Design of the Tallest Reinforced Concrete Structure in California – a 58-Story Residential Tower in San Francisco

Derrick D. Roorda, SE, Senior Associate
Nicolas J. Rodrigues, PE, Senior Project Engineer
DeSimone Consulting Engineers
San Francisco, CA

Abstract

This paper summarizes the analysis and design as well as special component testing performed for the Millennium Tower project located at 301 Mission Street in San Francisco. When completed in 2008, at 645 feet, this project will be the tallest reinforced concrete structure situated in a seismic zone 4 region, the 4th tallest structure in the City of San Francisco, and the tallest residential building in the U.S. west of Chicago. The tower has 58 occupied floors and combines with an adjacent 12-story tower and surrounding podium to provide over 1,150,000 sf of luxury condominiums and recreational amenities. The tower's immense height posed many challenges and required the creative use of technologies and cutting edge innovations.

The tower's dual lateral system is comprised of a 36-inch-thick concrete shear wall core and partial perimeter Special Moment Resisting Frames (SMRF). The heaviest building ever constructed in San Francisco on a psf basis, the tower is supported by a mat foundation resting on 950 130-ton piles, each approximately 80 feet in length. Outrigger trusses connect the interior core with robust perimeter super-columns at three intermediate levels to control lateral deflections. In order to reduce the required floor-to-floor heights, shallow steel link beams are used within the shear wall core as coupling beams, in lieu of deeper diagonally reinforced concrete beams.

Closely spaced ties in columns and walls posed a challenge to the placement of the high strength 10 ksi concrete needed for this project. To alleviate some of this congestion, DeSimone specified a system of Welded Reinforcement Grid (WRG) that eliminated all hooks, significantly reduced the volume of rebar, and decreased overall labor costs. A successful laboratory test program was implemented, in conjunction with supporting calculations, to demonstrate the adequacy of this product for use on the project.

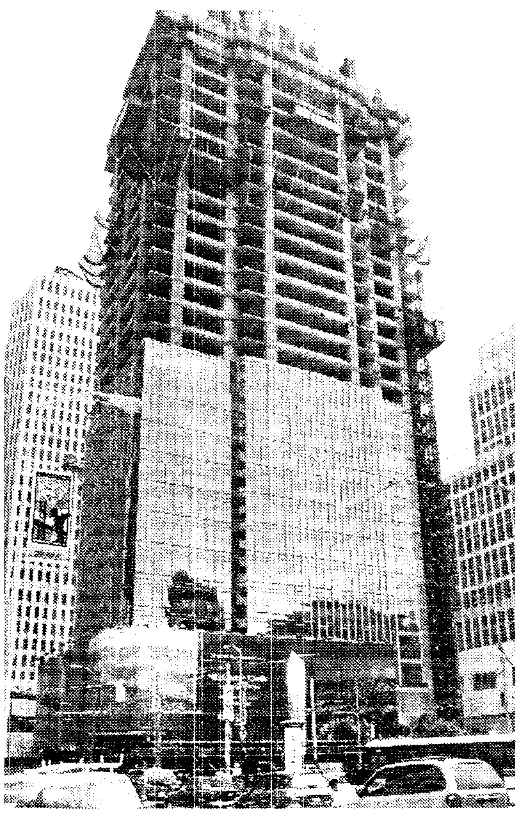


Figure 1: Construction Progress as of July, 7th 2007.

Introduction

The demand for residential living in San Francisco has spurred a new era in high-rise residential construction. In such an era, new challenges and new engineering solutions have become necessary.

The design of this building utilizes 10 ksi concrete, and reinforcing of grade 75, both of which are firsts for high rise construction in San Francisco. Even with these special materials, the reinforcing ratios were so dense for a project of this height that alternative systems had to be employed to allow for successful concrete placement.

Structural steel link beams were used in lieu of diagonally reinforced concrete beams, special WRG was used for confinement, and self-consolidating concrete was specified. These features relieved congestion and permitted properly consolidated concrete to be placed more reliably and also allowed faster construction.

Lateral System: Single System or Dual System?

