Architectural precast concrete panel systems used for lateral-force resistance

J. Paul Hobelmann, Macarena Schachter, and Matthew C. Cooper

- This paper presents a method of using a building's enclosure system of precast concrete panels as the lateral-force-resisting system for a steel framed structure.
- While the design, fabrication, and installation of the enclosure system are more complex and expensive, the costs are generally less than the savings in the structural frame costs.

Exterior enclosure systems for buildings often consist of elements that are supported by the building structure. For high-rise buildings, support of the exterior enclosure by the building structural frame is necessary, but for low-rise buildings, self-support of the exterior enclosure may be possible and could result in cost savings in the structural frame due to the reduced demand on its members. For self-supported systems, the structural frame typically provides lateral support at each floor and the foundation system supports the enclosure gravity loads directly.

For both frame-supported and self-supported enclosure systems, deformation compatibility between the enclosure system and the structural frame must be considered to prevent performance problems or failure of the exterior enclosure system. For self-supported enclosure systems, deformation compatibility presents a unique challenge because in-plane deformation of the cladding system is typically much different from that of the structural frame. Where enclosure systems are self-supported, it may be possible to use their in-plane strength to offer force resistance to the structural frame and eliminate the problem of deformation compatibility. These systems are referred to here as self-supporting lateral-frame-resistant enclosure (SSLFRE) systems.

Precast concrete cladding systems are good candidates for lateral force resistance where appropriate panel arrangements are available.

Design considerations

Where enclosure systems can be self-supported and can serve as the lateral-force-resisting system for the building (an SSLFRE system), significant savings can be realized. For one specific project, implementing an SSLFRE system increased precast concrete costs about $900,000, while the savings in structural steel were estimated at about $2,500,000, a net savings to the client of $1,600,000.

System suitability considerations

Not all projects are suitable for SSLFRE systems. High-rise buildings are not suitable for SSLFRE systems because of material limitations related to the height and weight of the enclosure. Precast concrete SSLFRE systems are typically 10 stories or fewer, though higher systems are conceivable.

For precast concrete SSLFRE systems, a good panel configuration is a story-tall panel with punched windows. Panels with this configuration have vertical-load-carrying elements at the jambs of the windows and horizontal members, which can be rigidly connected to the jamb members, and can provide the lateral resistance for horizontal loads. Panels in a U-shape or an inverted U-shape can also be suitable. Strip or ribbon window enclosure systems, as well as strip panel configurations with column cover panel systems, consist of horizontally spanning elements that are not capable of carrying any vertical load. Thus they are not self-supported and are not suitable for SSLFRE systems.

It is best to have redundancy in the SSLFRE system. Typically, all or nearly all of the precast concrete enclosure is part of the SSLFRE system. This way stresses in individual components are low and the system is not compromised if one or two panels are damaged. Also, it is best if most of the panels are similar to provide uniform force distribution and improve the redundancy performance. If an SSLFRE system is proposed, the architect and the owner or user should be aware and in agreement because changes during the design or in the future may be restricted.

Also, if a panel needs to be removed at some point in an SSLFRE system, the removal is more difficult than for systems in which panels are individually connected to the structural frame.

Structural design considerations

For gravity design of precast concrete SSLFRE systems, the weight of the system will be carried by the vertical elements within the panels; therefore, the vertical elements should align through to the foundation. Typically, the bearing between panels will occur only at these vertical elements by use of high-density plastic or steel shims. It is efficient to keep vertical loads off of the horizontal panel members to avoid the resulting flexural stresses. For this reason, the horizontal joints between adjacent vertical elements should not be grouted such that vertical loads are transferred to the horizontal members.

Foundation design for SSLFRE systems should take into consideration that the support points for the precast concrete may not be known until the precast concrete subcontractor is retained. For the best precast concrete panel performance, uniform bearing at the bottom of the precast concrete is not desirable. To complete the design documents for the foundations, it should be assumed that the precast concrete will impart concentrated point loads below each of the continuous vertical elements. It is possible that the panels will be supported at each end only, even if there are more vertical elements within each panel. To proceed, foundation design for both of these conditions could be considered (that is, support below each vertical within the panel or support at each end of the panel only). Also, it may be possible to refine the foundation design after the precast concrete subcontractor is retained, if acceptable to the project team.

Where the precast concrete enclosure is self-supported but does not serve as part of the lateral-force-resisting system, deformation compatibility between the structural frame and the precast concrete in the plane of the wall becomes a challenge. Self-supported precast concrete cladding systems that also serve as the lateral-force-resisting system will match the structural frame deformations, and compatibility will not be a concern.

When designing a precast concrete SSLFRE system, it is desirable to avoid or limit structural connections between adjacent horizontal panels (at vertical joints). If enough panels are used for the lateral-force-resisting system, design stresses will be limited such that the vertical stacks of panels can act independently. This independence significantly reduces temperature performance effects, compared with connecting all of the panels together, and simplifies the design of the panels for vertical and lateral loads. If adjacent panel connections are necessary to achieve appropriate stress limits in the panels, one should consider connecting only two or three vertical stacks so that temperature deformations have regular relief along the length of the wall. Stresses should be limited to avoid cracking of the precast concrete under normal load conditions. Depending on the project, cracking under seismic conditions may be acceptable.

