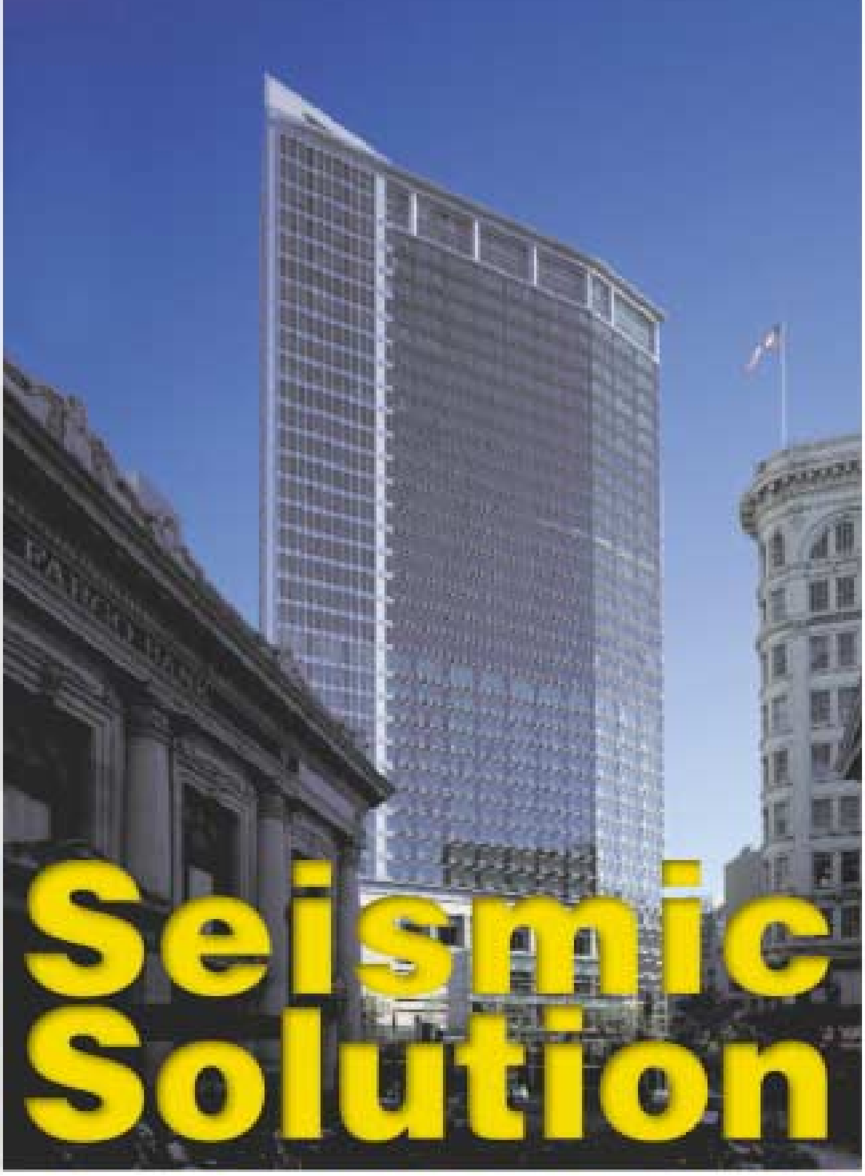

THE Seismic Solution

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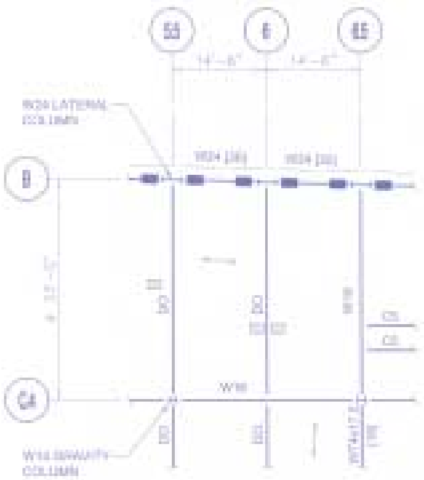
Long deck spans, upturned spandrels and toggle brace-dampers meet the complex seismic requirements of the irregularly shaped steel tower for San Francisco's Four Seasons Residences.

