mullions of the same dimensions, except for width, solve the problem of how the panel resists the loads, assuming side mullion reinforcement is similar.

The preliminary calculations indicate that Panel W1A receives the greatest gravity load for a panel between the first and second floor. In terms of working load, assuming the load carried directly by the mullions, the center mullion supports 663 kips and the side mullions about 85 kips.

Final design calculations should review in detail the load distribution via Vierendeel truss behavior using a computer program such as "STRESS."

LATERAL LOADS

The lateral load analysis considered both seismic and wind forces. The loads used for the analysis of forces to the wall panels are based on previous Paper 2, "Analysis of Lateral Load Resisting Elements" by John V. Christiansen.

The controlling lateral load is wind in a direction parallel to the 72-ft building dimension. The calculations on Sheets B1 and C3 pertain to the lateral load behavior of the wall panel shear walls. No analysis is presented for wind pressure and suction loads to individual panels. Comparatively, this is of secondary importance although it would be checked in the final design.

Lateral Load Behavior

The lateral load behavior of the building indicates that the exterior wall panels about the perimeter do not behave as a "structural tube" cantilevered from the foundations. The 72-ft side wall plus a portion of the 96-ft walls just around the corner, acting somewhat as a channel section, resist the lateral loads. The channel type behavior is summarized on Sheet 2/11 C3 of the calculations, and shown shaded on Sheet 3/3 B1. Channel type behavior is caused by shear lag of the 96-ft walls, and the effective channel flange width varies with the height of the building depending on axial shear deformations of the 96-ft wall.

Channel type behavior of the building creates large loads to the corners. These loads result in 668 kips (working) loads near the corners between the first and second floors.

As summarized in the calculations on Sheets B1, a major concern is whether or not the 668-kip force at the corners results in net tension loads to the wall panel mullions. Dead load of the floor framing, which loads the entire building perimeter, plus the weight of the panels, conservatively shows that no tension loading will occur. In making the dead load calculations for resistance of the 668-kip force, only a 18-ft side wall length is assumed. The tying together of the building, and type of connections used, most probably insures that the entire 72-ft wall could be considered for resistance of wind tension forces at the building corners.

DETERMINATION OF PANEL MULLION SIZE

The analysis for determining the required panel mullion sizes are given in the calculations on Sheets C. Panel mullion size will be controlled by dead and live gravity loads or dead gravity loads plus axial loads induced by wind. At ultimate, between the first and second floors, the gravity load at 954 kips to the center mullion of Panel W1A controls. Wind, plus gravity, at the corner mullions results in an ultimate load of 640 kips.

The determination of panel mullion sizes can be strongly influenced by the manner in which the panels are joined or connected. The most practical connection type is one that allows for realistic tolerance, and assures transfer of load between panels at horizontal joints. A grouted type connection satisfies this criterion. It also is desirable that a realistic minimum amount of field grouting be done. If grouting is done only in the mullion area (see Sheet 3/5 C), it complies with the assumptions concerning loads carried by the mullions. Likewise, by selecting a given force path for load transfer, a safe and realistic design can be achieved.

Mullion Grout Area

A check of the initially selected panel dimensions for the mullions indicates that a mullion size greater than that on Sheet 2/3 C of the calculations is required. The final determination of mullion size is based on design methods presented in the *PCI Design Handbook.* A trial and error type solution indicates that a 6000-psi non-shrink grout is satisfactory for the mullion dimensions presented on Sheet 3/5 C of the calculations.

The mullion grout area design is based on also using reinforcement within the joint. Even if reinforcement is not required by design, it should be used to insure that the building is tied together. The details of the mullion-to-mullion connections required between the first and second floors is presented in Sections A, C and D of Sheets C4. Since connection standardization, and tying the building together are so important, these details would be used for all panels at all story levels.

The details for mullion grout reinforcement or tying the panels together must be carefully selected considering the construction requirements. The method selected is based on the approach given by the *PCI Design Handbook.* Suggested minimum dimensions are given with the grout details. Also, the designer should carefully review the panel production setup for maintaining tolerances and alignment of inserts that are part of the detail. Preferably, the precast panel end forms should employ steel templates insuring coil rod placement exactly (within $\pm 1/8$ in.) to minimize field construction problems. Once the tolerances for the insert or coil rods are selected (considering all three dimensions), it will determine the diameter of the flexible grout conduit.

DETERMINATION OF PANEL DIMENSIONS

The final general panel dimensions are selected considering production requirements, shipping or transporting limitation on size, weight relative to erection equipment, temporary connections required for erection, and final connections. Equally important when considering the shape is how the final panel dimension may or may not influence connection tolerances. A preliminary check of the panels structural capabilities should also be made when selecting the shape and dimensions.

It is necessary to have completed the connection design for temporary erection or final connections when relating panel shape to connections. Actually, the connection details, reflecting their requirements, should be selected during determination of panel shape, and then checked at a later time for structural capabilities.

The calculations on Sheets C1 show the panel shape and its relation to the building based on the above review. The top of the panel is placed 4 in. above the finished floor. This arrangement allows for easy down hand completion of temporary floor tee to panel connections with tolerance. Another advantage of plates cast into the panel for temporary connections is that their capacities are not controlled by edge boundary conditions at the panel top (Sections F1 and F2 of the calculations on Sheets C4).

Further, the 4-in. projection of the panel above the finished floor allows for elimination of boundary or edge conditions regarding shear cones for floor diaphragm connections (Section F of the calculations on Sheets C4). Another advantage of the 4-in. projection is the ease by which grouted vertical panel joint shear connections can be made and inspected (Section E of the calculations on Sheets C4).

Overall Panel Dimensions

A general rule is to make any precast panel as large as possible. For this 23-story building, the largest panel is that shown by the calculations on Sheets C1. This panel size appears to be the maximum based on the erection equipment and methods available for the project. The W1 panel size also lends itself to minimum handling as it will probably be plant stored and shipped in its erected position.

Another consideration for panel size is to review whether or not vertical joints between panels should be staggered floor to floor. That is, should the center panel mullion of a panel be centered over the joint between panels one floor below?

While horizontally staggered panels may be desired, they create many tolerance problems. High rise buildings are dependent on panel joints widths for accommodating the building's plan dimensions. Staggered panels can seriously reduce the available tolerances of coil rod dowels into conduits for the mullion connections.

The most serious drawback to using horizontally staggered panels is location of the floor beams. If staggered panels are used, this means that on every other floor the beam must bear on two different panels. The beam connection problems created by staggered panels should be avoided if at all possible.

PANEL GRAVITY CONNECTIONS

Aside from the mullion-to-mullion connection already discussed, two typical panel gravity connections must be selected. One connection is for the floor tees and the other for the floor beams.

Floor Tees

The design of the floor tee haunch connection is given by the calculations on Sheets C2. The haunch design is based on the *PCI Design Handbook*. This connection design brings out several practical considerations. Probably the most important is the practical factors involving the selection of the haunch depth. The haunch must not only accommodate the floor tees, but it must also have dimensions that can satisfy the beam connection requirements. A study of these aspects, combined with the panel shape, results in a 10-in. haunch depth. Another design aspect is that the load to the panel haunch is concentrated, not uniform.

The selection of a neoprene bearing pad between the bearing interface (see Section F on Sheets C4) allows for using $T_u/V_u = 0.2$ for design. Review of the bearing pad selection indicates that confinement reinforcement is not necessary, and that the haunch is satisfactory for plain concrete bearing. Further, a bearing pad stress of 281 psi combined with the shape factor for a ¼-in. neoprene pad shows that a compressive deformation, including creep of the pad, will not create problems with connections, or cause topping separation from the tee where the topping contacts the back of the panel.

Floor Beams

During determination of the panel shape, a 4-in. projection of the panel top above the finished floor requires a check of the beam end having a dapped connection. The use of a dapped beam connection greatly simplifies the beam-to-panel connection as a 10-in. panel haunch can accommodate the beam reaction details.

The beam-to-panel connection is given in the calculations on Sheets C2 and details are presented on Sheet 4/4 C2 of these calculations. Note that a neoprene bearing pad is selected to even out the bearing as well as control the T_u forces to the dapped beam end. The connection employs two C7 x 12.25 (A36) steel channels. As required by connection design, both the concrete bearing capacity and shear capacity of the channels is checked.

An important part of the detail is the connection at the top of the beam to the panel. This connection is used for erection and to positively tie the beam to the panel. The top beam connection behaves as a tie and does not attract negative bending due to the presence of the neoprene bearing pad eliminating the force couple. When designing the plate connecting the beam and the panel, it is required that this plate be the weak link. Advantage can be taken of the yield characteristics of the plate if welds are made only as shown. The plate dimensions then determine the maximum T_u force for design rather than some complicated analysis for volume changes. Also, once this connecting plate is selected, anchorage plates in the panel and beam can be designed for a greater ultimate strength. This type of design approach illustrates a main advantage of precast concrete connections. Namely, precast concrete connections details can assure assumptions and allow the design to control the connection behavior.

LATERAL LOAD CONNECTIONS

It is necessary to first determine the lateral load shear distribution to the 72-ft shear walls prior to starting any connection design. For the controlling lateral load case, wind parallel to the 72-ft building dimension, the behavior of the shear wall appears to differ due to panel joints from that for a monolithic cast-in-place shear wall with openings.

The main difference between the precast panel shear wall and a cast-in-place shear wall is the presence of the horizontal and vertical joints caused by the individual panels. A question can be raised as to whether the precast panel shear wall conforms to the assumptions and method of analysis used to determine lateral load behavior of the building.

The only realistic design approach

for the precast panel shear walls is one which insures that individual wall panels act compositely with each other. An ultimate design approach, based on shear friction, considering the horizontal and vertical panel joints as failure planes provides a means to insure ultimate composite action. Since it is important that the precast shear wall behave as a monolithic wall in the elastic range, the connections selected must also satisfy the elastic shear distribution. A means for comparing the difference in shear distribution for an ultimate and an elastic composite approach is to review the behavior of a composite beam.

The design of a composite beam by an ultimate shear friction approach insures its strength, but does not insure its deformation behavior. An ultimate approach assumes that the shear distribution at the composite interface is uniform, and thus allows the connections (composite stirrups) to be spaced uniformly and independent of the shear diagram. Elastic composite design considers the shear diagram and spaces composite stirrups accordingly. If the total number and capacity of ultimate composite stirrups are first selected, and then located approximately according to the beam elastic shear diagram, the composite beam will behave monolithically for practical design purposes.

The use of an ultimate composite design method places several requirements on the design. It requires that the connections be stiff and at the same time not have their capacity influenced or reduced by the volume changes inducing T_u forces into the connections. Another requirement is that the connections should not be eccentric to the plane of the shear wall. For these reasons, it is most undesirable to use more structurally flexible weld plate solutions for achieving composite action between precast wall panels. Grouted type connections should be used as they satisfy both design requirements and methods of rational design.

For the reasons just cited, concerning weld plates, a preliminary review of the panel arrangement at the building corners suggests strongly that a revision is in order. The corner arrangement in the proposed problem requires the use of weld plates. Instead, concrete dimensions should be adjusted to allow for the use of a grouted vertical panel joint.

Distribution of Shears

The determination of the shear distribution for the 72-ft panel shear wall between the first and second floors is presented in the calculations on Sheets 1/11 C3 to 4/11 C3. Between the first and second floor, a total ultimate horizontal wind shear of 505 kips is applied.

This distribution of 505 kips wind shear will be modified by the influence of the "flange" compression and tension forces in the 96-ft wall as shown on Sheet 2/11 C3. The assumed shear distribution is that which complies with the ultimate composite approach.

The elastic analysis for lateral loads indicates that the maximum "flange" shear at the building corner occurs near the 8th story level, and is 30.1 kips at ultimate. As discussed in the following, and in the calculations, the connection for composite action must also have sufficient strength to develop this peak type force. Most probably, all connections will be standard, and their capacity based on a 73.3-kip force for a B panel shown on Sheet 3/11 C3 of the calculations. A check of shear distribution at the 8th floor should be made to verify that the typical connection is satisfactory.

Horizontal Joint Shear

The design of the joint connections to resist horizontal shear can be achieved by one of two approaches.

One method is to consider dead load friction, and the other is to use shear friction where the 1-in. coil rods serve as the A_{vf} reinforcement, neglecting the dead load force. Actually, if the A_{vf} shear friction approach is used, the dead load at the mullion would reduce the required A_{vf} area.

The horizontal shear connection analysis is presented on Sheets 6/11 C3 and 7/11 C3. The connection design approach shown by the calculations is conservative, and the horizontal joint shear capacity of the wall panels greatly exceeds that imposed on it.

