34 Stories of Stacked, Load-Bearing Panels
Still Unique after 10 Years

Tower of America

John A. Tinker

Palanisami & Associates of Minneapolis, Minn., was fortunate to be a part of the design and construction of an apartment tower in 1995 that was sure to be an engineering milestone. The New York, N.Y.–based LeFrak Organization, the building's developer, was in the middle of a $10 billion mixed-use, multibuilding development on the Hudson River waterfront in Jersey City, N.J., called Newport, when it chose to undertake a new kind of building. Tower of America West (TOA), as this building is known, would have 346 rental units and an attached parking structure and would be set among many similar apartment towers proposed as part of the development. However, the uniqueness of this tower would be more than skin deep.

When planning for the tower began, many other towers constructed of the more commonly used systems of precast concrete cladding with structural steel or cast-in-place concrete framing were either proposed or under construction in this development. However, in this project, the developer was interested in attempting a completely precast concrete system. From the foundation up, precast concrete would be used, ultimately comprising the interior load-bearing walls, shear walls, hollow-core floor slabs, and exterior architectural wall panels. The only other component to the structural system would be a small number of full-height structural steel columns. These columns support steel beams, acting as a partial framing system to carry floor loads at the exterior walls and shortening the floor spans in critical areas.

At the time of construction, and perhaps even today, 34 stories of stacked load-bearing precast concrete wall panels without a supporting frame or core of steel or cast-in-place concrete was without precedent. Bearing-wall construction in high-rise buildings has been around for many years, but it has often employed vertical post-tensioning of wall panels in buildings with only 20 to 30 stories.[1] The difference in this building is its nonprestressed bearing-wall construction. This type of construction on such a large vertical scale—312 ft (95 m) from grade to the penthouse, comprising a gross floor area of 394,000 ft^2 (36,600 m^2)—presented many design challenges. Even though it was finished more than 10 years ago, it still represents the state of the art in precast concrete structural systems.

Above **The Tower of America West is beginning to be erected in Jersey City, N.J.** Photo courtesy of Tower of America Urban Renewal Co.

Previous page **This photo of the Tower of America West shows the south elevation along the marina.** Photo courtesy of Tower of America Urban Renewal Co.

Team members

Owner/developer:

Tower of America Urban Renewal Co., Rego Park, N.Y.

General contractor:

Tower of America NJ Construction Corp., Jersey City, N.J.

Engineer of record:

Churches Consulting Engineers, Washington, Pa.

Architect:

David L. Mammina, Rego Park

Precast concrete specialty engineer:

Palanisami & Associates, Minneapolis, Minn.

Structural precast concrete:

The United Precasting Corp., Buena, N.J. (now High Concrete Group)

Precast concrete choice and supplier

The LeFrak Organization is a large developer that has gained considerable experience in the nuances of different building methods in the projects constructed before TOA. The decision to use precast concrete as the primary building material in this tower was a complex one, but as Tony Scavo, the developer's vice president of construction, says, "The reason at that time was mostly economics." Erectors and producers had to be competitive with steel and cast-in-place concrete contractors in their bids for this project in order to make precast concrete attractive.

The unique site also contributed to the decision to use precast concrete. Access was only available from two directions due to the waterfront location, and the dense urban setting further constricted available site staging area. Because of this, a building technique that limited high-frequency ready-mixed deliveries, concrete pumping, on-site fabrication, storage of construction materials, and the number of field operations promised to make the erection process easier, faster, and safer. With precast concrete, a limited number of deliveries (averaging six trucks per day with two panels each) of large precast concrete panels could be made to the site instead. The site had a staging area large enough to accommodate 20 trailers holding about 40 panels for ready inventory at any given time. The panels could then be taken off the trailers as needed using a crawler crane and then lifted to the floor that was being erected all in one operation. Because the wall panels and the structure were one and the same, the building could be enclosed after each story was erected, allowing interior working conditions to be better controlled.

Another important factor in the selection of precast concrete for this project was the availability of precast concrete suppliers that could supply the large number of pieces that were required and keep up with the fast schedule proposed by the erector. This meant that there had to be companies willing and able to devote most, if not all, of their production capacity to one single building for a production period of four to six months. The supplier had to consider storage as well. If the precast concrete was produced too quickly, there would be an excess amount of inventory to store in the yard before it could be trucked to the site. If the precast concrete was produced too slowly, it could cost the owner money and time.

This load-bearing precast concrete structural system was proposed by the engineer of record (EOR), Churches Consulting Engineers, prior to having either a precast concrete supplier or specialty engineer on board. More than a few precasting companies were contacted before one was found that had confidence in the structural system that the EOR and developer were proposing.

"The precaster's engineers would walk when they saw the system we were proposing. They didn't think it would work," says Charles Churches, P.E., owner of Churches Consulting Engineers.

The search for a precasting company and a precast concrete consulting engineer who believed in this proposed system ended when the developer selected United Precasting Corp. of Buena, N.J. to provide the hollow-core floor slabs and interior wall panels. United had a strong relationship with P. Palanisami, P.E., structural engineer and president of Palanisami & Associates. Applying his past experience with precast concrete engineering, Palanisami said that this project could indeed be executed cost effectively. This is where the precast concrete specialty engineer's role in the project began. The developer selected a second company to fabricate the exterior architectural panels.

Design and construction

Many design and construction questions unique to this project were raised during design development. Among these issues were panel lifting and erection, coordination between the precasters and designers, panel volume change, and lateral design.

Panel lifting and erection

The erector's ability to lift and install large panels on the upper stories of a tall building is something that must be considered when designing with precast concrete. It was decided on TOA, as is common practice in the precasting industry, to make the panels as large as possible while allowing for road and handling restrictions.[1] Large panels decrease the amount of panel-to-panel connections required at the vertical joints, increase the stiffness of the shear-wall section by making it as monolithic as possible, and decrease the number of expensive crane lifts required.

The heaviest panels were 30 ft (9 m) long and 9 ft (2.7 m) tall, weighed between 20 tons and 25 tons (22 tonnes and 28 tonnes), and needed to be lifted as high as the 32nd floor. For flexibility, the erector began the project with a 230-ton (250 tonne) crawler crane with a 200 ft (61 m) boom and fixed jib. Once the erection reached above the height and load limits of the first crane, the erector switched to a larger crawler capable of a 300 ft (92 m) main boom height and fitted with a fixed jib reaching 40 ft (12 m) out. This worked well with the floor-plan dimensions and allowed most of the lifting to be done from the north and south sides of the building.

Component coordination

Because each floor of the building is made of panels from two separate precast concrete suppliers and steel from another supplier, fabrication schedules had to match closely. To minimize coordination problems, the instruction of the precasters had to be the job of a single engineering firm.