For temperature performance reasons, it is suggested that each vertical stack (or stacks, if connected together laterally) should possess only one lateral shear force connec-

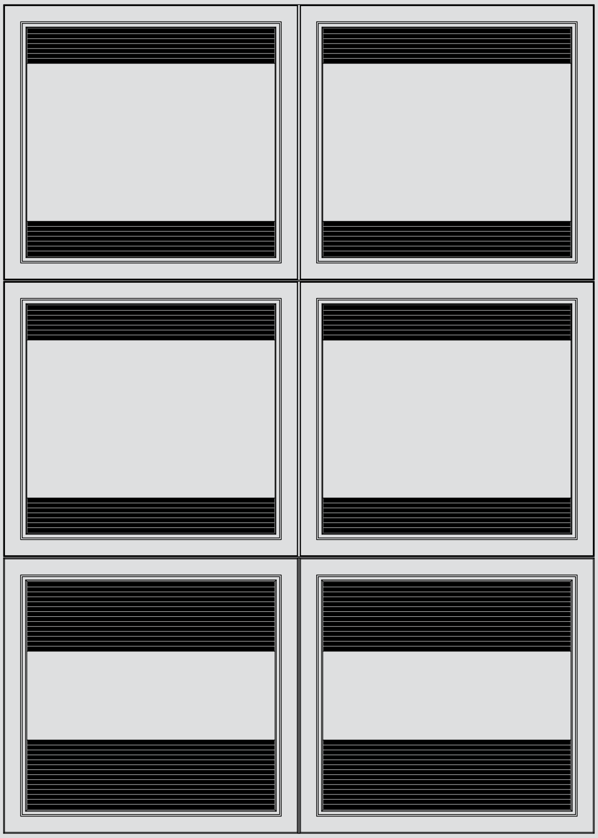

Figure 1. Architectural schematic of one bay of typical precast concrete panels.

tion to the structure at each floor. If more connections are provided, the thermal stresses between these connections should be considered because they can be overwhelming.

Although only one shear connection is provided at each floor, multiple connections for each panel are necessary at each floor to resist out-of-plane lateral forces and ensure stability. These additional connections should not resist vertical loads or in-plane lateral loads but should only provide lateral out-of-plane resistance.

For seismic design, American Society of Civil Engineers' (ASCE's) *Minimum Design Load for Buildings and Other Structures (ASCE/SEI 7-02)*[1] Table 12.2-1 includes values for response modification factor R, overstrength factor Ω, and deflection amplification factor C_d for ordinary and intermediate precast concrete shear walls. Ordinary precast concrete wall systems are allowed only for buildings in seismic design category B. Intermediate precast concrete wall systems are allowed without restrictions for buildings in seismic design categories B and C and in buildings up to 40 ft (12 m) in height in seismic design categories D, E, and F.

Consequently, ordinary precast concrete shear walls do not have to comply with seismic specifications of the American Concrete Institute's (ACI's) *Building Code Requirements for Structural Concrete (ACI 318-05) and Commentary (ACI 318R-05)*[2] chapter 21, while intermediate precast concrete shear walls, having higher ductility values, are required to comply with the seismic specifications. ACI 318-05 chapter 21 also includes the definition and additional requirements for special precast concrete structural walls; however, this system is not listed in ASCE/SEI 7-02 Table 12.2-1.

The difference in design between an ordinary and an intermediate precast concrete wall system is the design of the connections. For intermediate precast concrete wall systems, the connection should be designed to ensure ductile behavior of the system (that is, yielding of the wall reinforcement prior to fracture of the connections).

ASCE/SEI 7-02 recognizes that a different performance is expected if the lateral system is a bearing wall (that is, one supporting additional gravity load from the structure) or if it is a frame system. More ductile behavior is achieved with a frame system because the axial force on vertical members is reduced. An SSLFRE precast concrete wall system designed as a frame requires a backup frame (steel or concrete) that will carry the gravity loads. In a frame system, because of the reduced gravity forces, it is likely that the system will experience net uplift under lateral loads. Vertical members should be designed and properly anchored to the foundation. If intermediate precast concrete walls are used, appropriate seismic detailing should be implemented. The seismic demand will usually exceed the tie force requirements of ACI 318-05 chapter 16.

Documentation considerations

On most projects, the details for the enclosure system are typically indicated on the architectural drawings. For SSLFRE systems, some details should be indicated on the structural drawings. It is important that the contractor be aware that the enclosure system is self-supporting and serves as the lateral-force-resisting system for the structure.

Because the design of enclosure systems is typically performance based, the criteria specific to the SSLFRE system should be clearly identified on the contract documents. These criteria include the following:

- each SSLFRE panel type and location
- panel-to-structure shear connection locations for each panel and the forces for which these connections should be designed
- panel-to-panel shear connection locations and the forces for which these connections should be designed
- foundation support design assumptions
- forces (or alternately the reinforcement) at the panel joints