DeSimone began investigating alternative designs for the project in 2001, initially exploring structural steel systems. As the project went through the entitlement phase, there was a shift in economies of high rise residential construction in California that allowed concrete to be more competitive. When the design started in earnest in 2004, concrete appeared to be the most cost effective. DeSimone suggested that the owner could consider the use of a core-only design using a Performance Based Design approach. After evaluating the risks involved, and noting that other such projects had been tied up in peer review for years, the owner chose to use a dual system in conformance with the UBC (1997) with the expectation that the review and approval process would be faster.

Lateral System: Perforated Outrigger/Shearwall System with Concrete SMRF

The lateral system is comprised of a rectangular box shearwall core with concrete SMRF's located at the building perimeter as shown in Figure 2, thus qualifying as a dual system recognized by the 1997 UBC.

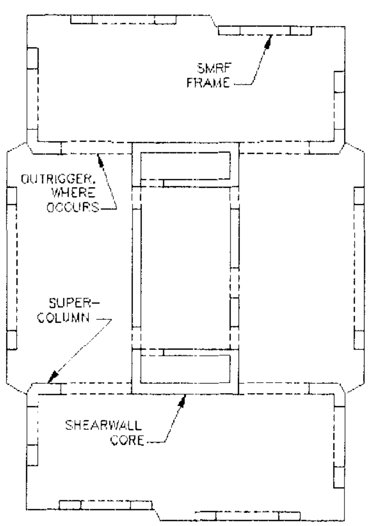


Figure 2: Plan of building

The long direction of the building has adequate stiffness for the design level earthquake. However, the short direction of the box is not adequate on its own. Therefore, large super-columns were introduced at the perimeter of the building and connected to the shear wall core with perforated outriggers at three locations up the height of the building as shown in Figure 3. The outrigger perforations allow pedestrian access around the entire core at each level. Without these openings all residential units adjacent the outriggers would have been two level-units, which was not acceptable to the building owner.

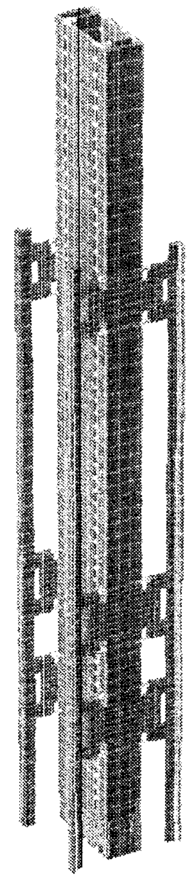


Figure 3: Isometric View of the Shearwall and Outrigger Lateral System

Each of the outriggers is comprised of two distinct elements. The first, providing connection to the shear wall core, is a multiple-story, robust, solid concrete element. The second, connecting the solid portion to the super column, is a pair of diagonally-reinforced link beams.

The diagonally-reinforced link beams were designed for the demands obtained from performing a response spectrum analysis using the provisions of ACI 318 section 21.6.7.4 (1999). A capacity design approach was utilized to design both the solid portion of the outrigger, as well as the connection to the core. The maximum probable shear capacity of the link beams was calculated, and the results were used as design forces for the rest of the outrigger and for the design of the connection to the core. The resulting outrigger rebar detail is shown schematically in Figure 4.

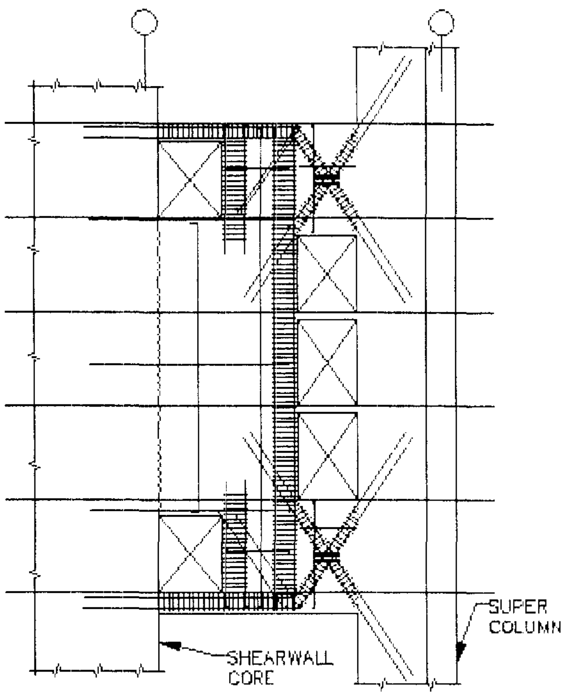


Figure 4: Detail of the Outrigger Reinforcement

The super-columns were also designed using the same capacity design approach. The maximum probable shear capacity of the link beams was added to the tributary gravity loads in order to determine the design strengths of the column. This approach was slightly conservative since it assumes that all of the outriggers would reach their maximum capacity at the same time. With the higher-mode effects of tall building response, this condition is unlikely to occur.