Amid San Francisco's elegant skyline stands an example of what innovations in tall building design can achieve in demanding engineering environments. The Four Seasons Residences is a 40-story, mixed-use building that opened in fall 2001. At over 400', it is one of the tallest and most irregular shaped steel-framed buildings built in San Francisco since the 1994 Northridge earthquake. The project is a joint venture development between Millennium Partners and WDG Partners.

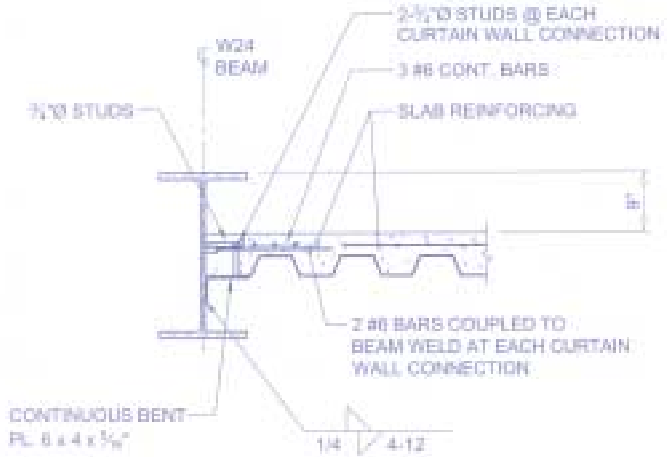
This one-million-square-foot building houses 277 rooms for the prestigious Four Seasons Hotel, 136 luxury condominium units, The Sports Club/LA-SF and exclusive retail shops. Valet parking is provided in the basement levels. The hard cost for construction was approximately $350 million. The building has five stories below grade and 37 stories and mechanical penthouses above. The below-grade portions penetrate to a depth of 60', not including the 7'-6" mat foundation. The basement and podium levels fill the entire 55,000-square-foot footprint of the site, while the hotel and condominium floors encompass an area of 20,000 square feet.

FOUNDATION WORK

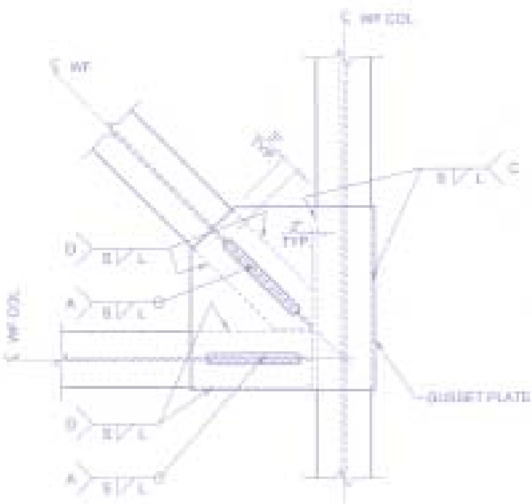
The site is located immediately south of the Bay Area Rapid Transportation train lines (BART) and the San Francisco Municipal Transit (MUNI) tunnel running along Market Street. A 3'-thick slurry wall constructed around the perimeter of the site and braced by large-diameter steel pipe sections provided the necessary shoring for the 70'-deep excavation. Site conditions, the depth of the excavation, the water table (at 36' below grade) and relationships to critical transit systems required the slurry wall to be extended down 100' from grade.



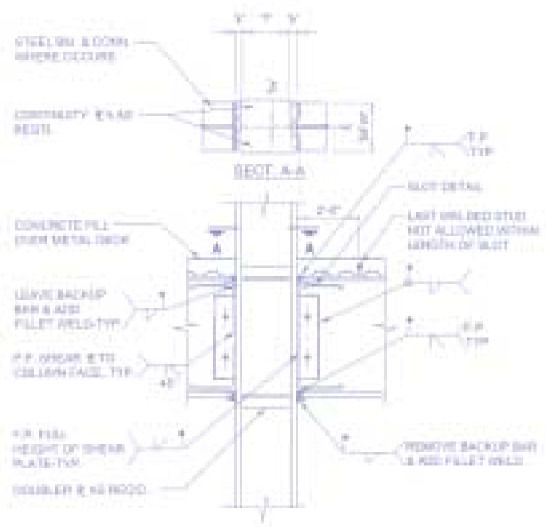
Typical 14'-6" floor framing module. This span allowed for an entire hotel unit to be constructed with its ceiling at the underside of the floor decking instead of at the underside of the W16 beams. This results in a 16" savings in headroom.