Palanisami & Associates was responsible for issuing fully coordinated sets of erection drawings—totaling over 60 sheets—and thousands of production drawings to each precast concrete supplier. Each set was drawn using each supplier's own drafting and production conventions in order to limit possible interpretation problems. The precast concrete specialty engineer was also in charge of coordinating the thousands of connections between cast-in-place foundations, architectural panels, structural panels, hollow-core floor slabs, and structural steel. All production and field questions were channeled through the precast concrete specialty engineer to ensure proper coordination and single-source information.

"The precaster's engineers would walk when they saw the system we were proposing. They didn't think it would work," says Charles Churches, P.E., owner of Churches Consulting Engineers.

Panel volume changes

An important consideration in high-rise, precast concrete design is the differential shortening that occurs between vertical panels with different characteristics and functions. This is a problem typically caused by interior and exterior temperature and humidity changes, unbalanced compressive loads in load-bearing panels, the age of the panels since casting, and concrete creep. Another consideration was that in this building, the amount of stress in the load-bearing wall panels was not balanced by alternating hollow-core slab direction from floor to floor as is commonly done in high-rise buildings.[2]

The potential expansion and contraction of the exterior panels were compared with the elastic shortening and creep values at the interior load-bearing walls. The *PCI Design Handbook: Precast and Prestressed Concrete* was used in these calculations with an annual temperature range of 50 °F (27 °C) and an annual average ambient relative humidity of 70%.[3]

First, it was determined that the exterior architectural wall panels would have the potential to expand and contract from temperature changes across a range of 0.42 in. (10.7 mm) when measured at the building's top story.

Second, it was found that a maximum shortening difference of 0.78 in. (19.8 mm) would occur at the 32nd story between the bearing wall on grid line 8 and the nonbearing elevator wall on grid line D, primarily due to the effects of creep and elastic shortening.

Third, adding the effects of temperature shrinkage, elastic shortening, and creep for the exterior panels and subtracting the elastic shortening and creep values for the nonbearing wall on grid line D, a difference of 1 in. (25 mm) was found. The difference in shortening between the concrete panels and the steel-column framing was also a critical consideration.

Potential fluctuations and differential movement between dissimilar precast concrete panels or between panels and steel framing could not entirely be avoided and had to be accommodated in the following ways. In order to minimize the potential for differing amounts of elastic shrinkage, it was determined that both precasters had to set up similar casting sched-

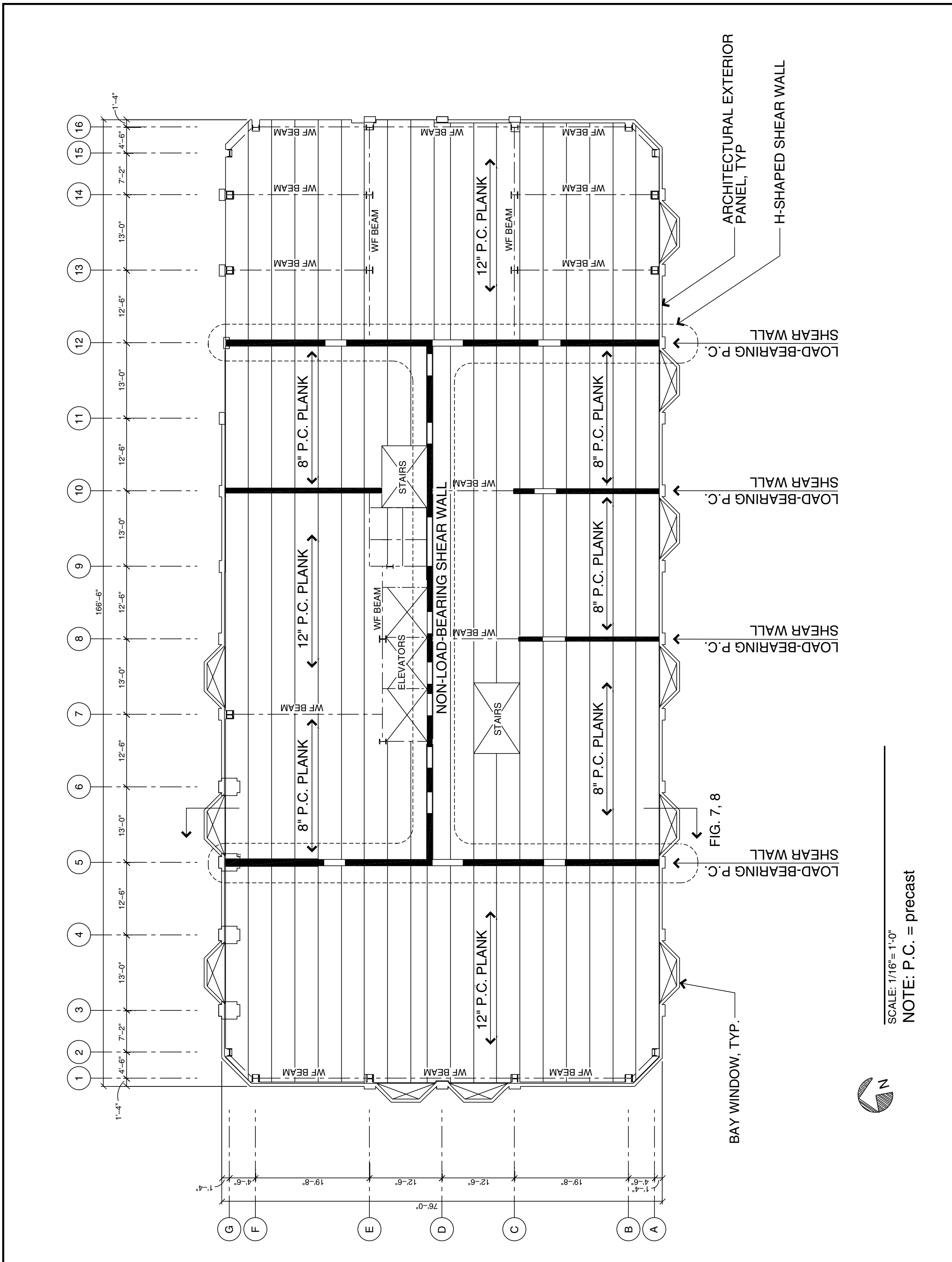
Summary of estimated elastic shortening and creep values

Material response	12 in. wall grid 5	12 in. wall grid 12	10 in. wall grid 10 from F to D and 10 in. wall grid 8	10 in. wall grid 10 C to A	Non-bearing wall grid D	Architectural precast concrete panels
Elastic shortening due to dead load, in.	0.361	0.253	0.439	0.285	0.193	0.394
Creep due to dead load, from two to six months, in.	0.303	0.212	0.421	0.342	0.231	0.324
Creep due to dead load, from six months to final, in.	0.911	0.637	0.801	0.650	0.440	0.658
Elastic shortening due to 20 lb/ft^2 superimposed dead load, in.	0.072	0.046	0.070	0.057	0.076	0.108
Creep due to 20 lb/ft^2 superimposed dead load, in.	0.091	0.115	0.160	0.130	0.175	0.183
Total creep, from six months to final, in.	1.002	0.752	0.961	0.780	0.615	0.841
Total creep, from two months to final, in.	1.305	0.964	1.382	1.122	0.846	1.165
Total shortening, from two months to final, in.	1.738	1.263	1.891	1.464	1.115	1.667