Interaction between Shear Wall and SMRF

At the height of this tower, the SMRF attracted very little lateral load (about 5%-8% base shear) away from the shearwall core, calculated using response spectrum analysis. The UBC (1997) requirement to design the SMRF of a Dual system for 25% of the base shear controlled the design of the SMRF's.

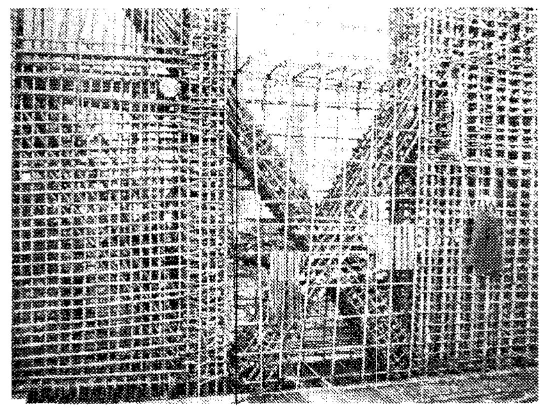


Figure 5: Photograph of the As-Built Outrigger

Since higher mode effects are so prevalent in a building of this size, the typical procedure of applying 25% base shear statically was not adequate. Therefore studies were performed where the core was given various reduced stiffnesses and response spectrum analyses were performed. The resulting force distribution in the frames was observed. This was done in an attempt to envelope the possible mode shapes and force distributions. Instead, we found that in order for the frames to attract 25% of the base shear, the core stiffness had to be reduced to less than half of what the design stiffness was, and the building had to drift to more than twice the code allowed displacement. The study made a good case that the UBC 25% base shear requirement is too high. More research should be done in this area, to reduce this apparent conservatism inherent in the code.

Since the UBC (1997) requires the frames to be designed for 25% of the base shear, DeSimone adhered to the code.

Reduced Floor-to-Floor Heights: Steel Link Beams

Mechanical systems in residential buildings are generally confined to small areas adjacent the interior stair and elevator core. For this reason, such buildings can often be built with shorter floor-to-floor heights than office buildings. For this project, a 9'-7" floor-to-floor height was desired for the lower levels, which allowed only 21" for link beams above the door openings into the core. Since diagonally reinforced concrete link beams would not fit within this dimensional constraint, and since conventional concrete link beams do not exhibit enough capacity, an alternative had to be found.

Section II.16 of the AISC Seismic Provisions (AISC, 2002) allows for Special Reinforced Concrete Shear Walls Composite with Structural Steel Elements. AISC requires these steel link beams to be designed following the provisions for the link portion of an Eccentrically Braced Frame.

However AISC does not go as far as to provide provisions for the design of the connection to the concrete wall.

After review of a research summary paper (Harries, Gong, Shahrooz, 2000), the equation proposed by Marcakis and Mitchell was used to determine the required embedment length. The demand strength, V_u used for the embedment calculation, as shown in equation (1), was based on the capacity of the wide-flange beam. The capacity was increased as described in table I-6-1 of the AISC Seismic Provisions (2002) by a factor of 1.1.

$$V_u = \frac{0.85\phi_c f_c' b'(L_e - c)}{1 + \frac{3.6e}{(L_e - c)}} \qquad (1)$$