Typical "Upturned Spandrel" detail between columns. A modified detail was used within the panel zone to provide the necessary lateral bracing of the column.



Slotted gusset plate detail used at heavily loaded truss connections. This detail allows the connection to develop the full axial capacity of the members framing into it while minimizing the weld lengths.



Seismic Structural Design Associate's Slotted Web™ seismic moment-frame connection detail. Beam slot lengths varied based on beam size and length.

In addition to shoring, the slurry wall ultimately became the building's basement/foundation wall.

In order to expedite the construction schedule for the above-grade structure, construction manager Bovis Lend Lease requested that the lowest basement levels not be constructed until all incoming underground plumbing and mechanical work was substantially complete. Therefore, a top-down construction technique was utilized to allow the above-grade steel framing to progress prior to completion of below-grade work.

CHOOSING STEEL

During the conceptual stages of the project, the design team studied different alternatives for materials and framing systems. Several major parameters of comparison were: speed of construction, safety, weight, availability of materials and skilled labor, and cost. The most important requirement driving the choice of structural system and material was the maximization of free window area on the perimeter of the building. Architect Gary E. Handel & Associates and developers insisted on large unobstructed views from all sides of the building. The design team's philosophy in this respect can best be described as a "solution in search of a structural system," for which steel fit the bill.

The Bay area lacks available high-strength concrete (f'_c > 6,000 psi). This means that the wall, column and beam sizes in a comparable concrete structure would be too large to fit the archi-

Photograph of the rear of the building during construction. The irregular "boot" shape of the structure is evident from this angle. Also shown are the cantilevered ballroom and basketball levels as well as the three story voided area for the future museum.

tectural and other programming requirements of the building. In addition, the weight of a concrete structure and the equivalent lateral force would be approximately 50% greater than that of a steel building. With a vast labor pool of quality steel workers, an erection schedule of two floors per week, stringent architectural constraints and the seismic requirements for ductility, steel was the obvious structural material of choice for this project.

FRAMING SYSTEM

The building is constructed of approximately 15,000 tons of structural steel. The contractor, PDM-Strocal of California, and detailer CanDraft of British Columbia, worked closely with the design team to accommodate the project's accelerated schedule. The mill order contained every section type available: standard wide flange, channel, angle, HSS, pipe, jumbo shapes, built-up wide flange and plate fabricated box sections. The majority of the steel used for the project was ASTM A572, Grade 50 meeting the special requirements of 1997 AISC Technical Bulletin No. 3. *(Editor's note: ASTM A997 is the current specification for this type of steel.)*

Several different framing systems responded to the various architectural constraints and requirements imposed on the structure. In this respect, form followed function. The tower framing consisted of standard wide-flange floor beams and special moment-resisting frame (SMRF) spandrel beams. The typical floor beams were limited to W16 sections and smaller while the perimeter SMRF spandrel beams were typically W24 sections. The sizes of these spandrel beams varied from W24×76 to as large as W24×229.

Typical interior column sections were standard W14 shapes except for columns supporting the braced core. These heavily loaded columns required jumbo W14×605 shapes. The SMRF columns used in the perimeter frame varied in size from W24×176 to W24×370 and many required added cover plates at the lower floors. Due to re-entrant corners and jogs in the exterior frame, 24"-square box columns were needed to resist biaxial bending forces from intersecting frames. These box sections were comprised of 1"- to 3"-thick plates, and they featured continuity plates (web-stiffener plates) at the moment connections. Due to extreme variations in size between interior gravity columns and the exterior SMRF columns, and due to the differences in the stress distributions, engineers developed a schedule whereby column lengths were corrected during the fabrication process in order to compensate for differential and overall column shortening. Typical compensations varied between 1/8" to 1/4" per two-story column lift.

Typical floor slabs consisted of heavy-gauge metal deck filled with light-weight concrete. Important diaphragm floors were constructed of heavily reinforced thickened slabs.

THE PODIUM

As can be expected, the framing of the lower six "podium" or public floors was extremely complex due to double-height spaces for sports and recreation facilities, pools, ballrooms and basketball courts. In addition, an area 45' wide by 200' long by three-stories high was omitted at the southern end of the podium. The structure was left void to allow for future construction of a museum within the space.