Note: All values are in decimal inches for the 291 ft building elevation (the 32nd story). For interior walls, relative humidity is assumed to be 70% from two to six months and 30% from six months to final. For architectural panels, relative humidity is assumed to be 70% from two months to final. Movement of architectural panels due to temperature change is 0.42 in. The fourth edition of the *PCI Design Handbook: Precast and Prestressed Concrete* was used to determine the values. Total elastic shortening of a typical W14 × 90 column at the 32nd story is 0.46 in.
1 in. = 25.4 mm. 1 lb/ft^2 = 992.88 kPa.

ules for the panels on each floor in order to match development of concrete strength and cement hydration. These potential differentials would be small for any given floor but could contribute to a higher cumulative differential, given the number of stories in this building.

As far as detailing the panel connections, the horizontal expansion and contraction of the architectural precast concrete wall panels were allowed by only connecting the tops and bottoms of each panel to the floor systems and the walls above and below. The ½ in. (13 mm) vertical-panel joints were connected with slotted plates bolted to the inside face and simply sealed from the elements. All of the connections, from the exterior wall panels to the interior load-bearing walls, steel beams, steel columns, and precast concrete hollow-core floor slabs, were designed as vertical slotted connections to allow movement of at least 1 in. (25 mm) up and down. Loose connection angles with long slotted holes and slotted inserts were used for this purpose.



The keyplan of a typical floor shows the load-bearing walls, H-shaped shear walls, and architectural walls. Note: 1" = 1 in. = 25.4 mm; 1' = 1 ft = 25.4 mm. Courtesy of Palanisami & Associates.

Lateral design

The design of the lateral support system was another issue. The floor plan of TOA is rectangular, measuring 76 ft × 166 ft (23 m × 50 m), giving it a 2:1 aspect ratio. This diaphragm aspect ratio was reasonable for the H-shaped shear-wall layout proposed, but the 1993 BOCA (Building Officials and Code Administrators International Inc.) building code gave main wind-resisting-system pressures up to 44 lb/ft^2 (2.1 kPa) at the top floor of the building due to its exposure D classification and 1.1 importance factor because it was sited in a hurricane-prone coastal environment. This, as well as the building's height, caused high tension and compression loads at the ends of the shear walls that would have to be transferred into the deep foundation system designed by Churches Consulting Engineers.

At the time that this building was designed, little information was available about lateral-load analysis of tall precast concrete structures. In the past 10 years, a lot of progress has been made in the treatment of jointed precast concrete structures as a single entity. However, in 1995, during the building's design, the prevailing belief was that in precast concrete construction, two shear walls in each direction were necessary to resist diaphragm torsion.[4] TOA did not meet this engineering rule. This is where the idea of joining separate precast concrete panels together to form a monolithic shear-wall section began.

The EOR believes that connecting all of the shear-wall panels together to form an H-shaped shear-wall assembly was one of the groundbreaking and most elegant elements of this design.

Shear-wall design

The interior H-shaped structural shear walls are made up of two load-bearing flange segments and a non-load-bearing web segment. These shear walls take the brunt of the lateral forces to the foundation. Both wind forces and seismic forces, as dictated by the building code, were considered in the design.

As was common in this region at that time, wind loads governed the design by a 2:1 ratio and resulted in a total base shear of 1365 kip (6071 kN) for each of the flange segments on grid lines 5 and 12.

A 1994 version of Images-3D finite-element software was used to model the shear-wall system to determine the distribution of individual story shears into the vertical resisting elements while accounting for torsional effects due to wind direction and accidental eccentricity. This software also aided in the analysis of the many shear-wall panels with blockouts for doorways. All of the plate model data were tabulated for each individual story and later used for individual panel analysis.

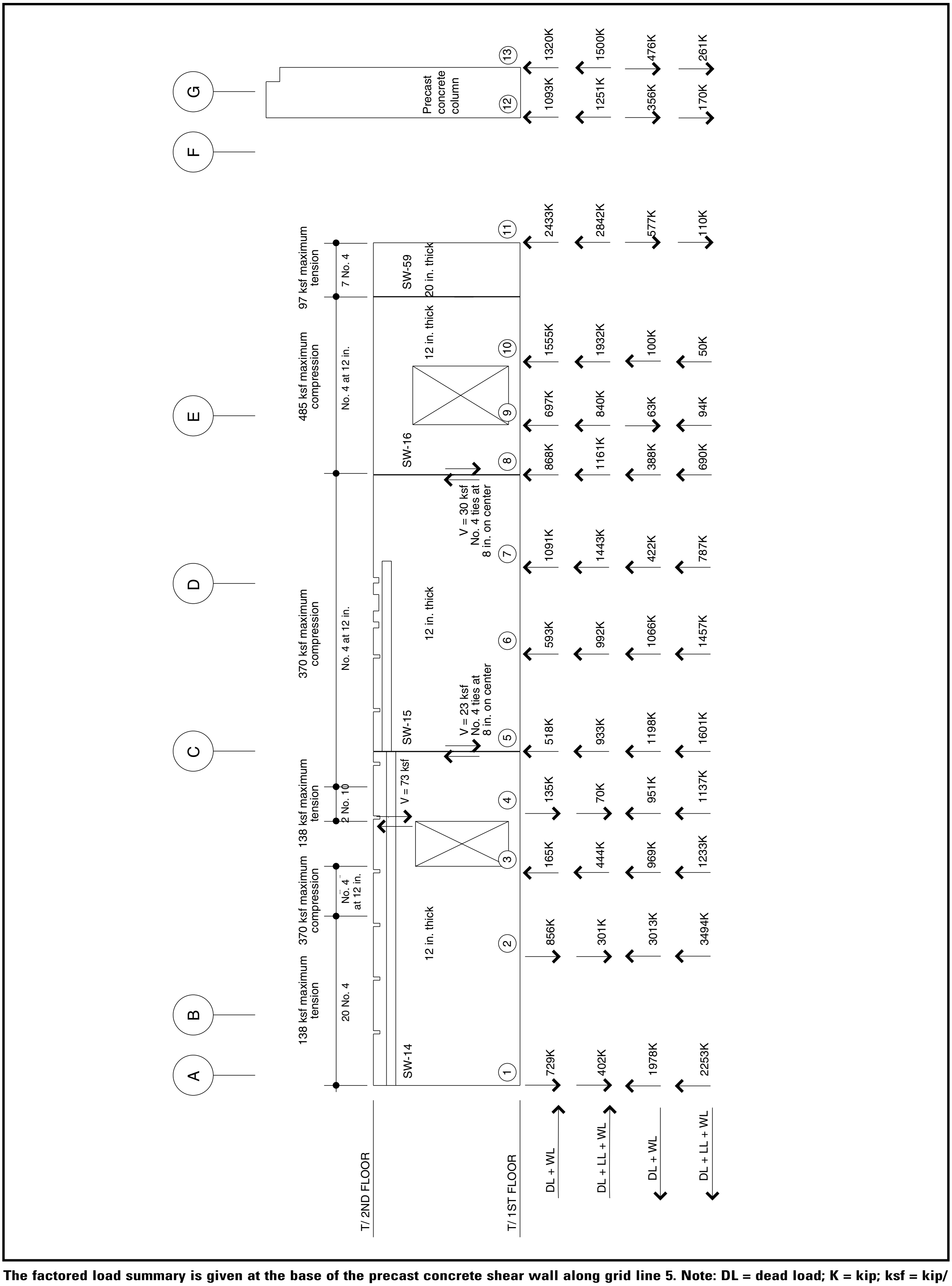
For a cost-effective design, the decision was made to use concrete with relatively modest 28-day compressive strengths of 6000 psi (41 MPa) for the wall panels from the fifth story down and 5000 psi (34 MPa)