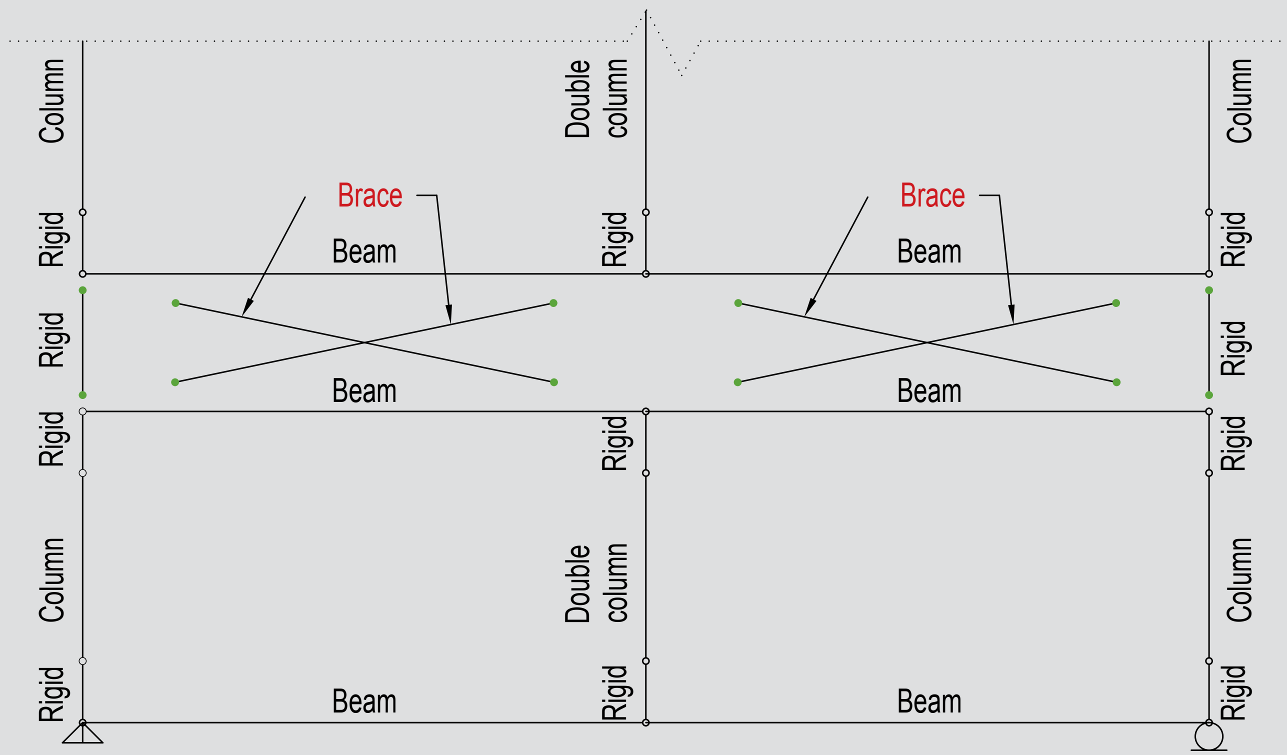

Figure 2. Detail of the structural analysis model showing supports for the panel-to-panel connections.

Example analysis and design

The following description represents a recently completed project that used a precast concrete SSLFRE system. The exterior enclosure for this multibuilding project consisted of precast concrete panels with punched windows. The buildings ranged from four to six stories tall and had considerable repetition in the exterior enclosure. For confidentiality purposes, the location and name of the project is withheld at the request of the owner.

General description of the panels

The precast concrete panels were typically 30 ft wide × 14 ft tall (9.1 m × 4.3 m). The windows were about 13 ft wide × 8 ft tall (4 m × 2.4 m) with two windows in each panel. The panel profile reflects the architectural requirements (**Fig. 1**).

Structurally, the panel was considered as a weak pier-strong spandrel type of wall, with three columns and two beams. The edge columns were about 12 in. × 12 in. (300 mm × 300 mm), while the center column was about 24 in. (610 mm) wide × 12 in. thick. The spandrel beams were 8½ in. (220 mm) thick with ½-in.-deep (13 mm) reveals and a border element 12 in. deep and 12 in. thick. Structurally, the spandrels were modeled with a constant cross-sectional thickness of 8 in. (200 mm). The design compressive strength of the concrete was 5000 psi (35 MPa).

The panels spanned one bay horizontally (between two building columns) and were one story tall. For the panels to perform as a lateral-force-resisting system, the internal panel forces were transferred vertically between the stacked panels, and each panel was connected to the structure so that diaphragm forces were transferred to the panels. Therefore, the panel-to-panel and panel-to-structure connections were critical. Because the panels did not carry gravity load from the structure, uplift forces were present.

The performance of the panels under service wind loads was important. Because the panels served as the facade of the building, cracking of the precast concrete under service loads was not allowed.

In addition to the typical panels (Fig. 1), there were solid panels without windows, panels with louvers, and panels with doors. Panels that were not suitable for use as SSLFRE elements were hung from the structural frame.

The shear wall panels were self-supported and designed to stack on top of each other. Only two supports were considered, one at each end of the panel. The tension or compression force at the panel piers resulted from the combination of the weight of the panels and the vertical seismic or wind loads.

High-density plastic shims placed between panels transferred the compression load to the panel below, while a