Floor Diaphragm Shear

The design method for the connections of the floor diaphragm to the shear wall is covered on Sheets 7/11 C3 to 8/11 C3. The case illustrated in the calculations is for the first floor where the wind pressure is 20 psf. Upper floor wind pressures of 40 psf as required by the code would control.

As discussed in the calculations, the shear stresses between the topping and the wall panels at ultimate are very small; not greater than 13 psi at ultimate for the upper stories. A shear friction approach can be used to design this connection, or the temporary erection connections can be employed to resist this shear.

Vertical Joint Shear

The connection design for vertical panel joint shear is presented on Sheets 8/11 C3 to 11/11 C3. A vertical joint shear of 73.3 kips at ultimate is used for this connection design in accordance with the previous discussion.

To insure that the vertical joint shear force of 73.3 kips can be transferred by the panel, a preliminary check of the panel's concrete shear capacity is made. The approximate analysis shown indicates that the 6-in. concrete thickness should be satisfactory. However, on completion of this preliminary design, a final check of the panel's concrete shear capacity would be made considering the panel to be a Vierendeel truss subject to the loads applied. If additional shear capacity is required, it can be obtained by increasing the 6-in. thickness or providing more mesh (shear reinforcement).

Design of a grouted vertical panel joint shear connection by methods set out in the *PCI Design Handbook* will require the use of A_{vf} and A_{sh} reinforcement. The definition or locations of the A_{vf} and A_{sh} reinforcement is shown by Section F of the calculations on Sheets C4.

The calculation of the grouted vertical shear joint area is based on v_u not exceeding 800 psi for a $\mu = 1.4$. This joint must use large interlocking keys if a μ of 1.4 is to be used. The detail selected following the design is given by Section E of the calculations on Sheets C4. A review of this Section E indicates the importance of initially selecting panel shape and dimensions prior to design.

Projection of the panel top above the finished floor allows for easy placement of grout connection as the erected floor serves as a work platform. Also, since the A_{vf} reinforcement is placed in the floor topping, the 4-in. top projection allows the grouted connection to be placed adjacent to the A_{vf} reinforcement.

A secondary consideration, yet still important, is the stiffness of the panel in the vicinity of the grouted connection. To prevent any torsional distortions of the 6-in. flat concrete adjacent to the panel side mullion, good design practice would require all panels to have a 10-in. haunch rather than just those supporting floor tees.

The A_{sh} reinforcement is required to insure that a failure plane between the topping and the back of the panel will not reduce the strength of the A_{vf} No. 5 reinforcing bars. The A_{sh} design is based on a $\mu = 1.0$. A $\mu = 1.0$ can be assured if roughened keyways as shown in Section E of the calculations on Sheets C4 are used.

Summary

The design of the precast panel shear wall is one of the main topics of this presentation. The analysis presented attempts to consider all of the structural design considerations regarding forces, and practicality.

An overall look at the shear wall behavior reveals that it is probably much stiffer than assumed in the lateral load analysis. The details used not only tie the walls together, but also make the walls composite with the floors. Thus, regarding stiffness, the shear wall should be considered to have flanges at each floor level when realistically determining the deformation characteristics of the building as a whole. Perhaps a second lateral load analysis would be in order to evaluate what influence the composite floors have on the shear wall force distribution.

A practical consideration regarding the details used is the effect the ½-in. coil rods have relative to topping separation. Grout keys in the panel at the panel-to-topping interface prevent topping separation. Topping separation is further prevented from occurring due to the erection connections shown in Sections F1 and F2 of the calculations on Sheets C4. The selection of these erection connections also must consider the prevention of topping separation by their number and location.

Important to the structural design of the building is the time of topping placement. It would appear that the topping should be placed at least two stories below the level of erection during the construction phase. The method of erection, regarding tower crane stability, can require the topping to be placed following the erection of a floor level.

The subject of shear wall joint connections should be reviewed in a general sense. These connections can be achieved by bolting, welding or grouting. Of the three types, only the grouted joint connection really satisfies all of the design criteria. Bolted connections require tolerance to achieve bolting, and this reduces the effective stiffness of the shear wall. Also, bolted connections would require location of inserts close to panel edges which is not desirable. Welded shear connections, while initially attractive, have questionable capacity since they attract building volume changes restraint forces. These volume changes or T_u forces can, over a period of time, reduce the connection's ultimate capacity. Moreover, the anchorage of plates into the precast panels suffers the same problems as bolted connections in that anchors are located near the free edges of the panel requiring the use of partial shear cones. Lastly, welded connections result in T_u shear loading towards the free edge of the panel (see Sheet 10/11 C3 of the calculations). The determination of T_u forces is at best, only a rough approximation.

The use of grouted vertical shear connections provides for volume movement, and if A_{vf} reinforcement yields, it does not impair the connection capacity. Grouted vertical joint connections also provide for excellent field tolerance. The most important aspect of the grouted vertical joint connections is the final field accomplishment of the detail. The final details selected such as pumping grout, vent tubes, and grout tubes should be made only after extensive discussions with contractors that will be making these connections in the field.

MULLION COLUMN BEHAVIOR

A structural evaluation of column behavior for the panel mullions is obviously required as the wall panels are load bearing. Three items are important when considering or designing the panel mullions to behave as columns. It is necessary to determine the short column load-moment interaction diagram, influence of column slenderness, and arrangement of column reinforcement such that it can accommodate placement of all connection hardware (i.e., embedded steel channels for beam-to-panel connection).

Interaction Load and Moments

The ultimate interaction analysis for the center mullion column is presented by the calculations on Sheets D. The straight line interaction plots for this center panel mullion is presented on Sheet 7/7 D. An analysis is made to illustrate the long-hand procedure for a trial and error method when the column cross section is nonrectangular. Realistically, this type of analysis should be done by a computer.

As illustrated in the calculations on Sheets D a column cross section similar to the grout area is used. A preliminary evaluation indicated that six No. 8 reinforcing bars might be satisfactory. A larger column cross section could have been considered initially in accordance with Section 4.3.5 of the *PCI Design Handbook.*

The approach presented in the calculations on Sheets D is based on a strain-compatability method. To develop a simple interaction diagram (straight line), it is necessary to solve for P_u only, M_u only, and P_u and M_u at the so called "balance point." A key item in making the column analysis is the determination of the centroid of the compression area loaded by the rectangular stress block, rather than using "$a/2$" as for rectangular cross sections.

Slenderness

The analysis for column slenderness is given in the calculations on Sheets D1. The analysis presented is based on ACI 318-71, and Section 4.3.5 and Fig. 5.4.1 of the *PCI Design Handbook.*

A column cross section equal to the mullion grout area results in a kl_u/r ratio greater than 22 considering the mullions to have $k = 1$ for a braced frame condition. The determination of P_c allows for calculation of δ, the moment magnifier. Based on the ultimate loads applied, and correcting the end moment by δ, then the short column loads are $P_u = 954$ kips and $M_u = 208$ ft-kips.

Plotting the above P_u and M_u values on the interaction diagram in the calculations on Sheets D indicates that the column, as evaluated, is not satisfactory. However, if the column cross section had been assumed to be larger as allowed, it is probably safe to estimate that the design would have proved satisfactory. Also, the fact that the actual interaction diagram is curved, not straight, should have been considered. Finally, if after considering these just mentioned refinements, the column still is unsatisfactory, it may be necessary to adjust the concrete strength f'_c for the lower stories and/or increase the amount of reinforcement in the lower story panels.

PANEL VOLUME CHANGES

When designing any concrete structure, whether precast or cast-in-place, a review of the structure's behavior as influenced by concrete volume changes is required. Building movements due to temperature, creep, and shrinkage should be reviewed relative to partitions, windows, shear to columns, axial loads to beams, and T_u forces to connections.

The calculations on Sheets F present an analysis of the creep behavior of Panel WIA (see Sheet 1/3 B) subject to a large load at the center mullions, and much smaller loads to the side mullions. The greatest creep deformation occurs at the upper stories due to creep accumulation. The analysis indicates that a ⅜ in. relative movement could occur between the center and side mullions for Panel WIA at the 23rd story. This analysis is consistent with the initial assumption that all panel loads are carried by the mullions directly, and not influenced by truss action.

As discussed, and illustrated in the calculations on Sheets F, the assumption that mullions carry load directly is not correct. The actual relative creep deformation between center and side mullions will be considerably less, if at all. Loads applied to center mullions will tend to be transferred to the side mullions by Vierendeel truss behavior of the panels. That portion of the center mullion load not transferred by truss action will in time, be transferred to the side mullions by the axial creep behavior of the panel under axial load.

Nevertheless, it is proper design procedure to recognize that some differential creep behavior may occur in the panel. Additional nominal reinforcement should be added to the panel as illustrated in the calculations on Sheet 5/5F. Likewise, good design practice would indicate that the side mullion column reinforcement should be similar to the center mullion reinforcement when considering the two side mullions together.

No analysis is made for the effects of temperature and shrinkage behavior of the wall panels. This does not imply that a review is not necessary. An item that temperature movement may influence is the panel connections at the building corners where the two perpendicular elevations under temperature movement could create additional vertical joint shears. Volume change deformations of the building as a whole probably will not affect the glass or partitions. Again, relative to the partitions, a prudent design may suggest the use of partition details that allow for accumulated movements in the upper stories.

PANEL STRIPPING DESIGN

Normally, the engineer of record is not concerned with the production and erection, or general handling design of a precast panel. Rather, he designs for in-place conditions, and leaves the handling design to the wall panel producer.

During the preliminary design phase, such as discussed in this presentation, it is most desirable to know whether or not the shape of panel, which satisfies in-place loadings, is satisfactory from a practical production standpoint. In short, is the panel a "boat in the basement" or an extremely uneconomical shape to produce because of excessive or very special handling requirements?

A preliminary review of panel stripping requirements is presented in the calculations on Sheets S. The analysis presented deals only with the initial stripping. In order to make the analysis it is necessary to determine the section properties and weights of each cross section as illustrated. The key factor is the calculation of bending stresses to insure that the panel does not crack when being stripped. Reinforcement is selected to further control, or eliminate the possibility of cracking, which can be the cause for rejection of a panel. The procedure illustrated in the calculations is done by a computer program. The program is based on the requirements set out in Section 4.3.6 of the *PCI Design Handbook,* and references cited in the calculations.

Also discussed in the calculations on Sheets S is a total review of handling inserts for the stripping condition. It is important to determine whether the steel of the insert or strength of the concrete shear cone determines the ultimate insert capacity.

While the calculations presented discuss only the stripping case, it would be normal practice of a quality producer to also check all other phases of handling. Usually, a similar analysis is made for all in-plant turning operations, shipping methods, and field erection procedure.

Completion of handling design allows for determination of panel reinforcement. Again, when selecting reinforcement for a panel, once the controlling structural or handling reinforcement is known, it is important to select a practical cage configuration. Selection of cage details requires that it has three-dimensional structural stability, that it can be lifted or moved, realistic tolerances, that it provides for placement of all embedded items, and careful consideration to how reinforcing bars are bent.

SUMMARY

The objective of this paper has been to present one designer's view on the procedures, concepts, and tools available for the design of a load bearing precast panel building. It is not possible to set out a complete design for all components in this discussion, and thus general typical and critical conditions have been examined in detail.

The text portion of this presentation has been written more or less separate of the calculations. Many additional rules of thumb and other reasons for making decisions are covered in the calculation sheets which follow.

Detailed numerical calculations supplementing the preceding design considerations are presented on the following 35 pages (Sheets 1/2 A through 5/5 C4). These calculation sheets comprise just over one-half of the computations. The remaining calculation sheets will appear in the next JOURNAL. For a summary of the topics see Table 1, page 16. Discussion of this paper should be deferred until the entire calculations have been published.

LOADS AND DESIGN CRITERIA — SHEET 1/2 A

Design Requirements

1. Uniform Building Code
2. PCI Design Handbook

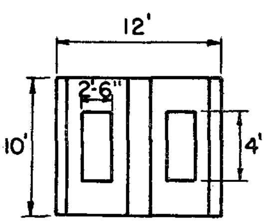
DESIGN SERVICE LOAD

Floor Dead Loads to Panels

(a)	Partitions	= 10 psf
(b)	2½ in. normal weight topping	= 30 psf
(c)	Suspended ceiling	= 3 psf
(d)	Mechanical equip.	= 4 psf
(e)	Beam (30-ft span)	= 20 psf
(f)	Beam (24-ft span)	= 13 psf
(g)	6-ft double tee	= 30 psf
(h)	Panel weight (see calculations below)	= 65 psf

$$\text{DL Floor} = \overset{(a)}{10} + \overset{(b)}{30} + \overset{(c)}{3} + \overset{(d)}{4} + \overset{(g)}{30} = 77 \text{ psf}$$

Panel Dead Load (based on initial assumed size)



Weight of concrete = 110 pcf

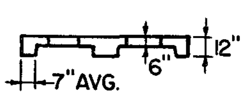
$$\text{Mullions} = \frac{7(6)4(10)\ 110}{144} = 1283 \text{ lb}$$

$$\text{Flat} = (12)(10)(6/12)(110) = 6600 \text{ lb}$$

$$\text{Windows} = (6/12)(4)(2.5)(2)(110) = -1100 \text{ lb}$$

$$6783 \text{ lb}$$

Therefore, the total panel dead load is:



6783/(12x10) = 56 psf, based on experience use 65 psf. For normal weight concrete, load bearing panels weigh approximately 85 psf relative to preliminary design.