Maintaining plumb a challenge

For a building this tall, the erector had to maintain a low tolerance for panel setting to minimize dimensional drift and to ensure that a given floor's panels would complete a closed rectangle when the final exterior panel was placed. Specifically, maintaining the correct elevations and plumbness between the precast concrete panels and steel framework proved to be a big challenge in this building.

Because development of a monitoring program for panel location was not given enough attention before the project began, there were drift problems between the precast concrete and the steel framework that were discovered after the building reached the 22nd story. Two to three stories of exterior wall panels had to be dismantled and reset to regain panel plumbness and alignment. This rework caused delays to the already-tight erection schedule.

One trick to ensure vertical panel alignment was varying the thickness of the wall panel shim stacks in the horizontal joints from floor to floor to make the panel elevations match exactly with the steel columns. When set perfectly plumb, the columns were used as a survey reference line for the adjoining precast concrete panels. The importance of checking and rechecking dimensions and closely monitoring tolerance build-up when erecting precast concrete structures of this height was demonstrated on this project.



The factored load summary is given at the base of the precast concrete shear wall along grid line 5. Note: DL = dead load; K = kip; ksf = kip/ft²; LL = live load; SW = shear wall; V = shear; WL = wind load. 1 in. = 25.4 mm; 1 ft = 0.305 m; 1 kip = 4.448 kN. Courtesy of Palanisami & Associates.

above. All of the precast concrete wall panels throughout the building were designed with mild-steel reinforcement in an effort to limit production cost as well as minimize the potential for further panel creep as a result of additional compressive stress caused by prestressing. All of the interior load-bearing walls were either 10 in. (250 mm) or 12 in. (300 mm) thick.

Panel-to-foundation connections

One area involving the shear walls that proved to be an engineering and construction challenge was the panel-to-foundation connections. These panels were set on a series of steel shim packs to allow for plumbing and leveling and were later dry packed with 6000 psi (41 MPa) non-shrink mortar, allowing no more than two stories to be erected between dry packing.

The forces at the base of the shear wall on grid line 5 were factored net tension and compression loads as high as 138 kip/ft^2 (6.6 MPa) and 485 kip/ft^2 (23.2 MPa), respectively. These tension loads were transferred into the foundation with connections capable of achieving a cumulative factored net tension of 1585 kip (7050 kN) over a 9 ft 6 in. (2.9 m) length of wall.

Twenty no. 11 (36 mm) reinforcing bars spaced at 6 in. (150 mm) on center were used to take this force. All of the bars had to be connected to the foundation with grouted couplers that were cast into the grade beams.

The contractor then used templates to record the as-built location of the couplers and sent these to the precaster to guarantee installation fit-up between the panel and the site-cast couplers. As a further check, exact as-built surveys of the coupler locations were sent to the precast concrete specialty engineer for review and were incorporated into the final piece drawings.

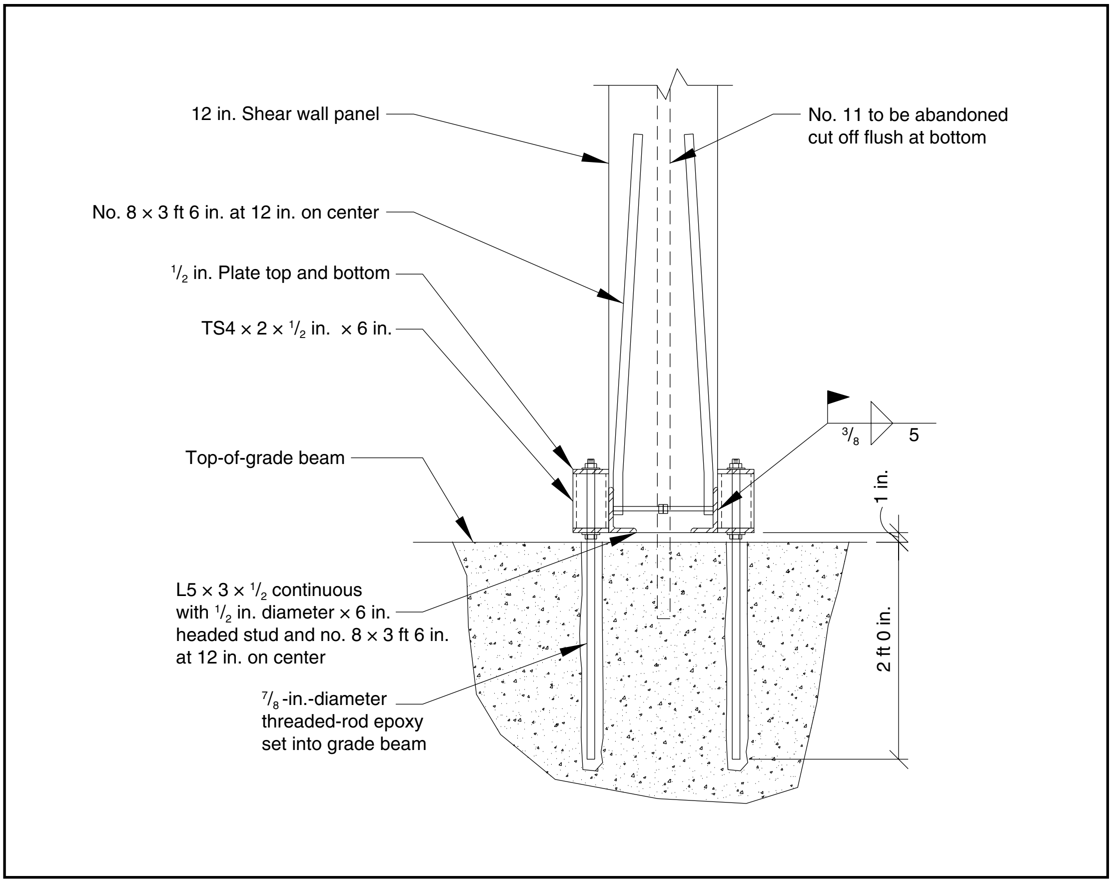
A retrofit panel-to-foundation emergency detail was developed to allow for the installation of a welded bracket that could be bolted to the foundation with an epoxy-set threaded rod drilled 24 in. (600 mm) into the grade beam or pile cap if the need arose. This bracket allowed field fit-up problems at the couplers to be rectified relatively easily.