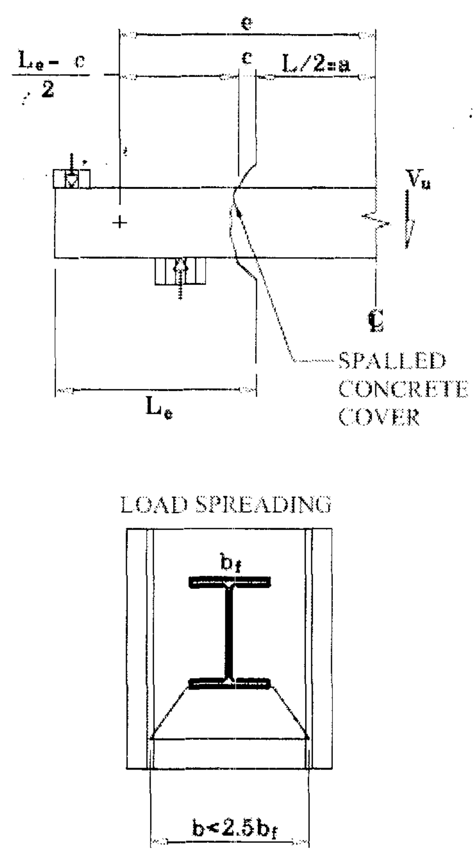


Figure 6: Embedment model as proposed by Marcakis and Mitchell

This approach assumes a rigid body rotation of the embedded portion and uses bearing against the concrete above and below the beams flanges for the development of the link beam moment. The bearing capacity is based on a 45 degree angle which spreads out to the vertical bars in the concrete wall.

An additional design consideration, as required by AISC (2002), is that two-thirds of the vertical steel required to develop the shear capacity of the beam must be placed in the first half of the embedded zone. A stiffness reduction for the steel beam was used in the analysis model in order to account for some inelastic action of the embedded zone. Confinement of the shearwall directly above and below the flanges required the use of one half-coupler to be welded to the steel beam at the face of the wall.

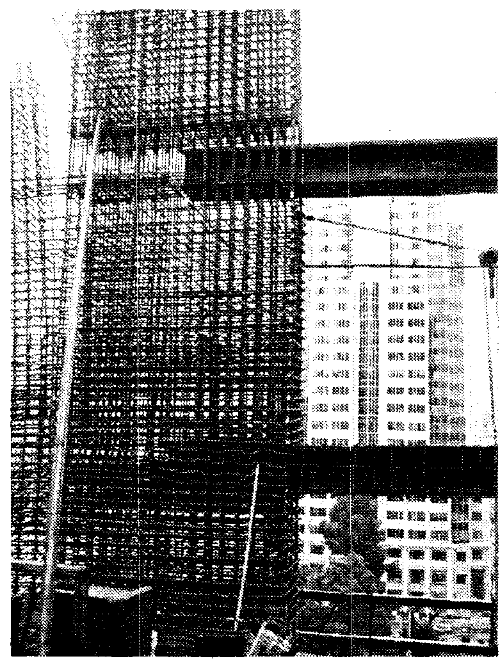


Figure 7: Photograph of Two Floors of As-Built Steel Link Beams

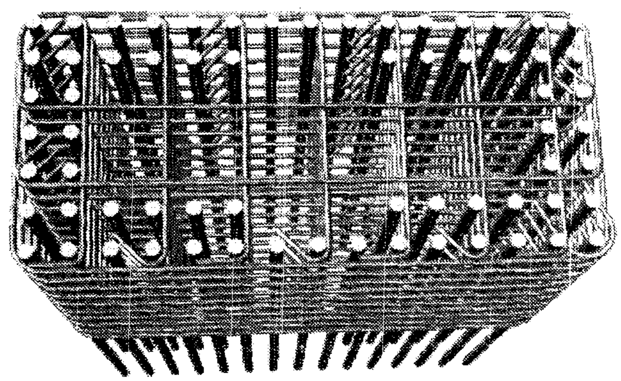
An added benefit of using structural steel link beams in lieu of diagonally reinforced concrete beams is that the steel sections are easier to install, and they relieve reinforcing congestion in boundary elements in the adjacent wall sections.