Floor Live Loads

Specified load = 40 psf. Use live load reduction to panels.

$$R = \frac{(D+L)100}{4.33L} = \frac{(77+40)\ 100}{4.33(40)} = 67.6\% \ (60\% \text{ maximum}).$$

Find area required for 60%: 60/0.08 = 750 sq ft

The W1 panel supports an area greater than 750 sq ft (use 60%).

LOADS AND DESIGN CRITERIA (CONT.) SHEET 2/2 A

Superimposed live load per floor to W1 Panel = (1-0.6)40

= 16 psf

Roof Dead & Live Loads

For preliminary design consider roof to load W1 panel same as floor. This is conservative since roof has no partitions or topping.

SUMMARY OF DESIGN LOADS (WORKING LOADS)

Loads applied to floor only

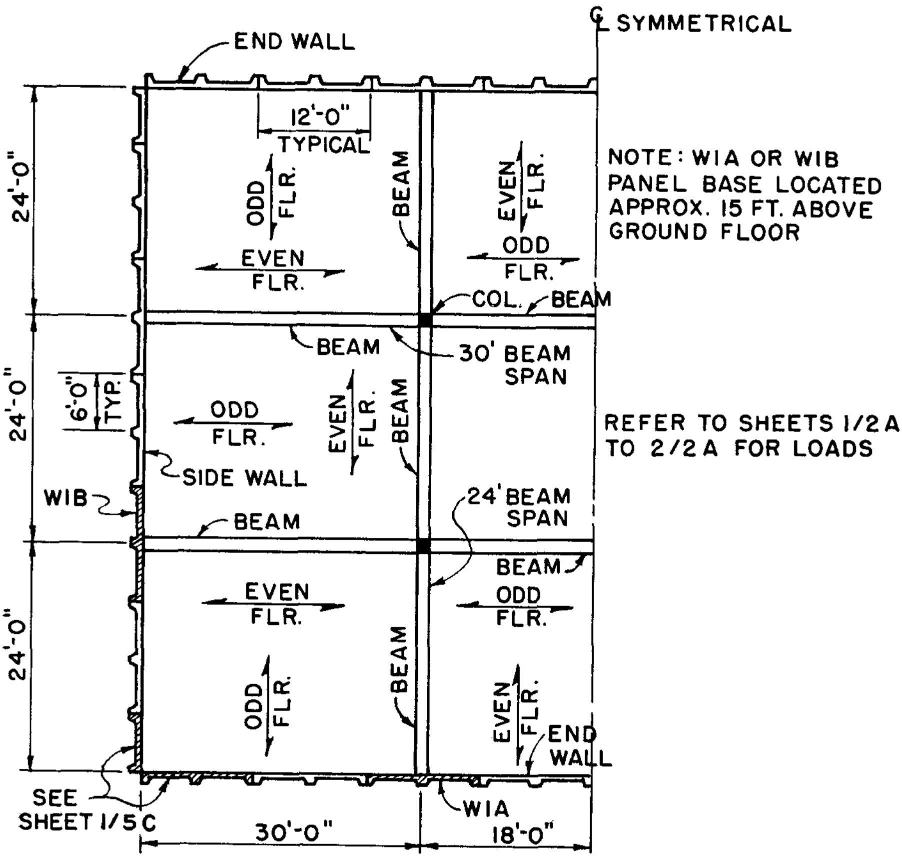
$$W_1 = \overset{(a)}{10} + \overset{(b)}{30} + \overset{(c)}{3} + \overset{(d)}{4} + \overset{(g)}{30} + \overset{(LL)}{16} = 93 \text{ psf (working)}$$

Loads applied to 30-ft beam

$$W_2 = \overset{(a)}{10} + \overset{(b)}{30} + \overset{(c)}{3} + \overset{(d)}{4} + \overset{(e)}{20} + \overset{(g)}{30} + \overset{(LL)}{16} = 113 \text{ psf (working)}$$

Loads applied to 24-ft beam

$$W_3 = \overset{(a)}{10} + \overset{(b)}{30} + \overset{(c)}{3} + \overset{(d)}{4} + \overset{(f)}{13} \quad \overset{(g)}{30} + \overset{(LL)}{16} = 106 \text{ psf (working)}$$



FLOOR PLAN

DETERMINE LOADS TO W1A PANEL

Odd Floors

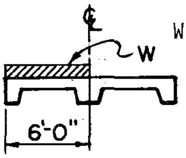
Concentrated Beam Load (use W_3 load, Sheet 2/2 A)

P = 11 (24/2) 36/2 (0.106)	=	251.8 kips (total load)
(90/106)(251.8)	=	213.8 kips (dead load)
		38.0 kips (live load)

Note that the number "11" in the "P" calculations represents the total number of odd floors.

GRAVITY LOADS TO W1 PANEL (CONT.) SHEET 2/3 B

Uniform loads (use W_1 load)



W_{TL} = 11(6)(24/2)(0.093) = 73.7 kips (total load)
(77/93)(73.6) = 61.0 kips (dead load)
12.7 kips (live load)

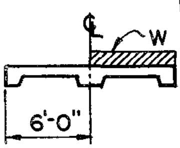
Even Floors

Concentrated beam load (use W_2 load)

P = 12(24/2)(30/2)(0.113) = 244.1 kips (total load)
(97/113)(244.1) = 209.5 kips (dead load)
34.6 kips (live load)

Note that the number "12" in the "P" calculations represents the total number of even floors.

Uniform loads (use W_1 load)



W_{TL} = 12(6)(24/2)(0.093) = 80.4 kips (total load)
(77/93)(80.4) = 66.5 kips (dead load)
13.8 kips (live load)

Panel loads

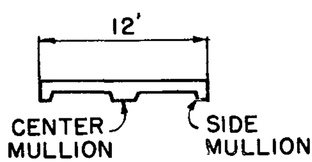
23 panels from first floor to roof.
Estimate the panel weight at 65 psf.

Center mullion load

6(10.042)(65/1000)(23) = 90.1 kips (dead load)

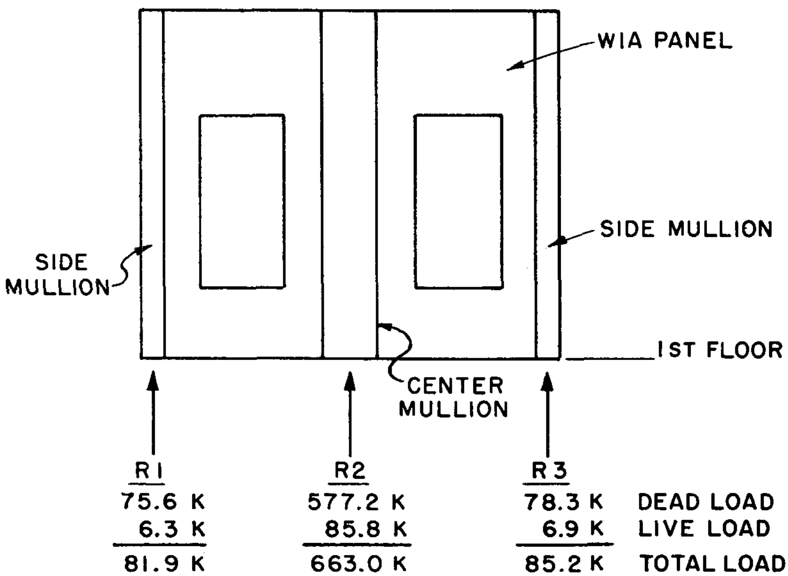
End or side mullion load

3(10.042)(65/1000)(23) = 45.0 kips (dead load)



Total loads applied to W1A

Assume that the panel loads are carried by the mullions only for sizing of mullions.




```
                odd            odd
       mullion unif.DL      unif.LL      DL     LL
R1 = 45.0 + (61.0/2) + (12.7/2) = 75.6 + 6.3 = 81.9 kips

                odd            odd          even          even          odd
       mullion unif.DL      unif.LL      unif.DL      unif.LL      conc.DL
R2 = 90.1 + (61.0/2) + (12.7/2) + (66.5/2) + (13.8/2) + 213.8

      odd         even      even
    conc.LL    conc.DL  conc.LL       DL        LL
       38.0  +   209.5   + 34.6   =   577.2 + 85.8   = 663.0 kips

                even           even
       mullion unif.DL      unif.LL      DL     LL
R3 = 45.0 + (66.5/2) + (13.8/2) = 78.3 + 6.9 = 85.2 kips
```

NOTE: A similar analysis of Panel W1B indicates loads of approximately the same magnitude but slightly less.

LATERAL LOAD DATA

Lateral load forces are obtained from Paper 2. Data for loads applied at first story level is summarized on Sheets 2/3 B1 and 3/3 B1.

Determine if any panel mullions have tension

Treat panels as a group over assumed distribution lengths as on Sheet 3/3 B1 due to connections details that will be employed. Critical panels in corner area.

Find gravity load — side wall acting opposite wind tension

Use mullion dead loads from Sheet 3/3 B that are applied to side mullions.

Avg DL = (75.6+78.3)/2 = 76.9 kips
This load is applied over 3-ft panel length at side mullion.

Avg W_{DL} = (76.9)/3 = 25.6 kips per ft

Gravity load = 18(25.6) = 461 kips (ok)
(That is, no tension and gravity load is greater than 310 kips applied).

Find gravity load — end wall acting opposite wind tension

Avg DL = 25.6 kips per ft (as above)

Gravity load = 24(25.6) = 614 kips (ok)
(That is, no tension and gravity load is greater than 358 kips applied).

Summation of corner gravity and wind loads

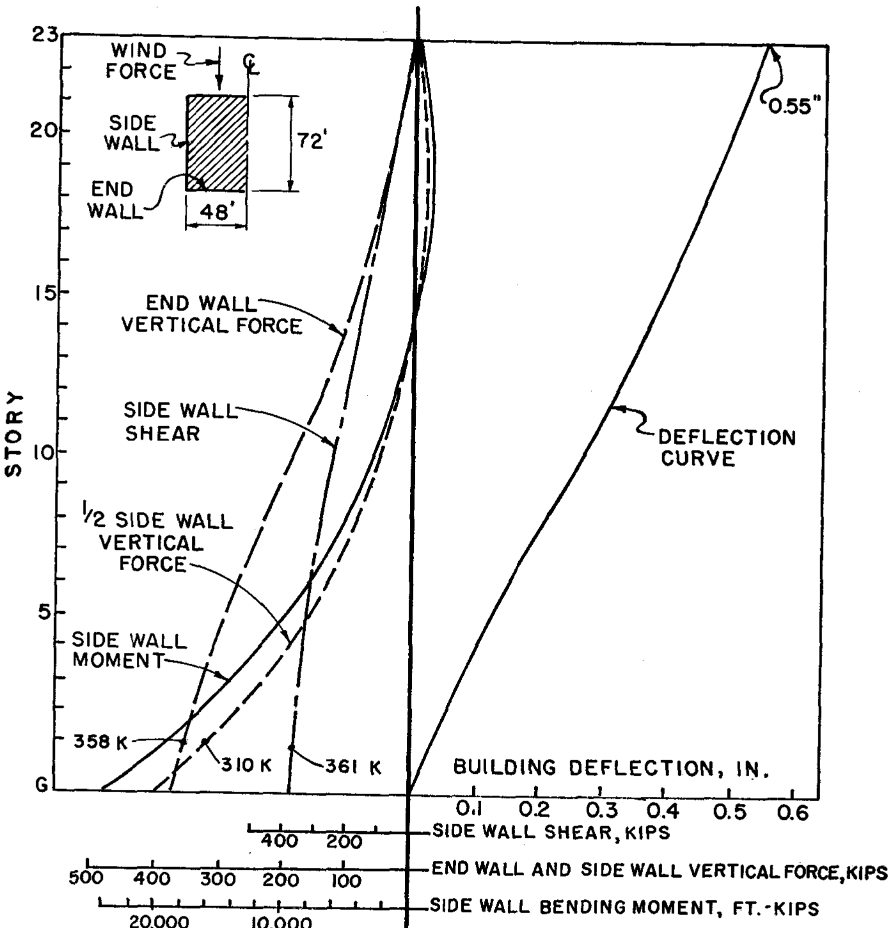
Total gravity force (DL) corners = 461 + 614 = 1075 kips

Total wind force corners = 310 + 358 = 668 kips

Tension safety factor = 1075/668 = 1.61 (ok)
This safety factor is conservative due to the assumed wind force distribution.