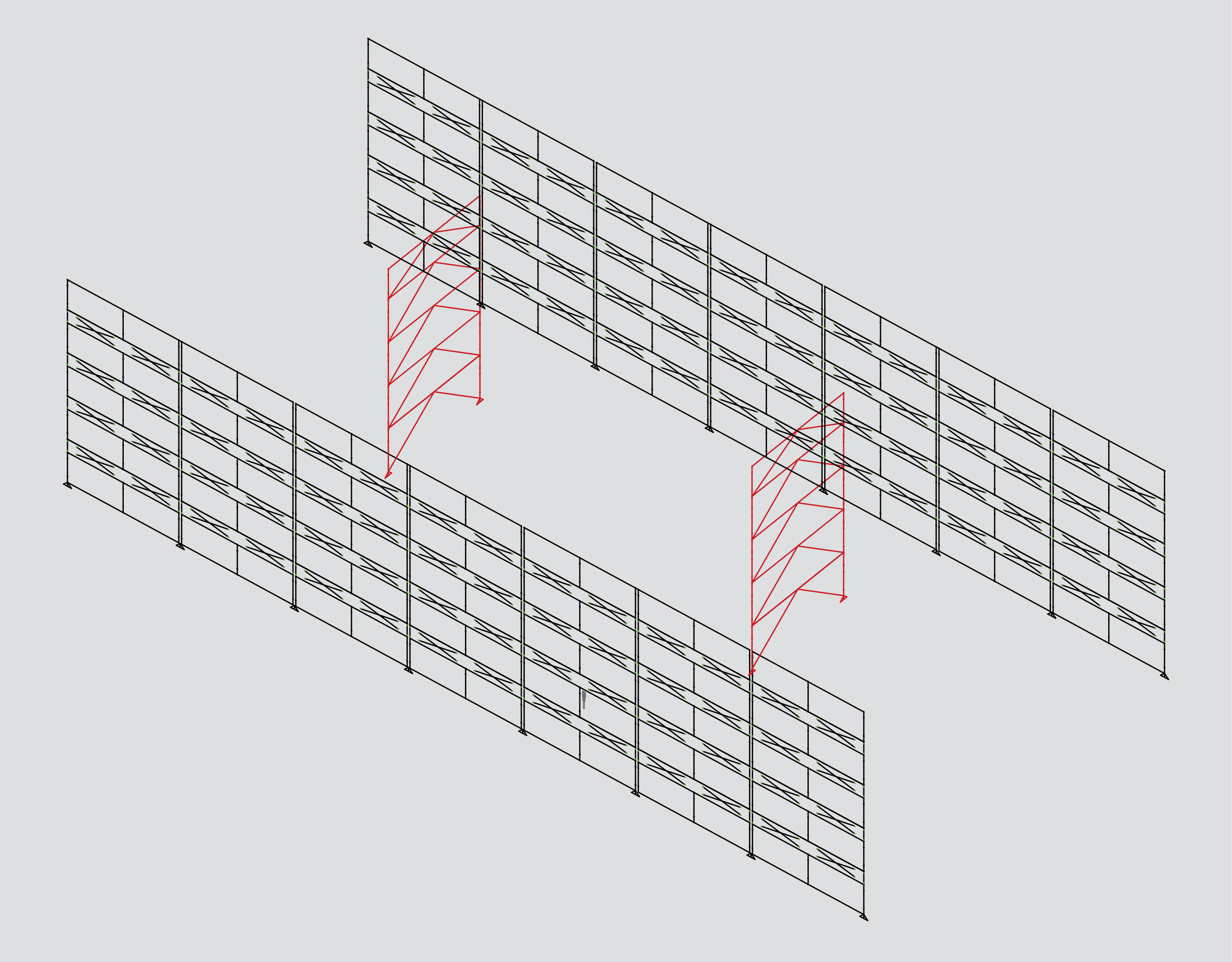

Figure 3. Three-dimensional view of numerical model of one of the buildings. Precast concrete panels are shown at the perimeter and internal steel K-braces are used for lateral resistance in the perpendicular axis.

vertical-shear connection transferred the tension load. The numerical analysis model reflected the lack of moment transfer between panels at the mechanical connection.

Just one connection at one end of each panel transferred the shear force from the diaphragm to the panel (the panel-to-structure connection).

This configuration allowed the panels to expand under temperature increases without being constrained by the structure. This single panel-to-structure shear connection resulted in a concentration of stresses in the diaphragm at the connection location.

Seismic properties of precast concrete shear walls

The project was designed in accordance with *International Building Code 2003*[3] and Department of Defense's *Seismic Design for Buildings* (Unified Facilities Criteria [UFC] 3-310-04).[4]

IBC 2003 does not specify seismic design coefficients and factors for a lateral-force-resisting system based on precast concrete shear walls. However, UFC 3-310-04 lists a response modification factor R of 4, an overstrength factor Ω of 2.5, and a deflection amplification factor C_d of 4 for ordinary precast concrete shear walls in the *building frame system* category. An ordinary precast concrete shear wall in the *bearing wall systems* category has the following design coefficients: R equal to 3, Ω equal to 2.5, and C_d equal to 3. The provisions of UFC 3-310-04 are shared by ASCE/SEI 7-02.

The project is located in an area with a seismic design category of B. The buildings are four to six stories high, and the total seismic base shear in the applicable direction is about 4% of the weight of the building.

The buildings have, in general, a rectangular shape with dimensions of about 120 ft wide × about 210 ft long (36 m × 64 m) (seven bays at 30 ft [9 m]). Precast concrete shear walls were used along both longitudinal sides of each building except at loading docks and entrances.