Meshing the amenity and mixed-use spaced within the podium caused a lack of space. The design team had to locate an 11,000-square-foot, column-free hotel ballroom directly over a 10,000 square foot column-free basketball court. This required that the entire ballroom structure span 90' over the basketball area below. This long span was accomplished by using W40×362 beams at 10' on center. The roof over the ballroom was framed with 4'-deep trusses spaced at 15' on center and composed of WT and double angle members.

To further complicate matters, one entire supporting edge of the basketball area, and hence the support of the ballroom, was cantilevered 15' over the building entrance below. To account for this stacking effect and the discontinuous load paths, a 3-D finite element analysis (FEA) of the structure was made. Using this FEA model, a state-of-the-art 3-D vibration analysis was performed to quantify and design for the effects of this stacking on the transmission of floor vibrations vertically through the structure. Ultimately, the solution contained a system of continuous bridging, or blocking, between the long span beams supporting the ballroom beams. This continuous connection engaged a larger extent of floor framing to resist the isolated vibration source, i.e. the dance floor. In addition, the full-depth bridging provided lateral restraint for the beam-bottom flanges, which we believe is a primary source of vibration in long span deep beams.

As functions within the building changed, the need to relocate or transfer columns became necessary. Transfer elements were generally in the form of trusses formed using the girders at two levels as chords and diagonal-compression web members. These A-frame trusses were composed of chord and web members as large as W14×605. As an alternate, a vierendeel configuration was also used. The vierendeel was formed by transfer girders at two adjacent floors linked with a vertical strut located under the terminated column, used mainly at exterior conditions.

PDM and CanDraft indicated early on that fabrication and construction of the large-truss gussets would be extremely difficult due to the design-weld lengths. DeSimone Consulting Engineers developed an innovative solution incorporating "slotted gusset plates" to allow for full-truss-member capacity development of the joint, while minimizing the size of the gussets and hence providing more usable space. This slot, which creates eight weld lengths per connection instead of the usual four, also eliminated any potential lag effect in the large truss members. A finite element analysis (FEA) of these gussets was performed to account for the complex distribution of stresses in the plates and welds. The analysis results were used to determine optimum dimensions of the slot as well as the size and location of the end radius. The size and location of the radius was designed to allow ductile, non-linear, stress concentrations to form in the "sacrificial weld" material placed in this area prior to initiating fracture in the plate material.

Construction photo of the bent-knee toggle damper assembly. Note the deviation angle of the main diagonal struts.

STEEL RESIDENTIAL CONSTRUCTION

A minimal distance between ceiling and floor construction is an important factor in residential construction to allow the maximum number of levels within the height limits set by local planning codes. To facilitate this requirement, beam spacing in the hotel and condominium level was set at the typical hotel unit module of 14'-6", allowing coffered ceilings to extend up to the metal deck. A 3", 16-gage deck was used to span these long distances without requiring shoring. An exception was at the corner units, which required unobstructed spans of more than 20'; this could only be accomplished by using temporary reusable shoring beams.

Beams in the exterior special moment-resistant frames were further positioned with the floor slab located at the mid-depth to provide maximum window-head height within the unit. The projection of the floor beam above the finished floor line filled a void created by the code requirement of raised sills at high-rise windows. This upturned spandrel arrangement was essential in meeting the architectural limitation of minimal floor height with maximum window areas.

LATERAL ANALYSIS—WIND AND SEISMIC

The building was analyzed to resist lateral forces from both wind and earthquake. Seismic forces were based on the provisions of the 1995 San Francisco Building Code (SFBC) (1994 UBC). Although the 1994 UBC was the basis for generating the lateral forces for the structure, some of the 1997 UBC requirements were implemented, particularly with regard to redundancy and SMRF detailing. Wind forces initially considered the provisions of the 1995 SFBC, but were ultimately based on wind tunnel tests conducted by RWDI of Canada. Generally, the earthquake forces governed the strength design of the building, but the wind-induced displacements and motions governed the serviceability requirements. The seismic design philosophy used by DCE incorporates a load / capacity pyramid. By this approach, no member is ever designed for load levels in excess of that which the surrounding structure, (i.e. connections, diaphragms, anchors, other members, etc.) can deliver to it.