PANEL LATERAL LOAD SHEET 2/3 B1

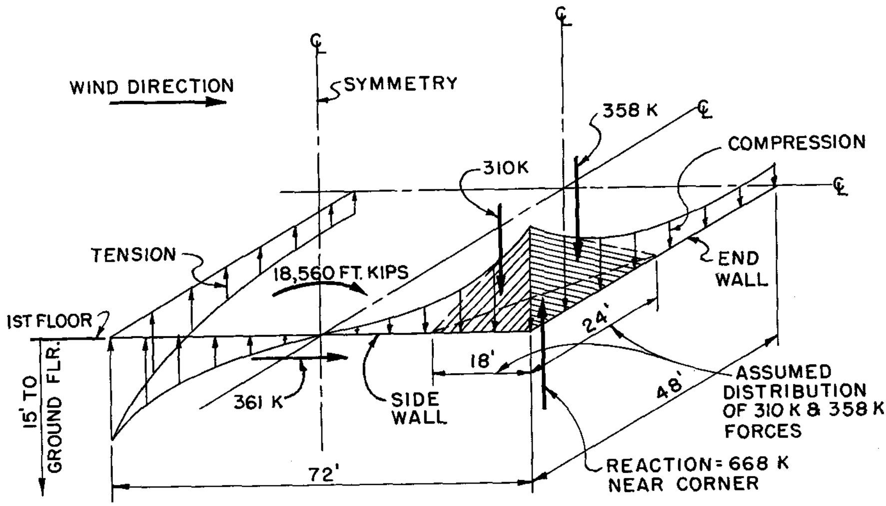
LATERAL WIND BEHAVIOR



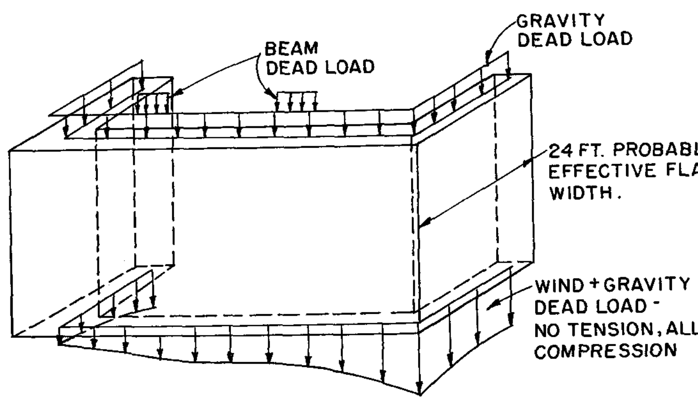
SEE SHEET 3/3 B1

NOTE: ½ SIDE WALL FORCE IS VERTICAL FORCE COUPLE DETERMINED FROM SIDE WALL MOMENT (SEE PAPER 2)

PANEL LATERAL LOAD SHEET 3/3 B1



WIND LATERAL FORCE DISTRIBUTION AT FIRST FLOOR



EFFECTIVE LATERAL LOAD RESISTING PORTION OF BUILDING - VERTICAL LOAD DISTRIBUTION FOR WIND PLUS GRAVITY DEAD LOAD AT FIRST FLOOR

DETERMINE PANEL MULLION SIZE REQUIRED   SHEET 1/5 C

LOAD DATA

Mullion size of panels will be controlled by:

1. Dead plus live load of center mullion of Panel W1A which supports concentrated beam reactions (1.4 D + 1.7 L at ultimate)

2. Dead plus live load plus wind load to mullions at corner of building or dead plus wind load at corner of building. The compression case will control.

$$0.75\ (1.4D + 1.7L + 1.7W) \text{ at ultimate}$$

or

$$0.9\ D + 1.4\ W \text{ at ultimate}$$

Check center mullion W1A panel

Use loads on Sheet 3/3 B.

$$P_u = 1.4D + 1.7L = 1.4(577.2) + 1.7\ (85.8)$$
$$= 954 \text{ kips at ultimate}$$

Additional load due to wind for case 0.75 x (1.4D + 1.7L + 1.7W) is less as can readily be seen from Sheet 3/3 B1.

Check mullions at building corner

Refer to Sheet 1/3 B. Assume a total of only three mullions are effective. Note that each mullion size is taken as a typical center mullion.

$$P_u = 0.75\ (1.4D + 1.7L + 1.7W)$$
$$= 0.75(6)\left[1.4(25.6) + 1.7(2.2) + 1.7(59.7)\right]$$
$$= 634 \text{ kips at ultimate}$$

This is equivalent to typical center mullion at corner.

$D = 25.6$ kips per ft (Sheet 1/3 B1)

$L = \dfrac{6.3 + 6.9}{2\ (3)} = 2.2$ kips per ft (Sheet 3/3 B)

$W = 358/6 = 59.7$ kips per ft (Sheet 3/3 B1)

$$P_u = 0.9D + 1.4W = 6\left[0.9(25.6) + 1.4(59.7)\right]$$
$$= 640 \text{ kips at ultimate}$$

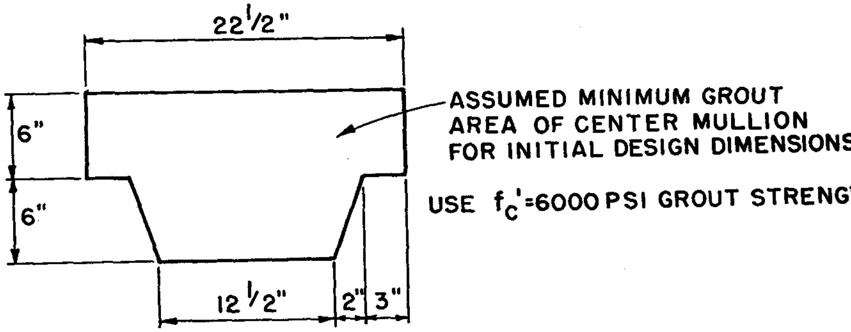
This is equivalent to typical center mullion at corner.

Note: Center mullion of W1A panel controls for dead plus live loads.

SIZE CENTER MULLION

Note: Sizing of the center mullion will be controlled more than likely by connections between mullions. Good design practice dictates that loads be carried by mullions. Further, connection economy requires that if mullion-to-mullion connection is completed by grouting, then grouting should be no more than required. Also, it is good design practice not to require additional "watch-like" hardware to make mullion-to-mullion connection resist design loads. Thus, use minimum reinforcement at mullion-to-mullion connection.

CHECK LOAD CAPACITY OF INITIAL MULLION SIZE
AT GROUT AREA (CENTER MULLION)



USE f_c'=6000 PSI GROUT STRENGTH

Area = 6(14.5) + 22.5(6) = 222 sq in.

P_u = 0.7(0.85) (6000/1000)(222) = 792.5 kips(not ok)

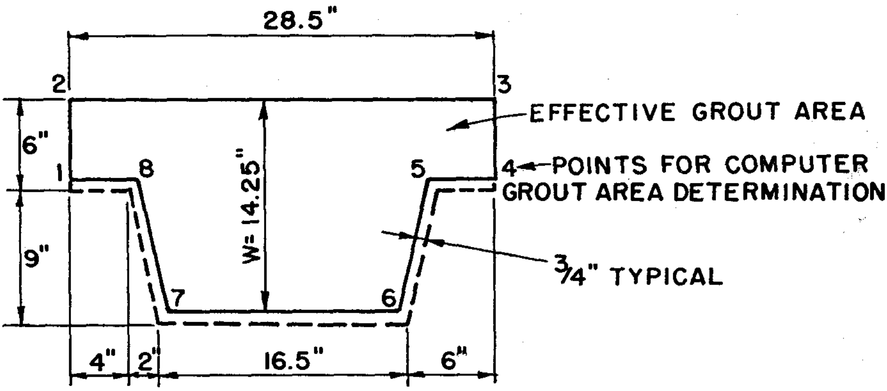
Note that this load is less than the 954 kips (Panel W1A) required.

Also, the actual grout area will be less due to joint appearance requirements.

Therefore, revise the center mullion size.

REVISED MULLION SIZE (Center Mullion — W1A Panel)

Several trials were made but are not shown here.
A suggested mullion size is given below.



See Sheet 4/5 C for grout area (about 303 sq in.).

CHECK GROUT AREA CAPACITY AT ULTIMATE

Determine capacity by Section 6.1.6 PCI Design Handbook (use Eq. 6-11)

$W = 14.25$ in. for bearing length
$S = 6.14$ (minimum value)

$\frac{S}{W} = \frac{6.14}{14.25} = 0.43$

Use $T_u/V_u = 0$

This ratio is realistic since mullion-to-mullion details will most probably use plate details.

$$f_{bu} = C_r \Phi 70 \sqrt{f'_c} (S/W)^{1/3} \qquad C_r = 1 \text{ for } T_u/V_u = 0$$

$$f_{bu} = 1.0(0.7)\, 70\sqrt{6000}\, (0.43)^{1/3}$$
$$= 2870 \text{ psi at ultimate}$$

Find P_u using two No. 8 bars within joint

$$P_u = (303/1000)(2870) + 2(0.79)\, 60$$
$$= 870 + 95 = 965 \text{ kips (grout area capacity at ultimate)}$$

This value is satisfactory since it is greater than 954 kips.

Therefore, 6000-psi grout is satisfactory.

```
ARCHITECTURAL PRØGRAM #1

TEKCØM CØRP.**HINSDALE,ILL**312/986-0460


INPUT JØB NØ.. P/C MARK, CRØSS-SECTIØN MARK ?PCI-23,C-MULL,GRØUT

INPUT NUMBER ØF NØDES IN ØUTER PERIMETER ?8
INPUT X,Y CØØRDINATES FØR EACH NØDE:
NØDE1 ?0,9
NØDE2 ?0,14.25
NØDE3 ?28.5,14.25
NØDE4 ?28.5,9
NØDE5 ?23.75,9
NØDE6 ?21.75,0
NØDE7 ?6.75,0
NØDE8 ?4.75,9
INPUT NUMBER ØF NØDES IN ØPENING 1 ?0
INPUT NUMBER ØF CØNCENTRATED PØINTS ?0

***************************************************************

ARCHITECTURAL PRØGRAM #1

TEKCØM CØRP.**HINSDALE,ILL**312/986-0460


 JØB NØ. PCI-23   P/C MARK C-MULL   CRØSS-SECTIØN GRØUT

THE CØØRDINATES ØF THE CENTER ØF GRAVITY ARE:

  X = 14.2500         Y = 8.11199

TØTAL AREA =       302.625 SQ. IN.          2.10 SQ. FT.


SECTIØN PRØPERTIES ABØUT THE X-X AXIS THRU THE CENTER-ØF-GRAVITY:

         YB                YT                SB(IN3)     ST(IN3)     IXX(IN4)
       8.112 IN.         6.138 IN.           619.335     818.513     5024.041
        .676 FT.          .512 FT.


SECTIØN PRØPERTIES ABØUT THE Y-Y AXIS THRU THE CENTER-ØF-GRAVITY:

         XL                XR                SL(IN3)     SR(IN3)     IYY(IN4)
       14.250 IN.        14.250 IN.          972.877     972.877     13863.492
        1.188 FT.         1.188 FT.


***************************************************************

INPUT 1 FØR RERUN, 0 IF CØMPLETE ?0
```

DESIGN SUMMARY

1. The design of grout area is probably conservative due to the confinement by the plates.

2. Increasing the grout area size by more grout under the 6-in. panel portion decreases "S" and therefore will reduce f_{bu}.