Rebar Congestion Relief: Welded Reinforcement Grid

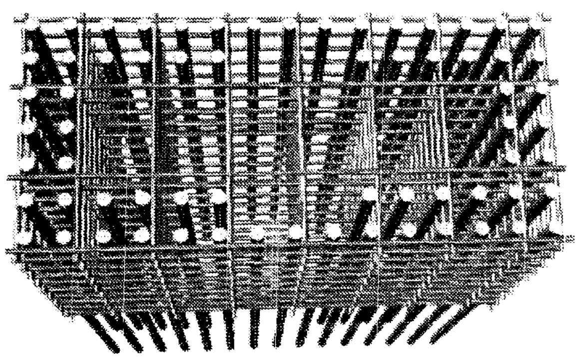
The concrete shear walls and the outrigger super-columns required large amounts of tension reinforcement. For example, the base of the super-column was reinforced with

152-#14 bars resulting in a reinforcement ratio of 6%, the code maximum. However, the vertical steel reinforcement only comprises about half of the total reinforcing volume in these elements.

The current ACI 318 and 1997 UBC equations for column and boundary element confinement reinforcing are directly proportional to the concrete strength. For 10 ksi and grade 75 ties, the requirement is #5 bars at 4" o.c. vertically and about 6" o.c. horizontally. Full scale pre-construction mockups constructed for the project showed that conventional ties with hooks could be built with this spacing, and the placement of concrete was possible, although quite challenging.



Conventional Hoop and Ties



Welded Reinforcement Grid System

Figure 8: Conventional vs. WRG System

DeSimone originally considered the use of 12 ksi concrete for this project, as recent construction on the new Bay Bridge has proved that such strengths are achievable in the area at this time. However, this would have required even more confinement reinforcing, and may have resulted in a design that is not constructible. For this reason it was decided to limit the strength to 10 ksi.

At the request of the contractor, DeSimone allowed the use of WRG. This system allows for quick erection and greatly reduces reinforcing congestion in confined zones, thus increasing the likelihood of proper concrete consolidation.

While the WRG system product used has ICBO and ICC approval for use with all strengths of concrete, it had never been used before in San Francisco in a building with 10 ksi concrete. For this reason, the Structural Peer Review Panel (SPRP) questioned the use of WRG on the project and the City of San Francisco ultimately required laboratory tests to demonstrate that the product would perform as expected.

The primary concern of the SPRP was that the WRG welds would not be able to develop the full tension capacity of the grid bars when subjected to an in-air weld shear test (Figure 9). This test represents one of four QA/QC tests performed on the proprietary WRG system in order to keep it's ICBO/ICC approval current.

Average ultimate stress of in-air test = 47 ksi.

While $f_{y\ \text{base material}}$ = 80 ksi

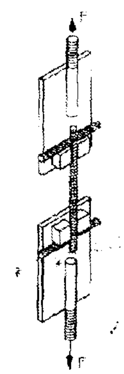


Figure 9: WRG Quality Control In-Air Weld Shear Test

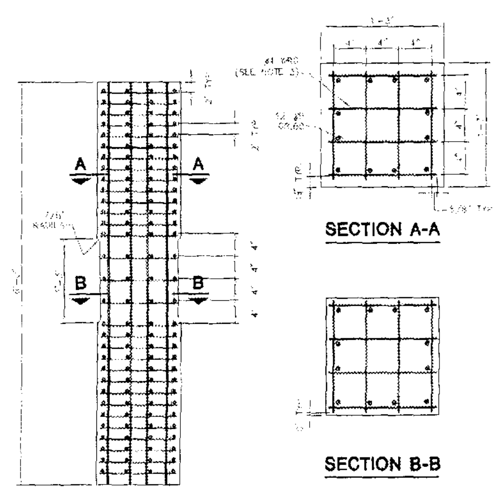


Figure 10: Test Specimen Compression Tests

In order to demonstrate that the product was acceptable for use on this project, the SPRP suggested testing a group of specimens constructed similarly to the portion of the shear wall core that is expected to undergo the largest strain demand. A test specimen configuration having the same vertical reinforcing ratio, concrete strength, reinforcing bar strengths, and volumetric confinement ratio was thereby determined and agreed to. The resulting specimen was a 15" square column reinforced as shown in Figure 10.

It was also agreed that each of the three specimens would be subjected to a monotonically applied compression load.

The 1997 UBC section 1921.6.6.5 provides a procedure for calculating strains in shear wall boundary elements, and allows confinement reinforcing to be eliminated if the strain is low enough. DeSimone proposed the use of this procedure to calculate the demands in the walls on this project, and to thereby set the criteria for the test specimens.