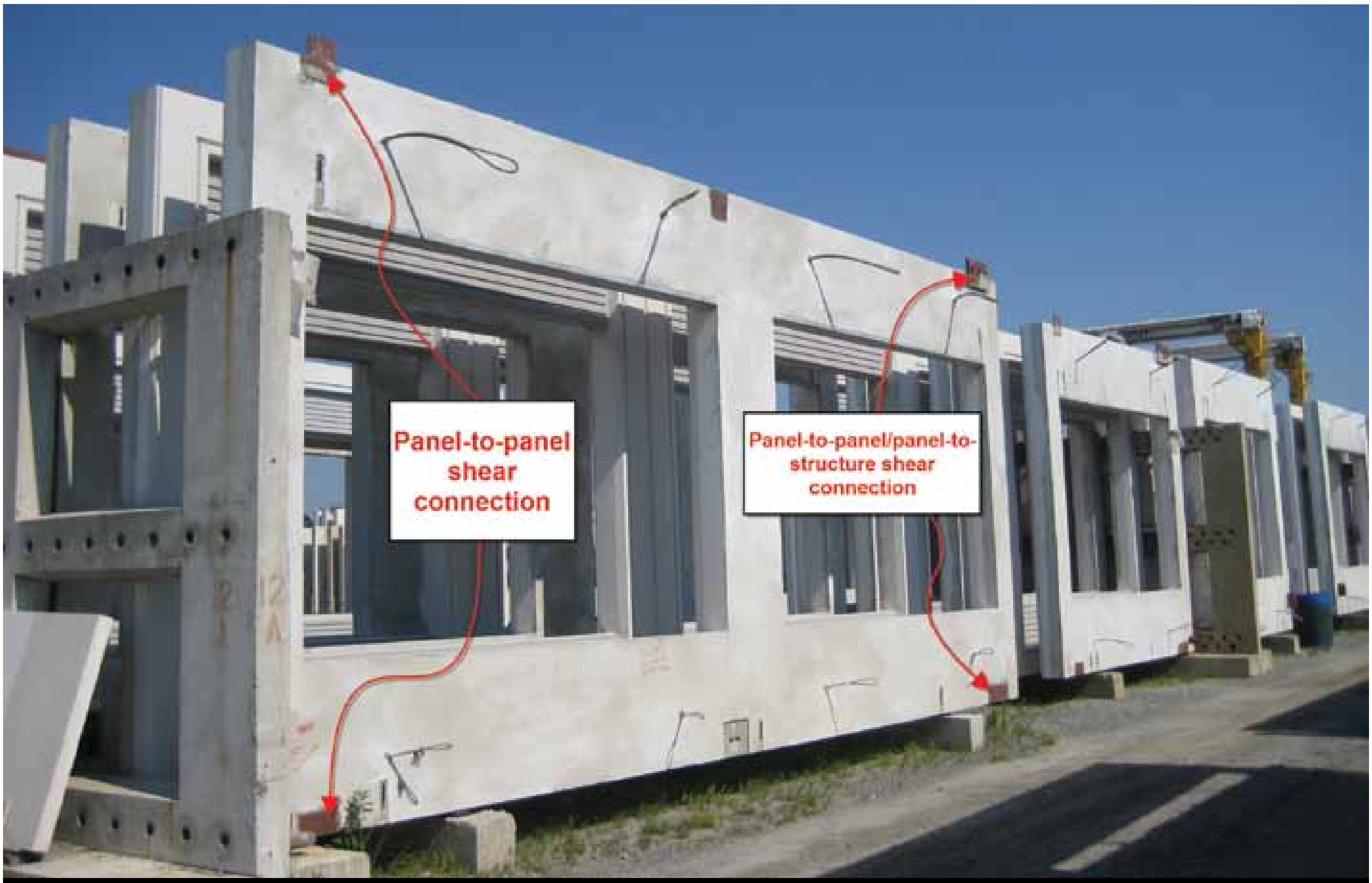

Figure 4. Rear elevation of a typical panel in storage. Figure 5 shows the connection details.

Description of numerical model

The precast concrete pier-spandrel panel system was modeled in a structural analysis program with frame elements.

Minimum reinforcement for columns and deep beams was required for these elements. The frame elements were modeled at their center of gravity. Rigid pinned elements connected two panels at the end columns, and rigid pinned braces were added for lateral stability. Shear and tension or compression forces could then be transferred with this configuration while moments would not (**Fig. 2**).

The panel-to-structure shear connections were located at the bottom of each panel except at the panel that spans between the fifth floor and the roof, which was connected to both the fifth floor and the roof. In the numerical model, these connections were modeled by applying lateral loads at each of these connection points. When the full structure was modeled, all of the nodes were connected by a rigid diaphragm such that the load was transferred to the panels according to their relative stiffness. **Figure 3** shows a three-dimensional view of the numerical model of one of the buildings.

For service wind analysis, the uncracked properties of the sections were used, but for the seismic design of the panels, the cracked section properties were assumed.

Results from numerical model

The models were first subjected to the lateral forces from service wind load (designed wind velocity of 90 mph [150 kph]) combined with the self-weight of the panels (about 1 kip/ft [15 kN/m]). All of the panels were subjected to a combination of axial compression and moment. Tension forces were not present under wind loads alone. Cracking was limited by the following equation:

$$f_r = 7.5\sqrt{f_c'} = 7.5\sqrt{5000} = 530 \text{ psi (3660 kPa)}$$

ACI 318-05 Eq. (9-10)

where

f_c' = compressive concrete strength

f_r = cracking stress of concrete

Analysis of all buildings showed that the stress in most panels was below this limit. In a few cases the stress slightly exceeded this limit, which was considered acceptable.

Global values for the design of the various connections were obtained. The upper two floors always remained in compression. Thus, it was decided that the upper two panels would not require panel-to-panel tension connections. For the bottom levels, however, tension in the panels varied