The lateral analysis was performed using a three-dimensional computer model created with ETABS by CSI. Due to the structural irregularities and complexities of the project, a three-dimensional model representing the spatial distribution of mass and stiffness was created. This model accounted for the effects of mass offsets as well as seismic loading from any direction. Analysis of the building's substructures and diaphragms was accomplished using the finite element capabilities of SAP2000 by CSI. A dynamic response spectrum analysis was performed using the site-specific elastic response spectrum developed by the geotechnical engineer. This spectrum represents a ten percent probability of being exceeded in one hundred years.

To achieve the required drift limitation of the structure, as well as redundancy, a dual system was implemented. A special concentrically braced frame extends up through the tower of the building at the elevator core, which has two parallel elevator banks. In one direction, two sets of braced bents were coupled together with a specially designed shear link beam, which is intended to absorb much of the energy that otherwise would be taken by the axial loaded columns and braces. Additional special concentrically braced frames were added within the podium levels.

The moment frames were designed using the special moment resistant frame (SMRF) provisions of the 1995 SFBC (1994 UBC). The frame provisions of the 1997 UBC were also considered, especially with regard to redundancy and detailing. The SMRF provisions were based on the failure of moment-frame connections in the 1994 Northridge earthquake. The new code provisions required that moment connection details be pre-qualified by special testing procedures.

"SLOTTED WEB" SEISMIC MOMENT CONNECTION

When the design team was considering which moment-frame connection to use, the structural engineering profession was well into the "Post-Northridge" (P-N) era. Virtually every type of accepted P-N connection was re-

24” diameter extra-strong steel pipe cross-lot braces with W36×182 spreaders were used for the lateral support of the slurry walls during excavation.

viewed, and two were given serious consideration: the reduced beam section, or dog bone; and the Seismic Structural Design Associates (SSDA) Slotted Web™ connection. The SSDA connection was chosen.

The Slotted Web connection had a number of benefits over the dog bone for this particular project. The first of these benefits was the elimination of the lateral-torsional mode of buckling required by the dog bone connection for effective hinge development. Also, architectural limitations driving the upturned-spandrel configuration, with perpendicular beams framing-in only at the columns, did not allow for bracing of girder flanges within the clear span. The factor that eliminated the use of the dog bone connection was the typical 12’-6” girder clear span. This short span has a moment gradient that is sufficiently steep to produce excessive moments at the column face due to full development of the plastic hinge moment at the reduced section.

Conceptually, the slotted-web connection resolves bending forces into two uniformly distributed axial forces at the beam flanges and directs nearly the entire shear force into the web connection. Although this load mechanism is assumed by most designers of moment connections, it does not reflect the true load path, as was evident during the 1994 Northridge Earthquake. By decoupling the flanges and the web, shear and bending forces go where they are supposed to go and not where they are assumed to go. The detachment, or decoupling, of the flanges also produces a more uniform distribution of stresses across the width of the flange. This uniform distribution results in minimal stress concentration in the flange weld, normally occurring just above the web location. This in turn reduces the possibility of K-area fractures resulting from these stress concentrations.

WIND DAMPERS

While seismic forces governed the lateral design of the building, wind tunnel tests indicated there might be perceived lateral motion in the north-south direction of the building due to wind loading. To minimize this perceivable motion, DeSimone Consulting Engineers worked with RWDI and Taylor Devices to design dampers to be installed within certain portions of the building. These state-of-the-art, non-linear-fluid, viscous wind dampers, arranged in a bent-knee “toggle” configuration, were used on alternate floors above the 17th floor. This arrangement, developed by Taylor Devices of Buffalo, amplifies the relative story movements by as much as four times. This increased sensitivity allows for dissipation of even the smallest wind-induced motions. The damping effect also reduces the seismic demand on the structure by changing the dynamic characteristics of the building. This effect was evaluated with a time-history analysis. Since the dampers are intended to limit service-level wind effects only, the lateral analysis and design of the structure was conducted assuming conditions of functioning dampers and subsequently destroyed dampers. In fact, Taylor devices established the load at which the dampers would fail in a sacrificial manner during a seismic event. This project represents the first practical use of this damper arrangement in a building.

CONCLUSION

In addition to the above-noted technical advances, the building’s structural design met the requirements of providing the greatest volume of space per vertical foot of height. All of these solutions made this building a great success, and it has been well received by the occupants and the surrounding community. Only by utilizing recent innovations such as upturned spandrels, slotted gussets, toggle-braced dampers, slotted-web moment connections and 3-D floor vibration analysis was it possible to design this highly complex project in the one of the most demanding engineering environments in the country.

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