Shear-wall coupler beams

In most buildings, it is difficult to find a completely uninterrupted length of interior shear wall that does not require openings for occupant circulation. In this building, the long interior shear walls required door openings at the corridor locations for elevator and stair access.

The floor-to-floor height was kept at an economical 9 ft (2.7 m), as is typical in residential high-rise construction. This only left a 1 ft 8½ in. deep × 1 ft 0 in. wide (520 mm × 300 mm) header above the doors, which had an additional blockout that was 5 in. (125 mm) deep at the top of it for plumbing-pipe horizontal runs. Instead of breaking the shear wall into separate segments at the doors and significantly decreasing the wall's stiffness, the header above the door was designed as a shear-wall coupler beam.

This coupler beam had unusually high shear and moment forces at the lowest stories and required special attention. The plane principal and shear stresses on this beam were calculated using the finite-element software. The shear values calculated required the use of



This diagram shows the panel retrofit to the foundation bracket. Note: 1 in. = 25.4 mm; 1 ft = 0.305 m. Courtesy of Palanisami & Associates.

an unconventional ¾ in. × 6 in. (19 mm × 150 mm) Grade 50 (345 MPa) steel bar cast into the panel to take the shear along with the more typical deformed-bar stirrups. The moment at this connection required the seldom-used no. 18 (57 mm) ASTM A615[5] reinforcing bars on the top and bottom.

Because of the large amount of reinforcing steel at this joint, the working drawings were plotted out at full scale by the precast concrete specialty engineer and were examined to determine whether the precaster would be able to achieve enough reinforcing-bar cover and clear spacing for the aggregates that were used. The possibility for voids and improperly consolidated concrete had to be considered in the development of the mixture proportions for these panels.

Flange-to-web connection

The H-shaped shear-wall section again helped limit the amount of potential tension on the non-load-bearing elevator shear wall for lateral force in the east-west direction by allowing the web to transfer vertical uplift loads into the flange shear walls. At this vertical panel-to-panel interface, the first-story shears were on the order of 30 kip (133 kN) per vertical linear foot of wall.

A reinforced grouted keyway connection was developed to accomplish this transfer of shear between the web and the flange and create a monolithic section. This connection used a field-grouted connection with projecting no. 5 (16 mm) hairpin bars staggered between panels instead of welding an embed plate to a loose plate at the joint. This method was deemed more cost effective than using numerous weld-plate connections that the erector would have to make in the field. Also, the cast concrete joint yielded a much stiffer net section by eliminating the flexibility inherent in steel connections.

Hollow-core floor system

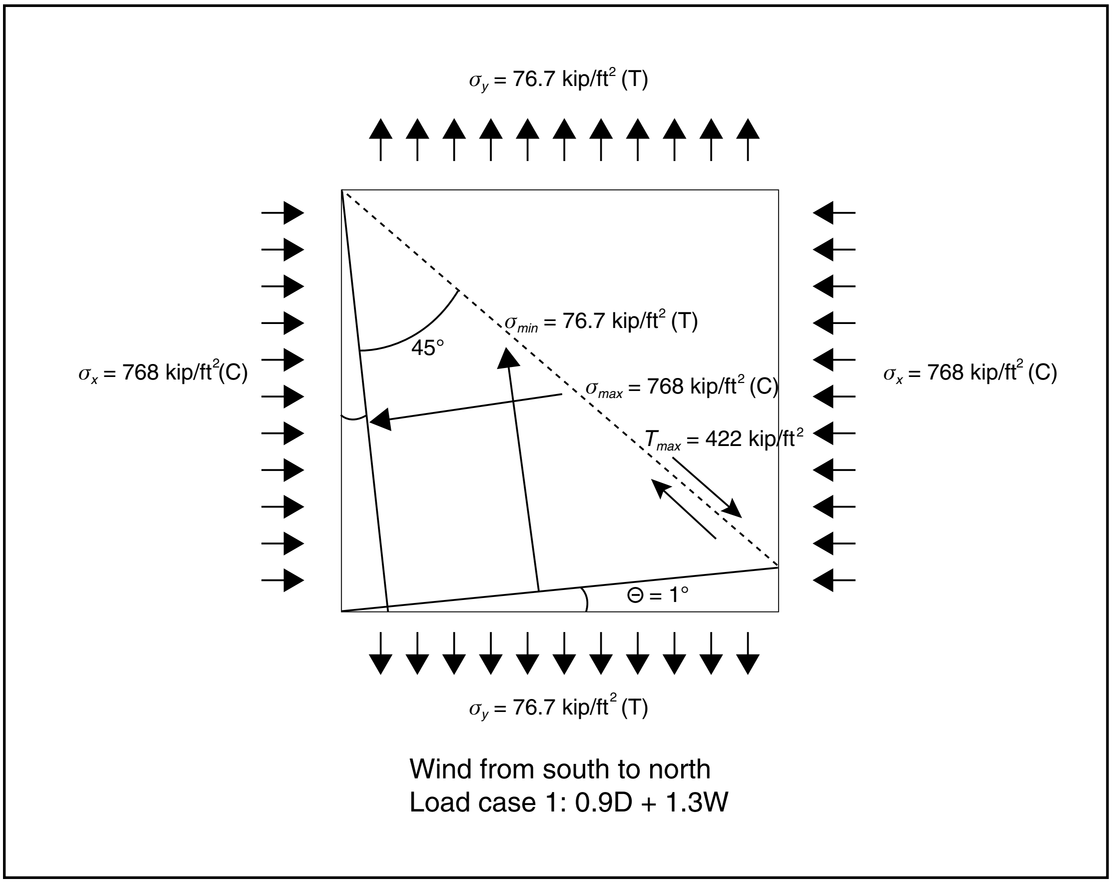
The floor system also had a number of detailing issues to consider. A 6-in.-wide (150 mm) continuous precast concrete haunch was used on all interior load-bearing wall panels to carry the 8 in. (200 mm) or 12 in. (305 mm) hollow-core floor slabs as a truly simple support.