3. The grout area selected is consistent with the economical concept of minimizing grout.

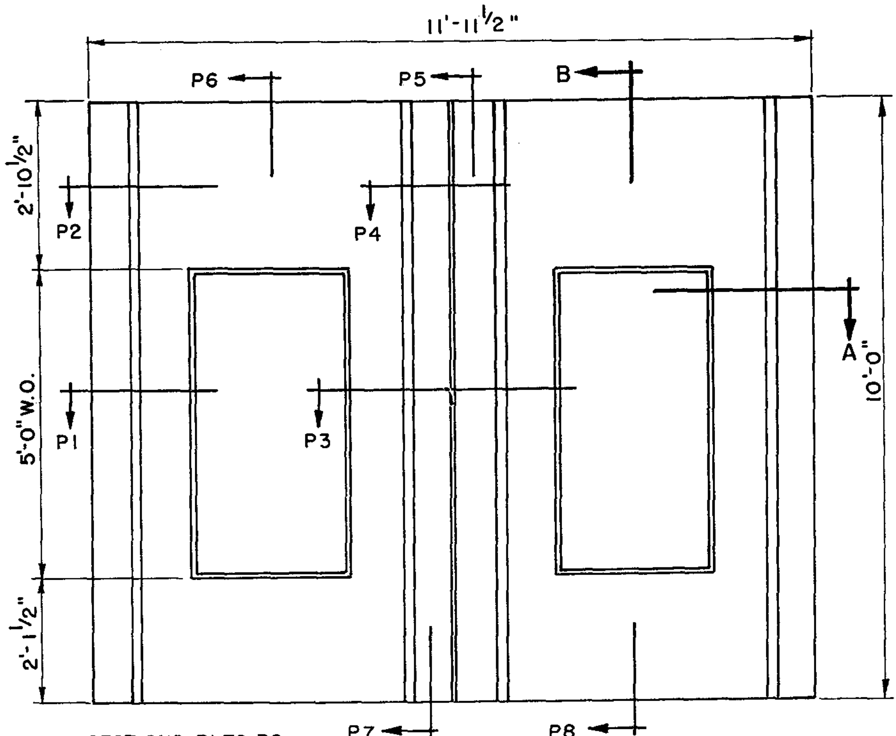
4. When details of grouting in terms of vent tubes are developed, the procedures should be reviewed by grouting contractors or with a precast erector.

5. The design of a mullion just above or below the grout joint requires more bearing (as shown above) than ordinary column design. Additional mullion ties should be placed near a joint. For example, ties at 4 in. center to center over a 2-ft length.

See Sheets 1/5 C4 to 3/5 C4 for mullion grout joint.

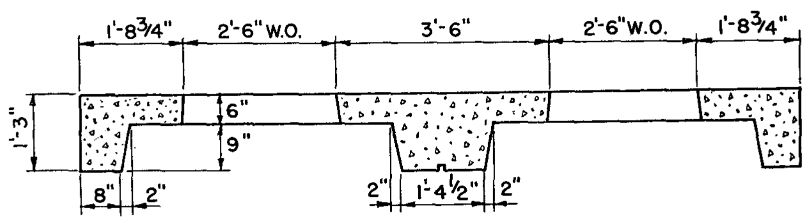
PRELIMINARY W1 PANEL SHAPE & DIMENSIONS

SHEET 1/2 C1



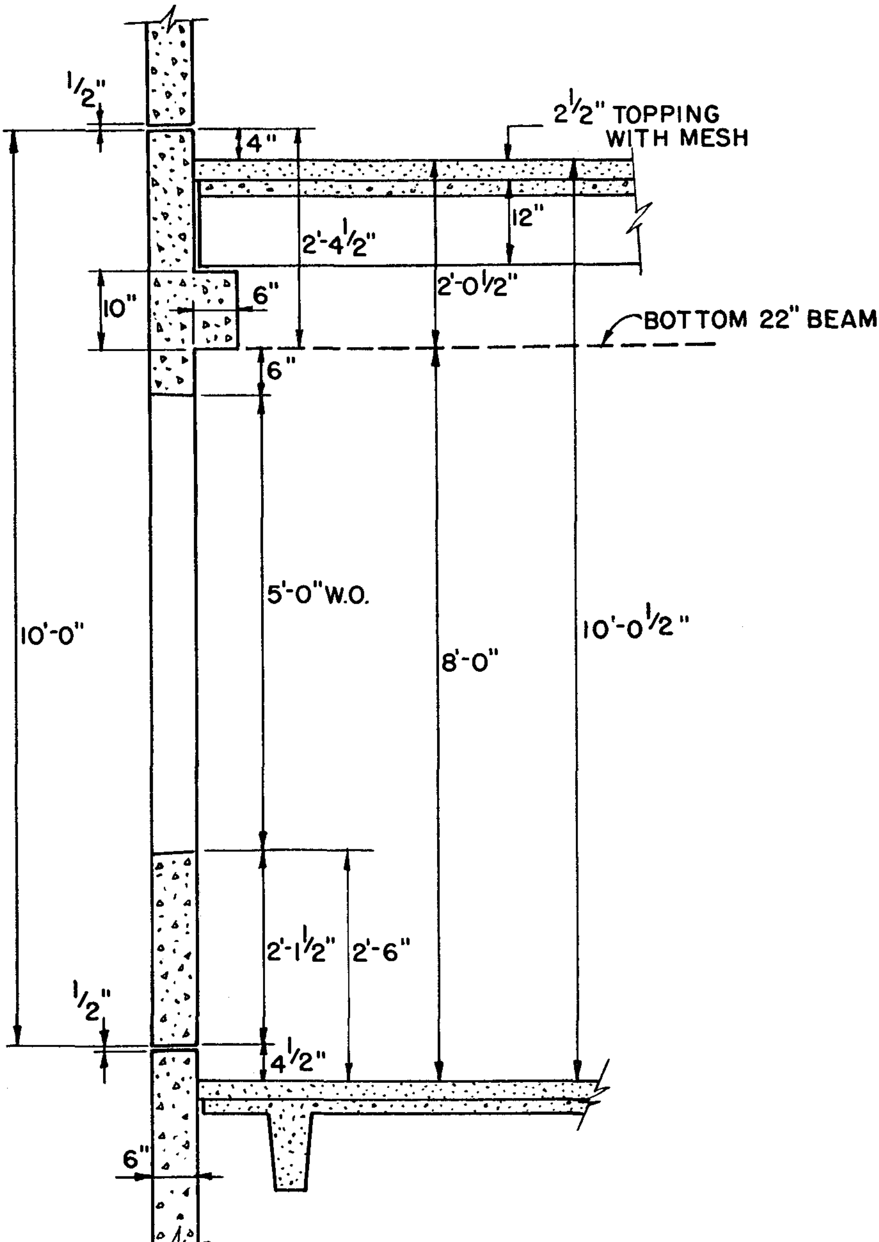
SECTIONS P1 TO P8
SEE SHEET 2/10S

ELEVATION - TYPICAL W1 TYPE PANEL
(PANEL WT. ≅ 8260 LBS. AT 110 PCF)



SECTION A

PANEL SHAPE & DIMENSIONS SHEET 2/2 C1



SECTION B

(ALSO SHOWS RELATION OF PANEL TO STRUCTURE)

DESIGN OF W1 PANEL SHEET 1/4 C2

FLOOR SUPPORT CONNECTIONS

DESIGN PANEL HAUNCH

Haunch Loads-Typical (refer to Sheet 2/2 C1)

Load from double tee floor slab bearing on panel (see Sheet 1/2 A)

DL floor = 77 psf
LL floor = 40 psf (non-reduced)
117 psf

Load factor $= \dfrac{1.4(77) + 1.7(40)}{117} = 1.50$

Load per tee stem

1. Design the section for a concentrated load (not as a uniform load).
2. Use an additional load factor of 4/3 to insure that the connection is stronger than the member being supported.

$V_u = (6/2)(30/2)\ 0.117(1.5)(4/3) = 10.53$ kips at ultimate

$T_u = 0.2\ V_u = 0.2(10.53) = 2.1$ kips at ultimate

Note that the coefficient 0.2 is selected because of the neoprene bearing pad (see p.6-15 of the PCI Design Handbook).

Select Haunch Size

The haunch depth should not project below the bottom of the beam framing into the wall panels. Also the practical size would be 8 to 10 in. Haunch depth will also be partly determined by considering the beam-to-panel connection. Try a depth equal to 10 in. as shown on Sheet 2/2 C1.

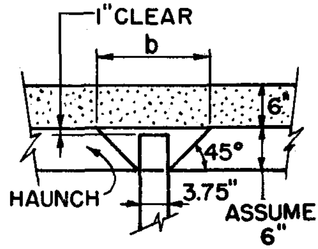
Check ultimate shear stress

Effective width

$b = 6 + 6 + 3.75 = 16$ in.

$v_u = \dfrac{10{,}530}{16 \times 8.7}$ (Note: 8.7 in. is the estimated depth d)

$= 76$ psi at ultimate



PLAN VIEW-HAUNCH

Since this stress is less than 800 psi, the section is satisfactory. Therefore, for design purposes use a 10-in. depth.

Select Haunch Reinforcement

Base the design on Section 6.1.11 (PCI Design Handbook)

Estimate $\frac{\ell_v}{d} = \frac{1+(2/3)5}{8.7} = 0.5$

C_1 = 4.87 from Table 6.1.4 (PCI Design Handbook)

Find C_2 Required for Loads

Using Eq.6-9 (PCI Design Handbook)

$10{,}530 = 0.85(16)8.7(0.85)\sqrt{6000}\,(4.87)\,C_2$

Note that the factor 0.85 reflects the use of lightweight concrete (see ACI 318-71).

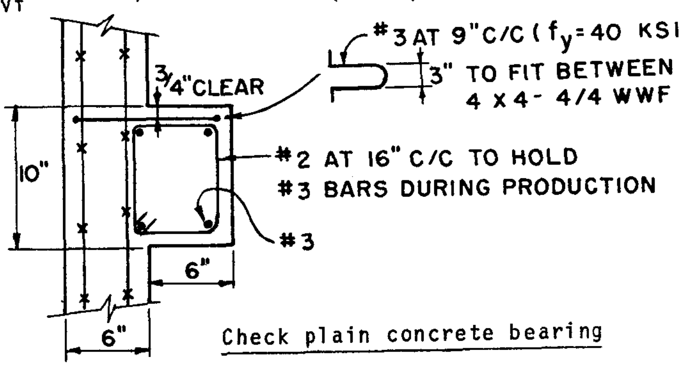
C_2 = 0.277. From Table 6.1.5 (PCI Design Handbook), select P_{vf} = 0.004.

This is the minimum value for the ratio T_u/V_u = 0.2.

Select A_{vf}

$A_{vf} = P_{vf}\,bd = 0.004(16)8.7 = 0.56$ sq in. per 16 in.

$A_{vf} = 0.56/1.33 = 0.42$ sq in. per ft



Check plain concrete bearing

$f_b = \frac{10{,}530}{(4/3)1.5(3.75)5}$ = 281 psi at working load

Use a 1/4 in. neoprene bearing pad (Section 6.1.7 PCI Design Handbook)

$f_{bu} = \frac{10{,}530}{3.75(5)}$ = 562 psi at ultimate load

The haunch is satisfactory if the bearing stress is less than 0.5 f'_c (see Section 6.1.11 of PCI Design Handbook). The maximum ultimate plain concrete bearing stress is 1480 psi for T_u/V_u = 0.2 and S/W = 0.5 (see Table 6.2.1, PCI Design Handbook).

Bearing is satisfactory since 562 psi is less than 1480 psi.

DESIGN BEAM SUPPORT

Panel support load

Odd floor Panel W1A (see Sheet 1/3 B) controls

Live load reduction = (36/2)(24/2) 0.08 = 17 percent

Live load for design = 0.83(40) = 33 psf

Dead load = 77+13 = 90 psf (see Sheet 1/2 A for calculating "13")

Total load = 123 psf (working)

Use a load factor equal to 1.5 as before.

V_u = 4/3(1.5)(36/2)(24/2) 0.123 = 53.1 kips at ultimate

Select haunch type

1. Due to size limitations, use an embedded structural shape that will fit within 10-in. tee support haunch.

2. A design check (not shown) indicates that the dapped beam will be satisfactory.

   That is, v_u = 402 psi for a dapped beam. (See Section 6.1.10 of PCI Design Handbook).

Find concrete width "b" required

From Table 6.2.5 (PCI Design Handbook).

t_c = 14 in. and projection = 6 in. (the actual projection is less)

$$b_{req} = \frac{5000}{6000}(5.5) = 4.6 \text{ in. minimum}$$

Note that "5.5" reflects the Table 6.2.5 value for 53.3 kips. The concrete strength "6000 psi" is used to modify width "b."

Select embedded shape

Try two C7x12.25(A36) steel sections. Using the PCI Manual on Design of Connections for Precast Prestressed Concrete, make sure that the width b (4.39 in.) and haunch depth are satisfactory.