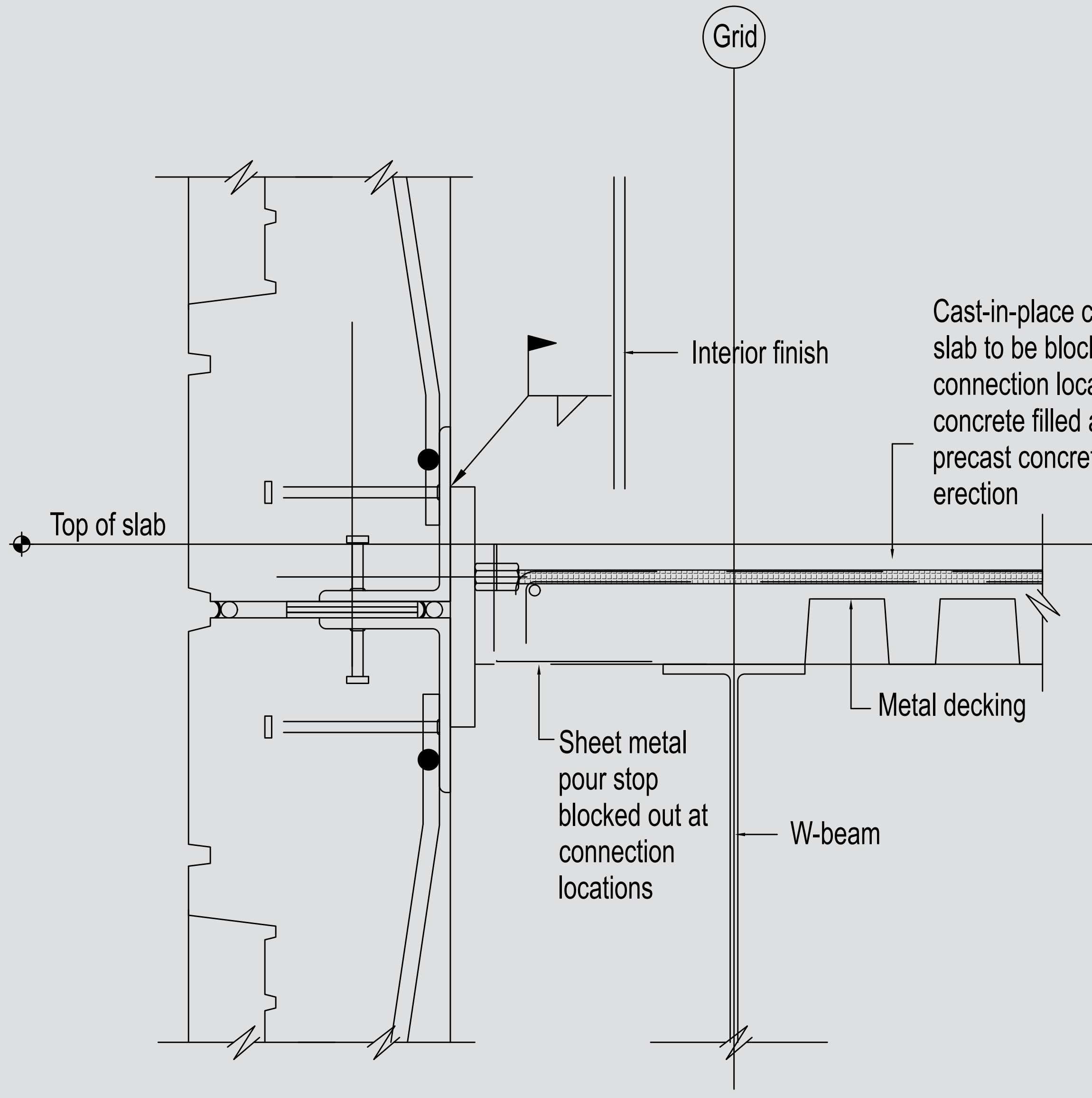

Figure 5. Section depicting the in-plane horizontal-shear-resisting connection.

with the height and ranged from 25 kip to 40 kip (110 kN to 180 kN).

The horizontal shear connection between each panel, as well as at the base, was established for each building. For the upper two floors, shear values ranged from 20 kip to 35 kip (90 kN to 160 kN). For the lower floors, maximum base shear values ranged from 45 kip to 75 kip (200 kN to 330 kN).

Finally, the panel reinforcement was checked. Minimum axial and shear reinforcement were determined to be adequate to resist the imposed loads. Provisions for minimum reinforcement of beams and columns rather than walls were used at panels with punched windows, while provisions for minimum reinforcement of walls—including boundary elements—were used at panels with no windows. The required reinforcement was specified on the structural drawings.

Although each panel behaves like a strong beam–weak column moment frame, the global system (collection of stacked panels) does not. There is no moment continuity at the columns along the height of the structure or at the beams along the length of the building. The overturning moment from seismic and wind demand is resisted through axial forces only at the ends of each panel, like boundary elements in shear wall systems.