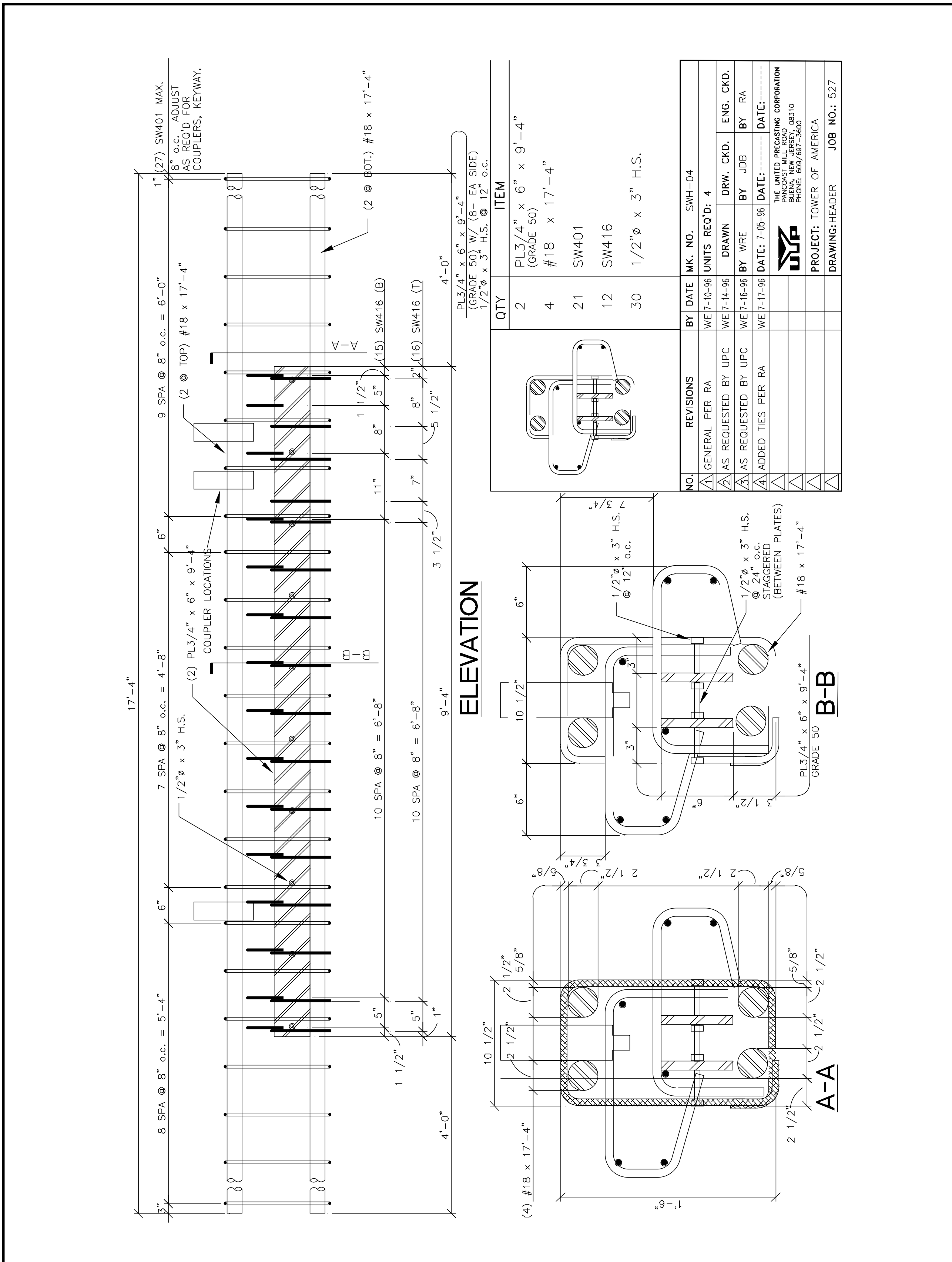
This system did not rely on a stacking system of panel on top of hollow-core slab at each floor, which is typical in low- and mid-rise construction. This alleviated the problem of high compressive stress on the webs of the hollow-core floor slabs caused by sandwiching as well as negative-moment slab cracking caused by partial-support fixity. The number of solid-grouted slab cores required at the bearing ends was reduced, thus reducing the hollow-core slab costs significantly. The possibility of overstressing the hollow-core slab bearing strips was also eliminated because the slabs did not have to carry more than one floor.

Diaphragm shear forces were transferred from the untopped hollow-core slabs to the shear walls by using no. 4 (13 mm) deformed bars that were 4 ft 0 in. long (1.2 m), which passed through 2-in.-diameter (50 mm) holes cast in the top of the shear-wall panel and grouted into the keyway between the hollow-core slabs on both sides of the shear wall. This detail was standard on all hollow-core-slab bearing conditions and is capable of handling the maximum service shear force of 300 lb/ft (4.3 kN/m) along the length of wall.[6] The detail proved to be cost effective in limiting the amount of field welding and cast-in plates required for the more than 1800 shear-wall–to–hollow-core-slab connections in this building.

A similar detail was used at the edges of the floor plate to provide a continuous tension chord for the diaphragm. At these locations, two no. 6 (19 mm) dowels that were 7 ft long

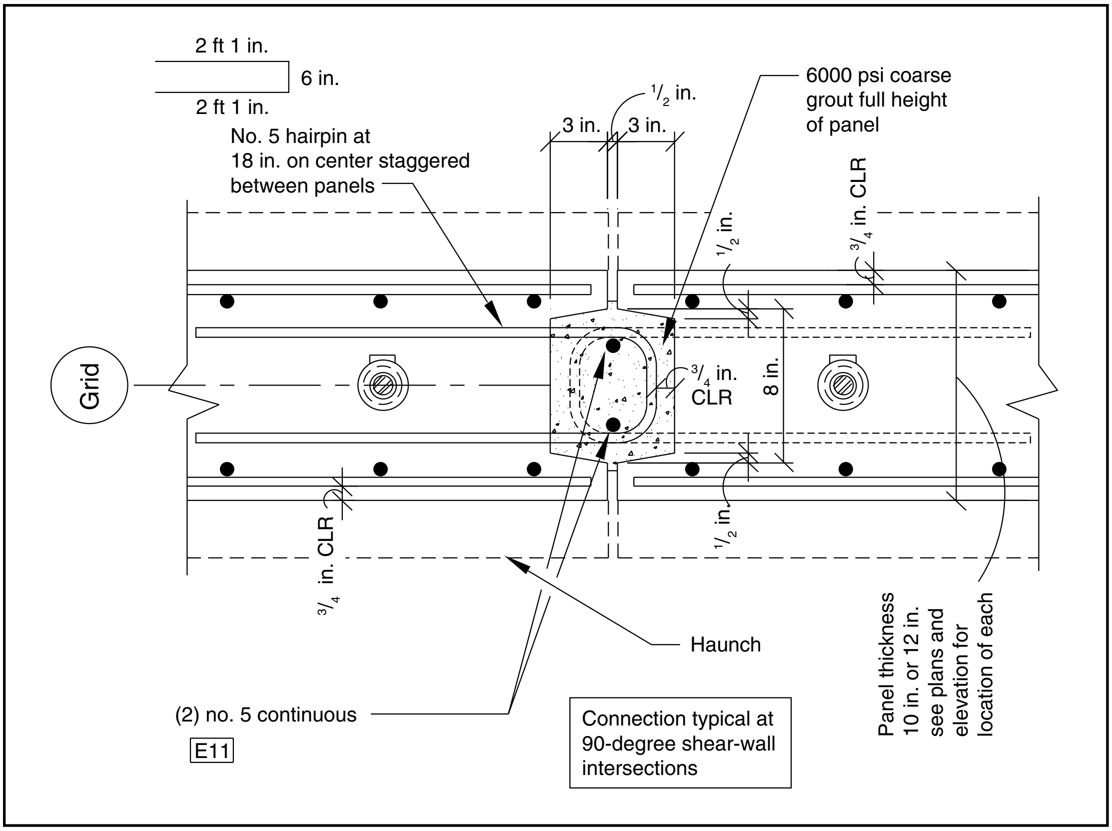
Plane stresses are shown at the SW-14 coupler beam. Note: SW = shear wall. 1 ft² = 0.0929 m²; 1 kip = 4.448 kN. Courtesy of Palanisami & Associates.