V_u = 87 kips (ok, greater than 53.3 kips)

$V_u = 0.55\ F_y td$ (AISC 2.5-1)

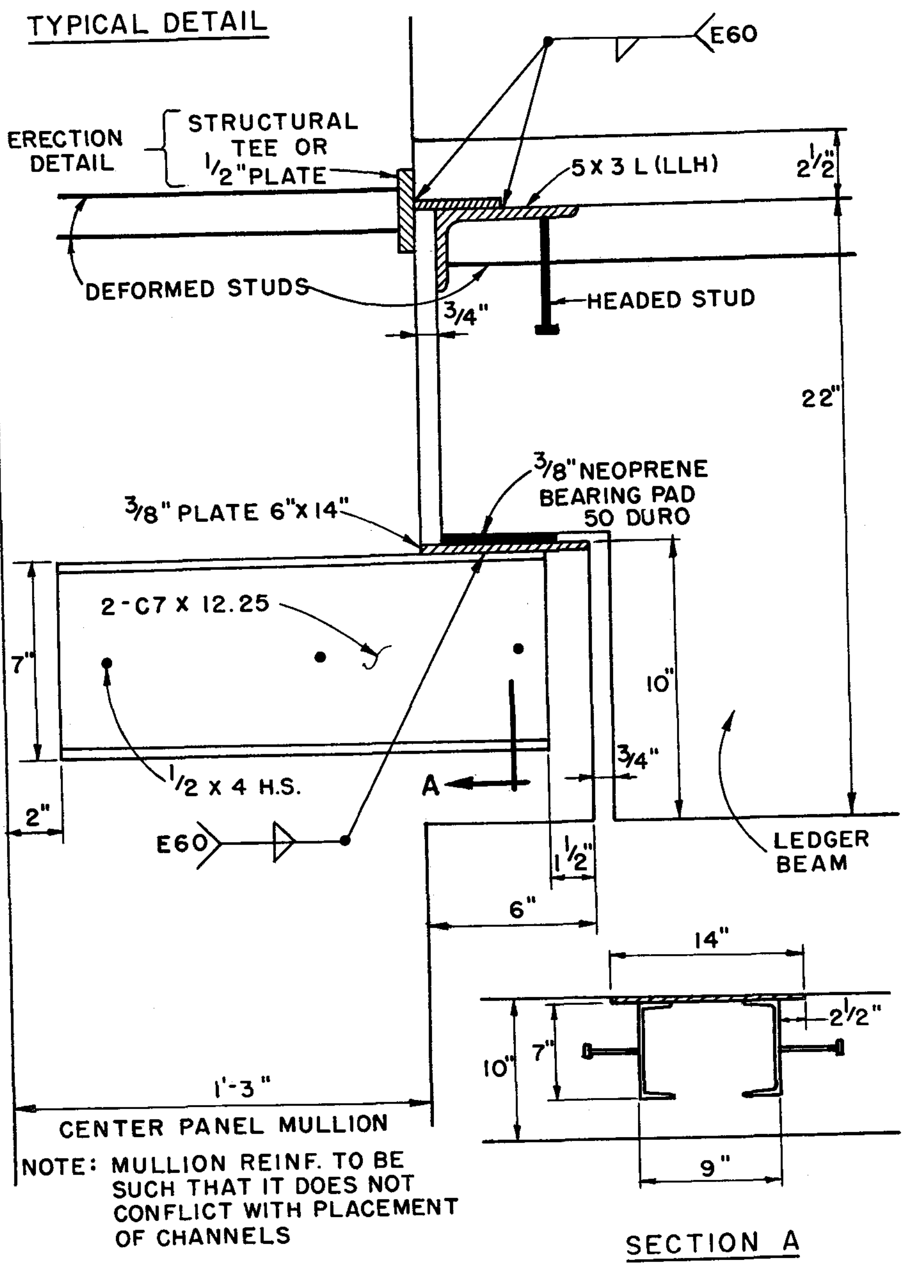
Check bearing pad on steel haunch

Use a bearing area = 12(3.75) = 45 sq in.
where "12" is the beam width.

$$f_b = \frac{53{,}100}{(4/3)(1.5)(45)} = 590 \text{ psi}$$

A 3/8-in. bearing pad will be satisfactory.

FLOOR SUPPORT CONNECTIONS SHEET 4/4 C2



CONNECTION DESIGN FOR LATERAL LOADS

DETERMINE LATERAL LOAD SHEARS

1. Forces applied by controlling wind case are given on Sheet 3/3 B1.
2. Use the ultimate procedure (Sections 6.1.8 and 6.1.9 of the PCI Design Handbook).
3. Use panel gravity loads summarized on Sheet 3/3 B.
4. Design for a typical panel between the first and second floors on the 72-ft sidewall.

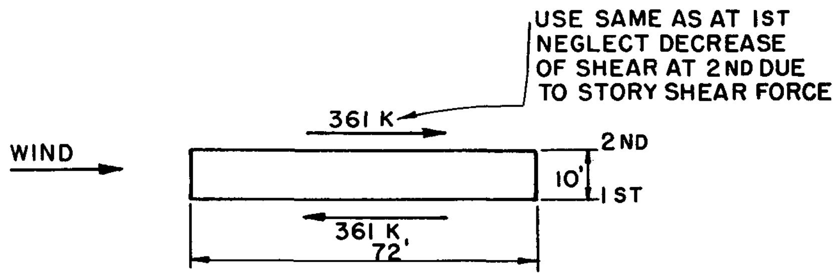
Determine panel joint shears

Panel joint shears result from two conditions:

1. Horizontal shear due to lateral load.
2. Vertical shear due to flexural behavior of building acting as a cantilever.

Horizontal shear only

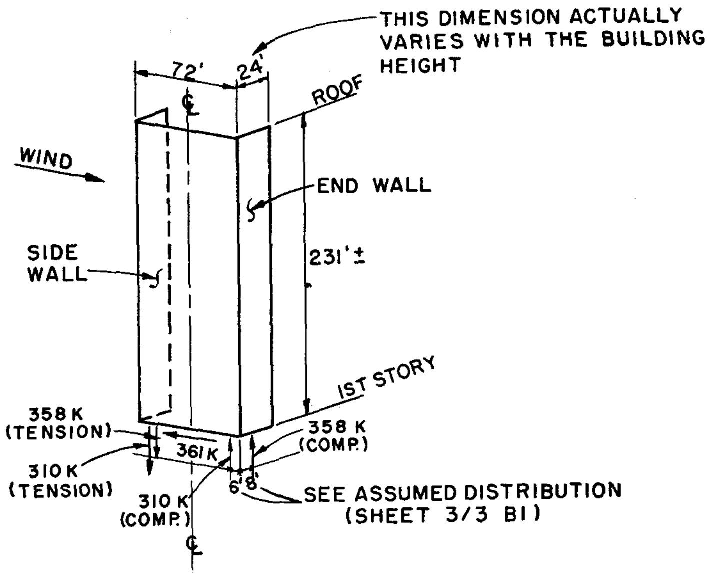
Horizontal shear per story = 361 kips



Vertical shear only

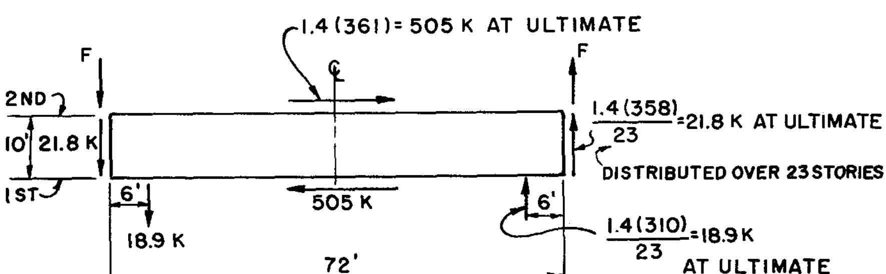
Results from tension and compression forces due to bending (see Sheet 3/3 B1).

The maximum vertical shear forces occur at the panel vertical joints away from the corners.



DETERMINE ULTIMATE DESIGN SHEAR FORCES

Use the ultimate composite approach. This method assumes that the ultimate shear is uniformly distributed on the shear failure plane. Note that in the elastic range, the sidewall will basically act as a total unit.



FORCES APPLIED TO ONE STORY SHEAR WALL

NEGLECT THAT SHEAR FORCES AT FIRST AND SECOND SLIGHTLY DIFFERENT DUE TO STORY SHEAR

Check Summation of Moments

The summation of moments about the centerline in a clockwise direction should equal zero.

505(10) - 18.9(60) - 21.8(72) = 5050 - 1134 - 1570
= 2346 ft-kips (clockwise)

To achieve equilibrium it is necessary to induce a force "F" shown on Sheet 3/3 C3. The connection at the corner capable of transferring F + 21.8 kips must be used. This force of F + 21.8 kips is resisted by the dead load weight of the end walls (see Sheet 1/3 B1).

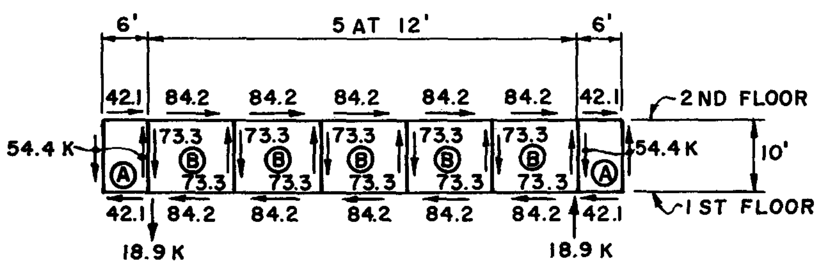
Find Force F

The summation of moments about the centerline must equal zero (clockwise positive).

- 72 F + 2346 = 0

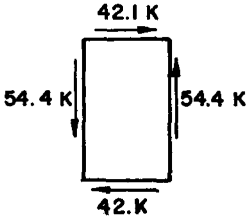
Therefore, F = 32.6 kips at ultimate

DETERMINE HORIZONTAL AND VERTICAL ULTIMATE PANEL JOINT SHEAR FORCES



SUMMARY OF APPLIED LOADS

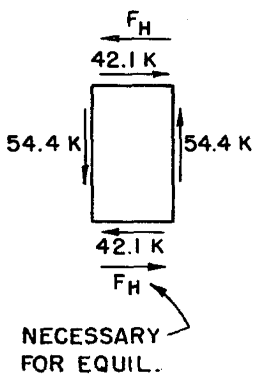
FIND JOINT SHEAR FORCES - PANEL Ⓐ



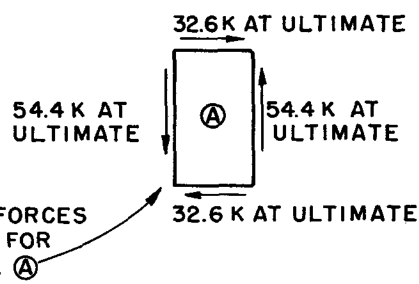
Σ M = 0 +

-6 (54.4) + 10 (42.1) = -326.4 + 421 =
+ 94.6 K' AT ULTIMATE
NOT BALANCED
ADJUST SHEARS

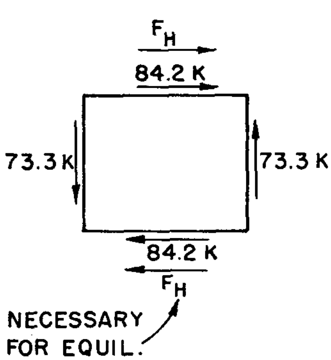
VERTICAL SHEARS ON PANEL Ⓐ CAN NOT CHANGE DUE TO EQUILIBRIUM REQUIRENTS AT CORNER. HORIZONTAL SHEAR MUST THEN VARY.



$F_H = 42.1 - \frac{94.6}{10} = 9.5$ KIPS



FIND JOINT SHEAR FORCES-PANEL Ⓑ



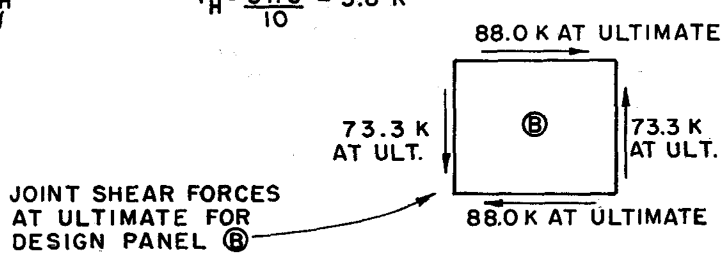
$\Sigma M = 0 \; +\!\circlearrowright$

$84.2(10) - 73.3(12) = 842 - 879.6 =$

-37.6 FT. KIPS NOT BALANCED

VERTICAL JOINT SHEARS CAN NOT CHANGE AS DISCUSSED ABOVE - FIND CORRECTED HORIZONTAL SHEAR.