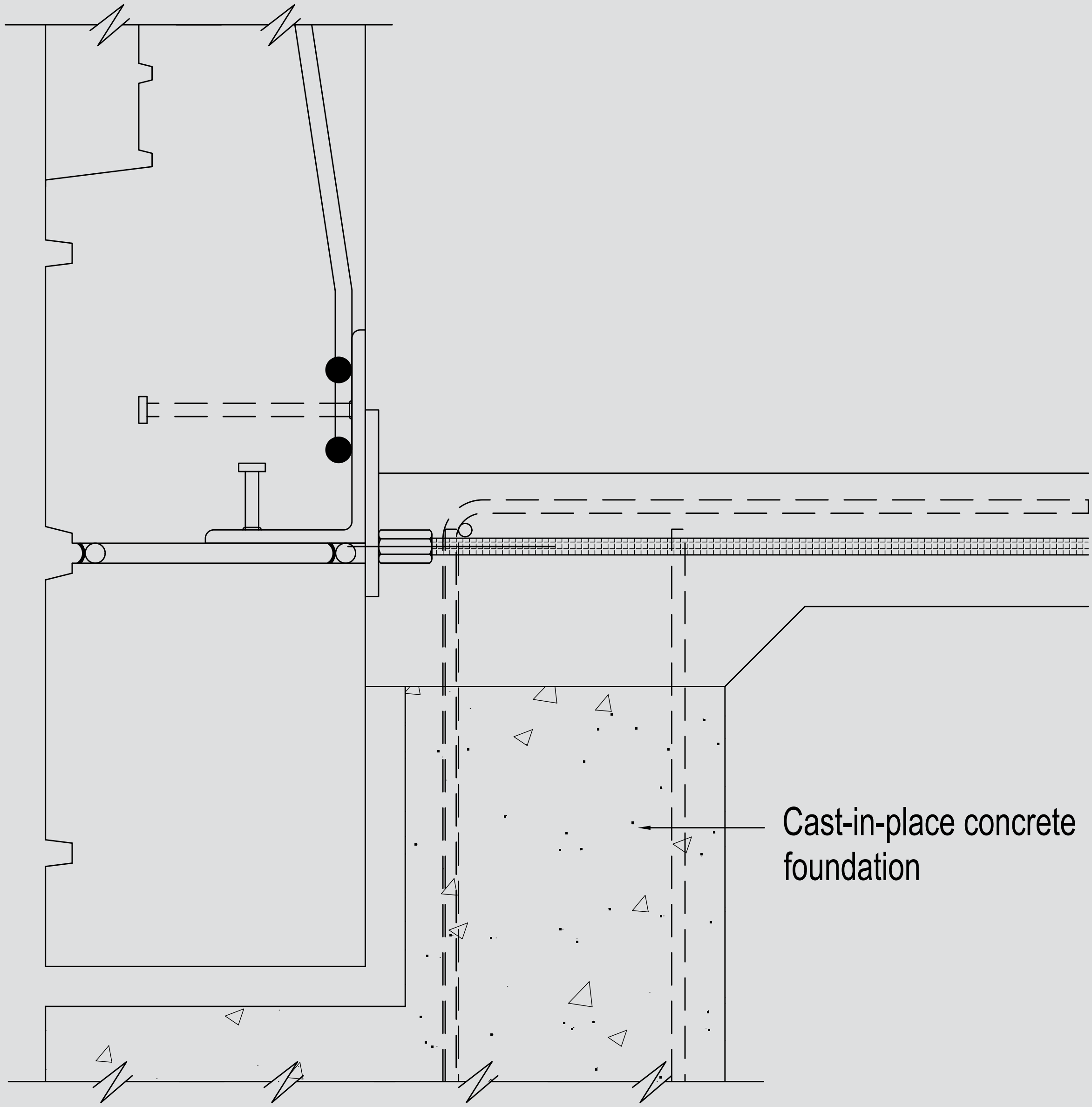

Figure 6. Panel-to-structure shear connection at base.

Documentation considerations

The analyses' results and connection designs were incorporated into the design documents in a simple way. The different panel types were identified on the plans. Elevations for each of these panels were provided on the structural drawings. Sections, location of connections, and main openings were also shown. **Figure 4** shows the elevation of the typical panels.

Connection details, reinforcement, and tables with connection design values were indicated on the structural drawings. During the shop drawing process, the precast concrete fabricator made adjustments to the connections to attain efficiencies. These adjustments included changes to anchorages and plate thicknesses. The subcontractor also modified the precast concrete–to–structure connection details and combined some of the connections to limit the overall number of connections to the panels.

Connection design

After obtaining the shear load values for connection design from the structural analysis program, connections were designed. The cast-in-place concrete topping and edge deck angle were blocked out to install the connection (consisting of threaded reinforcing bars fastened into the precast concrete embedment) and the composite slab was filled after precast concrete erection. These blockouts were required to be large enough to ensure that reinforcing in