This precast concrete production hardware detail SWH-04 shows coupler beam reinforcement. Note: 1″ = 1 in. = 25.4 mm; 1′ = 1 ft = 0.305 m.

Courtesy of Palanisami & Associates.



This is a precast concrete erection drawing of a grouted keyway detail at the intersection of the H-shaped shear-wall flange-to-web field connection. Note: CLR = centerline of reinforcing bar. 1 in = 25.4 mm; 1 psi = 6.895 kPa. Courtesy of Palanisami & Associates.

(2.1 m) were used and grouted into 3-ft-long (0.9 m) slots field cut in the hollow-core slab flange and the cores were grouted solid.

At the exterior architectural precast concrete wall panels, a diaphragm connection was provided that was capable of carrying wind forces and out-of-plane seismic loads as well as the minimum structural integrity requirements of ACI 318.[7] Connecting the wall panels to only the first parallel hollow-core slab on the lap side did not provide enough anchorage force for the wall, so the second hollow-core slab from the wall panel was strapped to the first using a steel bar welded to embed plates that were shop-cast into the hollow-core slab. This was also required due to the lack of a reinforced floor topping.

A loose clip angle was then field welded to the hollow-core slab embed nearest the wall and bolted to a vertical slotted insert cast into the exterior wall panel. This special connection employed the dead load and hollow-core-slab bearing connections of two slabs working as a deep horizontal beam to resist out-of-plane wall forces.

Cast-in accessories

One of the many unique features in using precast concrete wall panels and hollow-core floor slabs on this building was the incorporation of a majority of the mechanical, electrical, and plumbing chases and horizontal runs into the panels as they were cast. At each level, there are notches in the tops of all of the interior vertical wall panels so that fire-protection sprinklers, plumbing pipes, and electrical conduit could run unimpeded perpendicular to the walls.

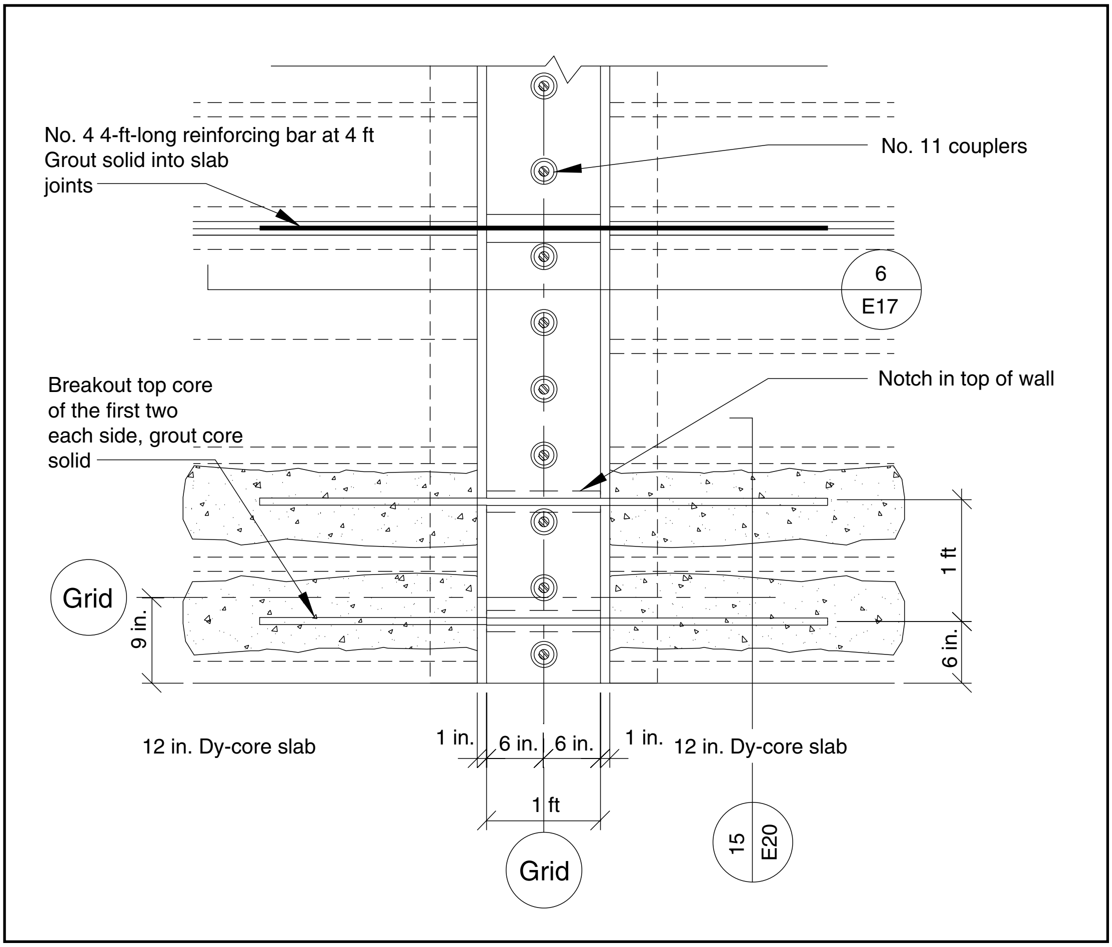
Electrical junction boxes and conduit within the wall panels were coordinated with the electrical subcontractor in order to achieve a shop-installed condition for much of the electrical wiring, thus saving countless hours of on-site installation time. The hollow-core floor slabs were also cast with openings for most of the larger mechanical ducts that would be running from floor to floor, thus saving on considerable core drilling and similar field modifications.