$F_H = \frac{37.6}{10} \cong 3.8$ K



Check the Summation of Horizontal Shear Forces

For equilibrium the summation of forces in the x direction must equal zero.

Story shear = 505 kips at ultimate

$$2(32.6) + 5(88.0) = 65.2 + 440 = 505 \text{ kips at ultimate}$$

This shear force is satisfactory since it is the same as the applied 505 kips force.

Summary of Shear Force Distribution

1. The flexural vertical shears influence the distribution of the lateral wind horizontal shear only slightly as shown in the calculations.

2. The distribution of joint shears at ultimate is based on an ultimate composite approach. Because of slip or deformation it can be assumed that the shear forces are uniform on the joint failure plane.

3. The distribution of shear forces as assumed provides a force path for the forces. Thus, minor variations will not affect the overall ultimate capacity or the building behavior.

4. Note that the building weight is important in that the end wall weight must be sufficient to develop equilibrium. If the end wall weight was less than 32.6 kips, it would be necessary to develop additional forces by tension anchors to the foundation.

5. In addition to assuming a uniform distribution of vertical shear at the corner for ultimate, a check for corner shear variation must be made. The wind load produces a maximum working story vertical shear of 21.5 kips at the eighth floor. Since all the vertical joint shear connections will be the same, the connections developed for the lower stories should be adequate for the maximum shear at the eighth story.

HORIZONTAL PANEL JOINT SHEAR

Design Criteria

1. Design for shear applied to the side wall at the first story, V_u = 505 kips (see Sheet 2/11 C3).

2. Use the ultimate procedure (see Sections 6.1.8 and 6.1.9 of PCI Design Handbook).

Find Ultimate Shear Stress (Horizontal Joint)

The approximate grout area per 12-ft panel length is:

2(303) = 606 sq in. (see Sheet 3/5 C).

v_u = 88,000/606 = 145 psi (ok, less than 800 psi).

Determine Friction Resisting Capacity

Refer to Section 6.1.8 of PCI Design Handbook. Assume that only the compression side of the side wall is effective. Neglect the additional load due to wind compression. The total dead load due to one-half of the side wall is: 2(75.6)6 + 577.2 = 1484 kips

See Sheet 3/3 B for calculating the dead load. Note that the "6" above represents the number of mullions and the "577.2" is the weight of the mullion with the beam load.

Using ACI 318-71 the required dead load is: 0.9(1484) = 1336 kips

Now $F_s = \mu V_u$ = 0.4(1336) = 534 kips (ok, greater than 505 kips)

Note that the shear friction coefficient μ is that between the concrete and steel (see Table 6.1.2 of PCI Design Handbook).

Find A_{vf} per Section 6.1.9 of PCI Design Handbook

V_u = 88/2 = 44 kips per center mullion of typical 10x12-ft panel

Assume that T_u/V_u = 0.2 due to horizontal volume change forces.

f_y = 50 ksi for 1-in. coil rod. Using Table 6.2.2;

A_{vf} = 0.9 sq in. for V_u = 40 kips and μ = 1.4.

From Table 6.1.3 select μ = 1.0. Then:

A_{vf} = 0.9(1.4/1.0) = 1.26 sq in. Use two No. 8 coil rods

having A_{vf} = 1.58 sq in. (ok, greater than 1.26 sq in.)

v_u = 40,000/303 = 132 psi

Note that no friction coefficient reduction is required since v_u is less than 600 psi.

DESIGN SUMMARY

1. Use mullion to mullion detail shown on Sheet 2/5 C4.
2. Design of ultimate shear capacity by either Sections 6.1.8 or 6.1.9 is conservative.
3. Use reinforcement between mullions whether required by design or not. It is good design practice to tie buildings together.
4. At tops and bottoms of all mullions, use additional confinement ties.

FLOOR DIAPHRAGM SHEAR CONNECTION

Design Criteria

1. The maximum diaphragm shear force per floor occurs at the upper stories where the wind pressure is 40 psf (see Uniform Building Code).
2. The design example shown is for 20 psf wind force at the first story. This solution is part of the vertical joint shear design in this C3 calculation section.
3. Use Section 6.1.9 (PCI Design Handbook) with μ = 1.0 for a roughened surface (see Sheet 4/5 C4 for optional spaced keys).

Find Ultimate Shear Force

The shear force per side wall at the first floor is:

$(20/100)(10.042)(96/2) = 9.64$ kips (working)

The ultimate shear force per foot of side wall is:

$1.4(9.64)/72 = 0.196$ kips per ft

Check the ultimate shearing stress (topping)

$$v_u = \frac{196}{12(2.5)} = 6.5 \text{ psi at ultimate}$$

This shear stress is satisfactory since it is less than the 600 psi maximum for the case in which the coefficient of friction is 1.0.

LATERAL LOAD CONNECTIONS SHEET 8/11 C3

Find Area of Connection Reinforcement A_{vf} for Diaphragm Shear

The area of shear friction reinforcement required, A_{vf}, is only a very minimum amount. This area can be found by adding the shear forces required (V_u = 0.196 kips per ft) for the vertical joint shear. Alternately, temporary erection connections can be used.

DESIGN SUMMARY

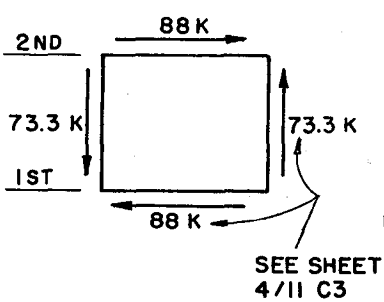
1. Compared to other connection requirements, very little, if any, connection reinforcement is required for diaphragm shear per floor.
2. Temporary erection connections such as Section F1 on Sheet 5/5 C4 provides sufficient A_{vf} reinforcement for the floor diaphragm shear.

VERTICAL PANEL JOINT SHEAR CONNECTION

Design Criteria

1. Design for shear in vertical joints resulting from wind shear of 361 kips at first floor.
2. Note that since overturning or cantilever action of building under wind load does not cause any tension in side wall mullions, the only vertical joint shear is that which occurs in Item 1 above (see also Sheet 3/3 B1).
3. Use ultimate procedures per Section 6.1.9(PCI Design Handbook).

ULTIMATE DESIGN SHEAR FORCES PER TYPICAL 10x12-ft PANEL



Approximately the same shear forces are obtained by:

(505/6) = 84.2 kips per 12-ft panel

(10/12)(84.2) = 70.2 kips vertical shear

Note that the "6" above represents the number of panels per side wall.

Check Panel Cross Sections for Shear

Horizontal nominal shear

Panel concrete area = 418.5 + 2(205.5) = 829 sq in.
(P3 = 418.5, P1 = 205.5)

(See Sheet 2/10 S for P1 and P3).

$$v_u \doteq \frac{88,000}{\phi(829)} = 125 \text{ psi applied at ultimate}$$

The shear stress allowed (somewhat conservatively) is:

$$2\sqrt{f'_c} = 2(0.85)\sqrt{6000} = 132 \text{ psi (ok, greater than 125 psi)}$$

Note that the factor "0.85" is for lightweight concrete.
See ACI 318-71, Sections 11.4.1, 11.4.2, and 11.4.3.

Vertical Nominal Shear

Panel concrete area = 5(12)6 = 360 sq in.(neglecting panel haunch)

$$v_u = \frac{73,300}{\phi(360)} = 239 \text{ psi applied at ultimate (greater than 132 psi)}$$

Provide shear reinforcement above and below window areas.
Note that the stripping reinforcement is:

A_v = 5(0.12)2 = 1.20 sq in. per ft
where 0.12 is the area in sq. in per ft of a WWF and 60,000 psi is its yield strength.

$$A_v = \frac{(v_u - v_c)\, b_w s}{f_y} \quad \text{Eq.(11-13) of ACI 318-71}$$

$$= \frac{(239-132)6(12)}{60,000} = 0.13 \text{ sq in. per ft}$$

This area is satisfactory since 0.24 sq in. is provided.

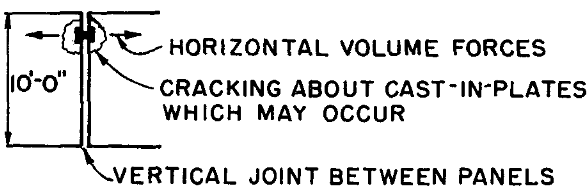
Note: This is only an approximate analysis and does not consider deep-beam vierendeel truss behavior. A more refined analysis is probably not required.

Design A_{vf} Reinforcement for Vertical Joint Shear

$V_u = V_{uv} = 73.3$ kips at ultimate

Use a grouted shear friction connection as shown by Section E (see Sheet 4/5 C4). This type of connection should be used rather than weld plates because:

1. Weld plates at vertical panel joints attract panel volume change deformation forces. These plates can be subject to very large forces in the plane of the panels which are highly indeterminate and can reduce the strength of the shear connection.



2. A grouted type of joint allows for some horizontal movement without damaging the connection capacity. This also simplifies panel production.

Determine A_{vf} Required

Assume $T_u/V_u = 0.20$ as a minimum for design.

$A_{vf} = 1.25$ sq in. (where $f_y = 60{,}000$ psi)

Use Table 6.2.2 (PCI Design Handbook) for a shear friction coefficient equal to 1.4 (see Section E, Sheet 4/5 C4).

Place the reinforcement in the topping and locate grout joint near topping, say start at 6 in. down from top of panel. Use three No. 5 bars and consider topping mesh plus tee flange weld plates.

Determine Size of Grout Joint

Use the maximum ultimate shearing stress at 800 psi to insure that the A_{vf} reinforcement will yield.

A_c = 73.300/800 = 91.6 sq in.

Use a 11x10-in. grout key as shown on Sheet 4/5 C4.
Use the same key size for all joints.

DETERMINE A_{sh} REINFORCEMENT

The area of reinforcement, A_{sh}, to prevent horizontal cracks is for the connection indicated in Section F, Sheet 4/5 C4. Note that this reinforcement is required to insure that the vertical joint shear force A_{vf} does not have a failure plane at the interface of the topping and back of the panel. Use the indicated option of spaced keyways to obtain μ = 1.0.

The force in three No. 5 bars at ultimate plus WWF equals:

1.25(60) = 75 kips (see Sheet 5/6 C3 for A_{vf})

A_{sh} = 1.43 sq in. from Table 6.2.3, PCI Design Handbook for f_y = 40 ksi.

The above reinforcement must be spaced over a 12-ft panel length.

Try using 1/2-in. coil rods into inserts cast in the panel shown in Section F, Sheet 4/5 C4.

A_{sh}/ft = 1.43/12 = 0.12 sq in. per ft.

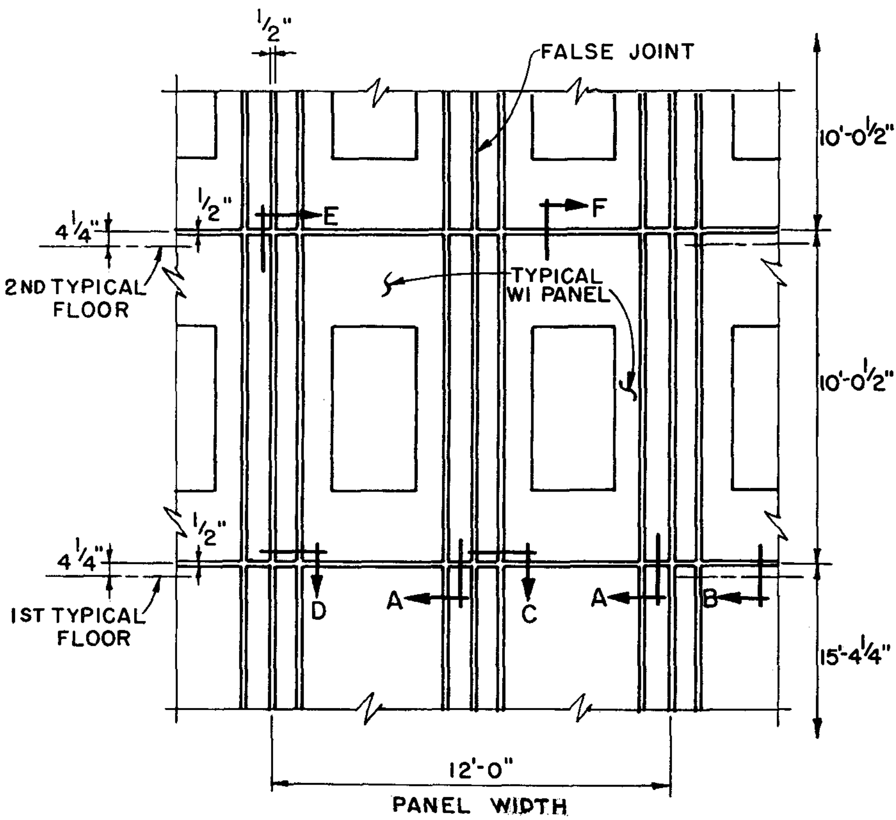
By inspection select 1/2-in. coil rods at 18 in.on center.