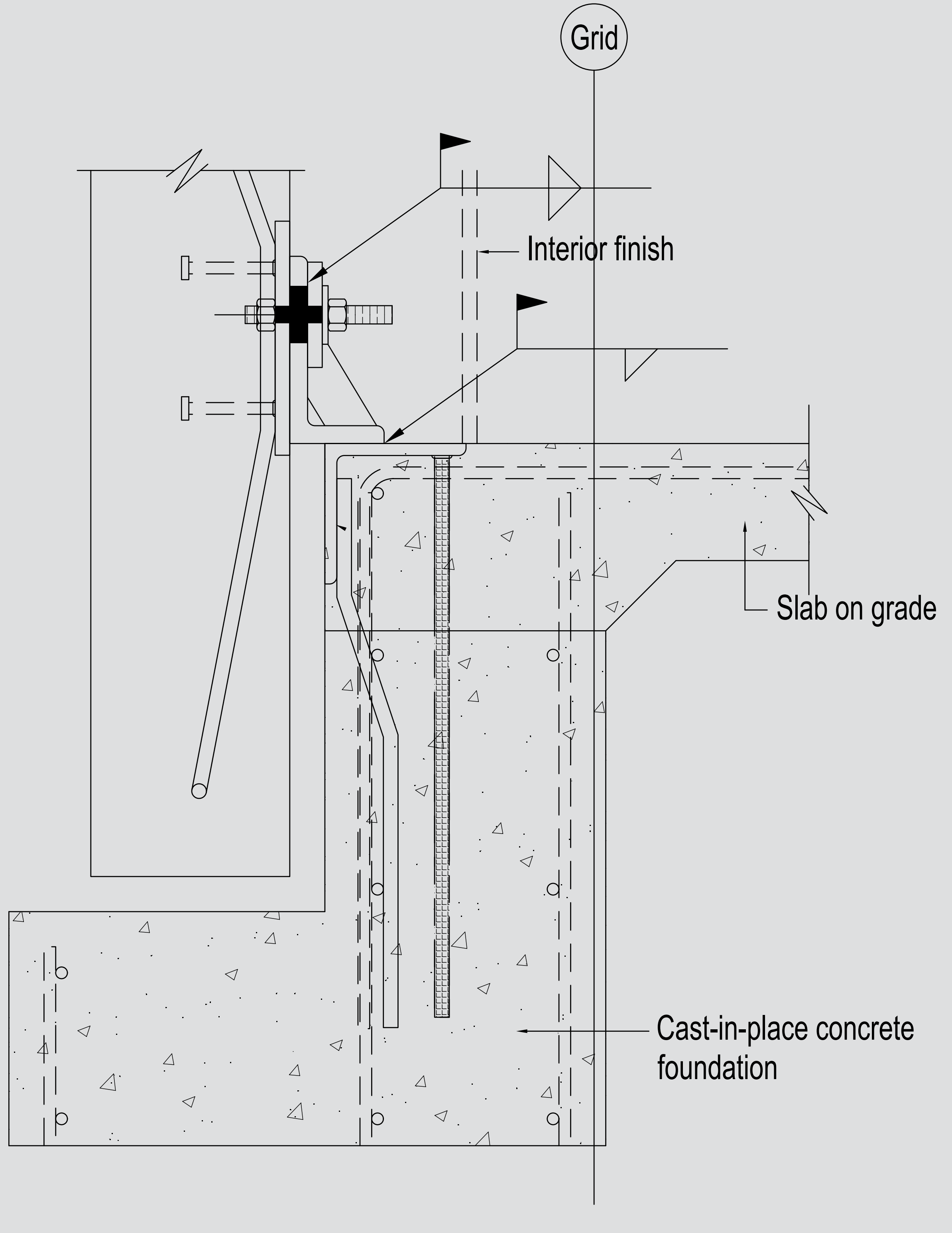

Figure 7. Panel-to-structure tension connection at base.

the cast-in-place concrete topping could lap sufficiently. **Figure 5** shows the typical structure-to-panel horizontal-shear and panel-to-panel vertical-shear (tension) connection design.

Shear connection

The shear connection to resist horizontal in-plane loading at the base was designed for the maximum base shear of 75 kip (330 kN). Analogous to the panel-to-structure shear connection, the slab was required to be blocked out. **Figure 6** shows this connection concept and a photograph of connection prior to fill.

The tension connection from the panel to the foundation was designed for uplift. At one end, allowing horizontal movement was imperative because of thermal expansion and contraction of the precast concrete panels relative to the structure. To accommodate this, an angle was installed between a cast-in-place concrete embed and the precast concrete embed. This angle contained oversized holes on the vertical leg for fabrication and erection tolerances. The angle was welded to the cast-in-place concrete embed and was bolted into the precast concrete embed using plate washers with slots to allow for in-plane horizontal movement. These plate washers were welded to the angle to transfer vertical shear from the threaded rods into the angle. **Figure 7** shows this connection schematic.

Construction considerations

When using an SSLFRE system, a few points should be considered during construction. Because the SSLFRE system only provides lateral-force resistance for the structure in the completed state, temporary bracing requirements for the structural frame may be increased. Temporary bracing will be required until completion of the exterior enclosure installation.

The provision of an SSLFRE system may also have an effect on the concrete placement details of a project. For the previously discussed project, floor slab blockouts were provided at the slab perimeter to allow for the installation of the embedded shear connections after placement of the floor slabs. The placement of the slabs preceded the installation of the precast concrete connections such that the blockouts were required. The blockouts required the cast-in-place concrete subcontractor to remobilize after precast concrete installation to complete the slabs but reduced the amount of field welding required during precast concrete installation.

Acknowledgments

The authors acknowledge Quanterra Structural Associates for their input.

References

1. American Society of Civil Engineers (ASCE). 2002. *Minimum Design Load for Buildings and Other Structures (ASCE/SEI 7-02)*. Reston, VA: ASCE.

2. American Concrete Institute (ACI) Committee 318. 2005. *Building Code Requirements for Structural Concrete (ACI 318-05) and Commentary (ACI 318R-05)*. Farmington Hills, MI: ACI.

3. International Code Council. 2003. *International Building Code 2003*. Whittier, CA: International Conference of Building Officials.

4. Department of Defense (DOD). 2007. *Seismic Design for Buildings*. Unified Facilities Criteria (UFC) 3-310-04. Washington, DC: DOD.

Notation

C_d = deflection amplification factor

f'_c = compressive concrete strength

f_r = cracking stress of concrete

R = response modification factor

Ω = overstrength factor

About the authors

J. Paul Hobelmann, P.E., S.E., is an associate principal with Weidlinger Associates Inc. in Washington, D.C.

Macarena Schachter, PhD, P.E., is a structural engineer with Weidlinger Associates Inc. in Washington, D.C.

Matthew C. Cooper, P.E., is an engineering manager with the Shockey Precast Group in Winchester, Va.

Abstract

This paper presents an innovative method of using a building's enclosure system of precast concrete panels as the lateral-force-resisting system for a steel framed structure. Compared with traditional methods of lateral force resistance systems, the savings in structural steel resulting from the use of the enclosure system as the lateral-force-resisting system can be significant. While the design, fabrication, and installation of the enclosure system is more complex when used as the building's lateral-force-resisting system, which results in additional precast concrete costs, the costs are generally less than the subsequent savings in the structural frame costs.

Keywords

Architectural, connection, seismic, shear, wind.

Review policy

This paper was reviewed in accordance with the Precast/Prestressed Concrete Institute's peer-review process.

Reader comments

Please address any reader comments to journal@pci.org or Precast/Prestressed Concrete Institute, c/o *PCI Journal*, 200 W. Adams St., Suite 2100, Chicago, IL 60606.