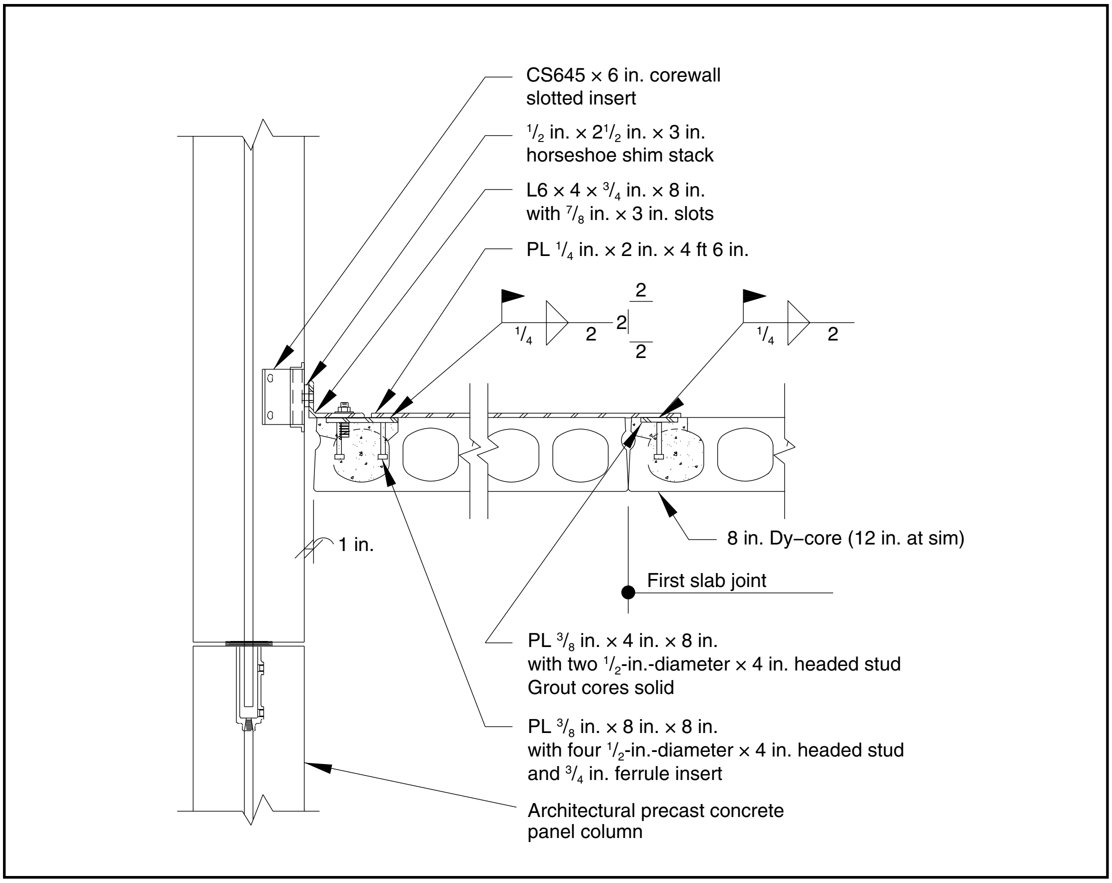
Project results

By the time this project was completed, the precast concrete specialty engineer had put in more than 6500 hours of engineering design, drafting, and administration on the project. The erection drawings were completed over a period of 10 months with 6 submissions for various phases of the project. More than 2000 individual production drawings for more than 5000 individual precast concrete pieces were completed for a four-month production schedule, and numerous field and clarification sketches were issued during the project's erection period of five months, with an average rate of 10 to 12 panels per day.

The coordination of the thousands of items present in a project of this size was strong testimony to the patience and forethought required for executing successful projects in precast concrete. As testimony to the building's creativity and groundbreaking design, the American Concrete Institute presented its Innovative Concept Award to the owners in 1998 following the structure's completion.

This is a typical grouted diaphragm connection at the ends of the shear wall. 1 in. = 25.4 mm; 1 ft = 0.305 m. Courtesy of Palanisami & Associates.





This is a typical architectural wall panel–to–diaphragm connection. Note: PL = plate. 1 in. = 25.4 mm; 1 ft = 0.305 mm. Courtesy of Palanisami & Associates.

The final construction cost for the building in 1997 was $46 million. The building has been open and occupied for more than 10 years and is proving to be both an engineering and financial success for the owner. The late Samuel LeFrak, one of the former chairs of The LeFrak Organization, also received a U.S. patent for this precast concrete construction method shortly after the building was completed.[8] LeFrak is currently developing many other buildings in the Newport area of Jersey City. Churches says, "The opportunity to do another high-rise building like this will certainly happen one day. It is just a question of when everything will fall into place again."

Precast concrete component quantities

Quantity	Component
2101	8 in. hollow-core slabs
1417	12 in. hollow-core slabs
419	Load-bearing wall panels
1150	Exterior wall panels
153	Hardware assemblies

Note: 1 in. = 25.4 mm.

Erection at the 20th story of the Tower of America West in Jersey City, N.J., is performed with a larger crawler crane. Photo courtesy of Tower of America Urban Renewal Co.

Conclusion

"Given what was learned on this building, 40 or 50 stories should be able to be achieved using this type of system without any problem," Palanisami says while reflecting on the project's history.

With today's high-strength concrete, computer analysis capabilities, and more developed understanding of precast concrete, the engineering challenges could be met. Given a better understanding of stacked precast concrete bearing walls, the ability of building designers, producers, and erectors to meet its challenges will be there.

What will drive the continued use of load-bearing precast concrete in high-rise structures all over the world is the constant push for high-speed construction schedules, fully integrated design, construction economics, and the ability of precast concrete manufacturers to produce a large number of high-quality pieces to meet the client's demand. As the precast concrete market progresses in locations such as India, China, and Dubai, the industry will certainly witness the construction of many more of these precast concrete high-rise buildings worldwide.

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Synopsis

Towering over the Hudson River waterfront in Jersey City, N.J., the 34-story Tower of America West won an ACI Innovative Design Award in 1998. This unique building is constructed almost entirely of precast concrete, including stacked interior bearing walls, hollow-core floor slabs, and stacked exterior architectural wall panels. Although construction was finished in 1997, it is still one of the tallest buildings with this type of structural system in the United States. This article presents some of the design and construction challenges that were overcome, as well as highlights of some of the important details involved in designing and erecting tall precast concrete structures.

Keywords

All-precast concrete, building, connection, high-rise, hollow-core.

Reader comments

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