DESIGN SUMMARY

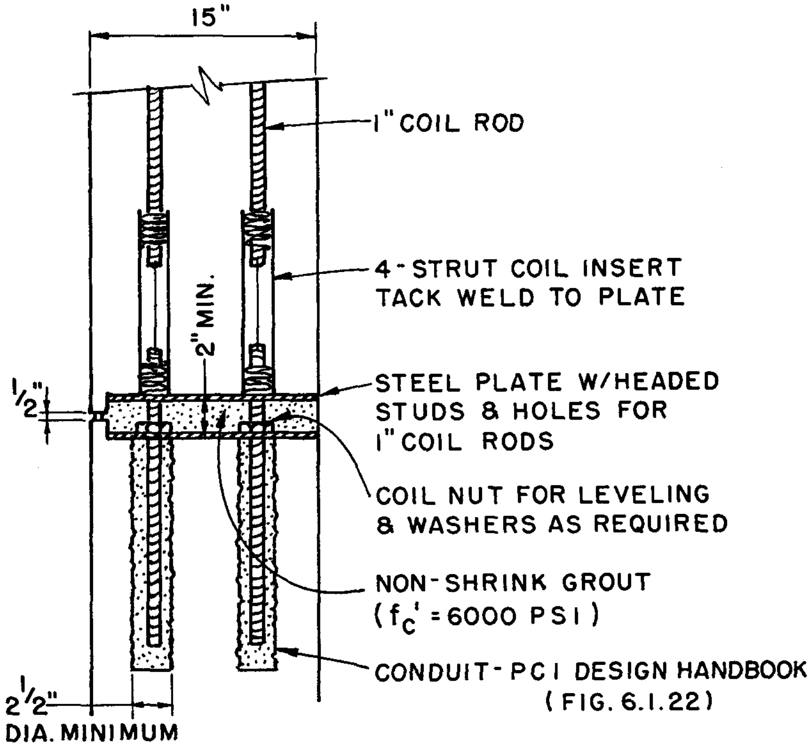
1. Check the inser concrete pullout capacity when selecting inserts.
2. The 1/2-in. coil rod should have a minimum length of "2ℓd" or 24 in.
3. The 1/2-in. coil rod selected should allow for easy placing in the topping and a good tolerance.
4. It will be necessary to modify the joint detail at building corners due to the panel arrangement preventing grout keys. For this case, a weld plate detail would be designed to resist vertical shear forces plus the horizontal volume forces.

   Note: The corner detail should be revised to include a grout-type joint.

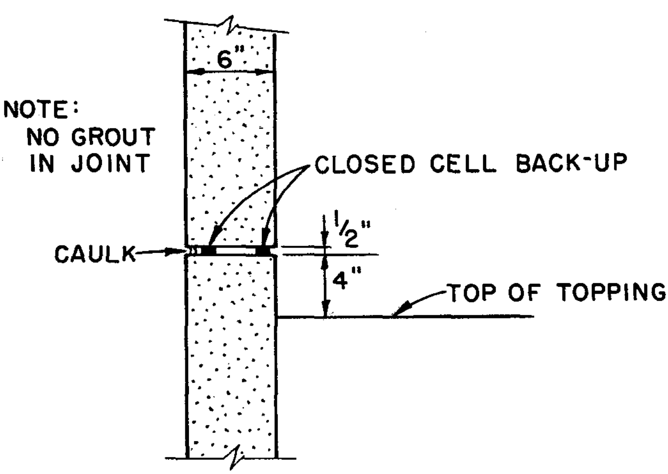
TYPICAL PANEL CONNECTIONS SHEET 1/5 C4



PARTIAL ELEVATION - 72 FT. SIDEWALL

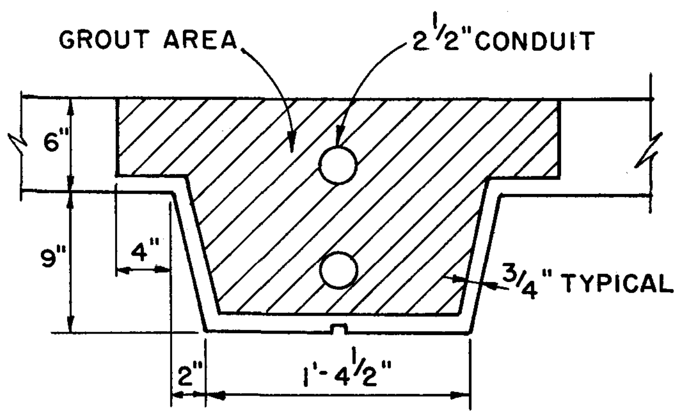


SECTION A-TYPICAL MULLION CONNECTION

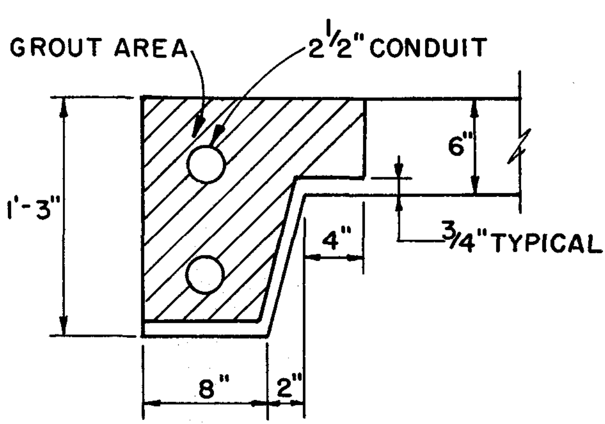


SECTION B - TYPICAL

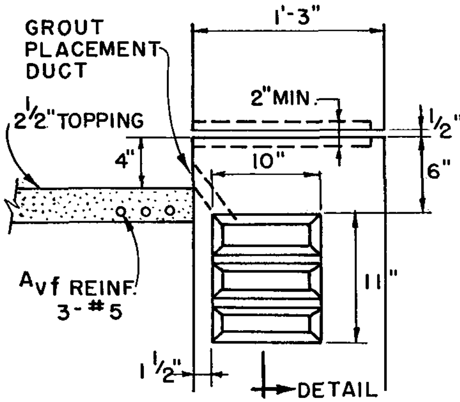
TYPICAL PANEL CONNECTIONS SHEET 3/5 C4

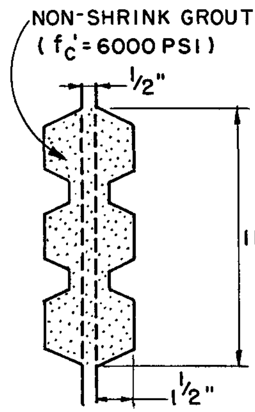


SECTION C-CENTER MULLION



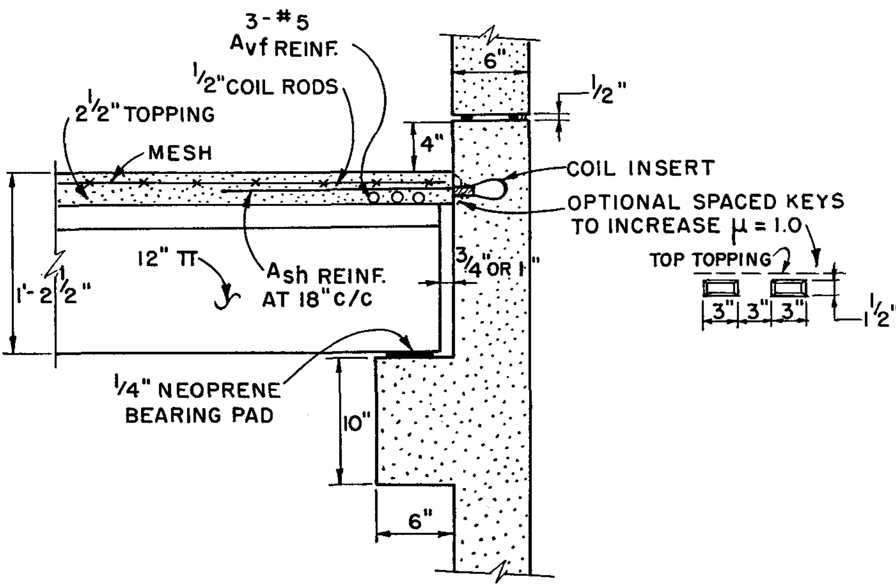
SECTION D-SIDE MULLION





SECTION E-TYPICAL VERTICAL JOINT SHEAR CONNECTION

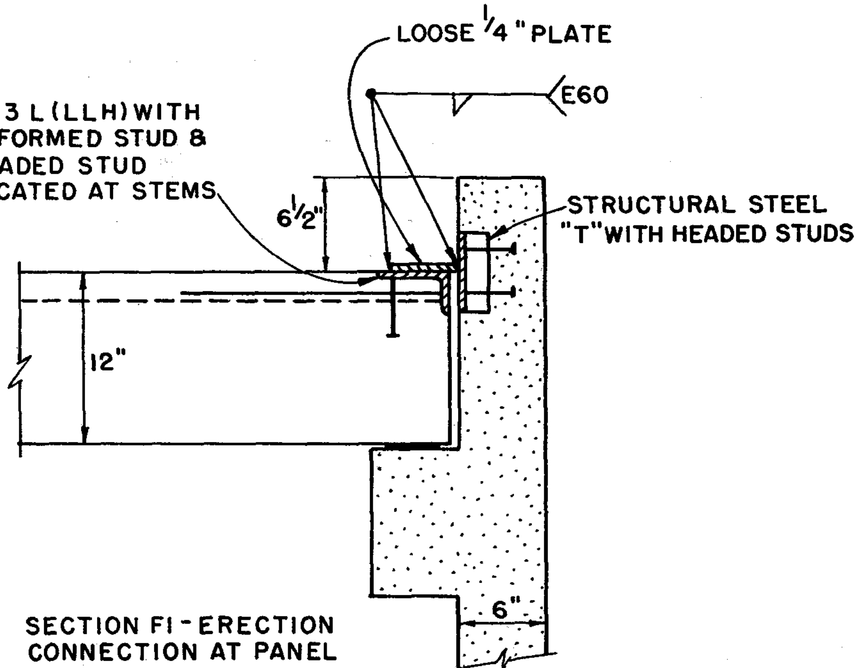
DETAIL



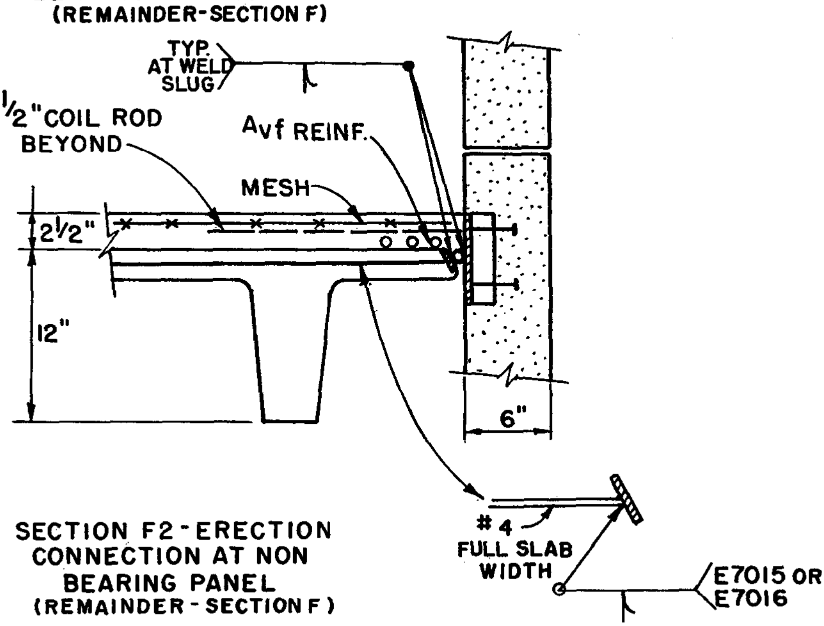
TYPICAL PANEL BEARING CONNECTION

SEE ALSO SECTIONS F1 AND F2

TYPICAL PANEL CONNECTIONS SHEET 5/5 C4



SECTION F1 - ERECTION CONNECTION AT PANEL BEARING CONNECTION (REMAINDER - SECTION F)



SECTION F2 - ERECTION CONNECTION AT NON BEARING PANEL (REMAINDER - SECTION F)