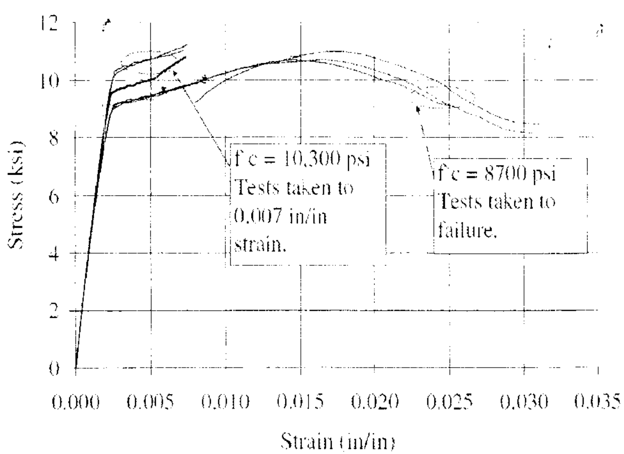


Figure 11: Results from WRG Compression Tests

The strain resulting from the UBC procedure, which is based on the Design Basis Earthquake (DBE) demand level, was 0.004 in/in in the most highly strained portion of the shear wall at the base of the building. The design of a tall building is generally controlled by stiffness demands, requiring walls that are thicker and stronger than they need to be for strength purposes alone. For this reason, such a low strain demand is not surprising. However, the SPRP suggested that this strain was too low to be used as the test acceptance criteria, and required the value be scaled up to 4/3 of the Maximum Credible Earthquake (MCE) demand level, resulting in a value of 0.007 in/in. (The 4/3 increase on the MCE demand level was chosen to provide something analogous to the use of a median plus one standard deviation value expected to result from a series of non-linear time history runs.) Thus, the acceptance criterion was set at a value 75% higher than that required by the UBC.

In early 2007, each of the three test specimens were created and loaded as agreed. Concrete spalling was observed in the test region around 0.003 in/in strain. The WRG did not show any signs of failing when the test was completed at the prescribed 0.007 in/in strain level. The test specimens all showed a positive slope at the end of the test indicating that additional strength gain was probable.

The results from these tests were adequate to demonstrate to the SPRP and the City of San Francisco that the WRG product was acceptable for use on this project.

Three additional tests were performed prior to the official City tests on identical specimens when the concrete strength had reached 8,700 psi. DeSimone and the building owner wanted to understand how the WRG product would behave past the agreed upon strain level of 0.007 in/in. As such, these three test specimens were loaded to failure.

The specimens first began to demonstrate concrete crushing, which was apparent when cracks began to form and spalling initiated around strains of 0.0025. As the concrete within the testing zone dilated, the test results showed that yielding of the vertical bars and the WRG grids occurred next. Finally, when the ultimate strain was reached, the WRG welded intersection burst and the specimen lost their load carrying capacity (much like hoop rupture in a conventionally tied column.)

These tests demonstrated ductile behavior up to an ultimate strain of approximately 0.025 in/in, which was more than three times the acceptance criteria strain level. The strain levels achieved in this test match well with other tests done on WRG (Giria, Saatcioglu 1996) and other tests done on conventionally reinforced high strength concrete specimens (Bing, Park, Tanaka, 2001).

The concrete specimens all exhibited ductile behavior and the performance of the WRG system was deemed a success, however, we were still at a loss to explain why the welded intersections perform so poorly when tested in-air. During the welding process, the two intersecting pieces of steel melt into each other a small amount creating a mechanical anchorage. We hypothesize that it is this mechanical anchorage between intersecting pieces that make the WRG system perform so well when confined in concrete. When the test is performed in-air, there is no restraint to hold the mechanical anchorage in place. However, this is also an area in which further research is recommended.

Please see Figure 11 for the results of all tests.

Ground Level Porte Cochere: Sloping the SMRF Column Out of the Way

At the ground level of the south side of the building, the project architect required a porte cochere having a minimum width and height for car clearance. However, these requirements were wider than the available clearance provided by the SMRF's. Therefore, we were challenged to find a way to take a two high-rise SMRF's and transfer one column about 18' to the West and another one 15' to the East from level 3 to the basement floor.

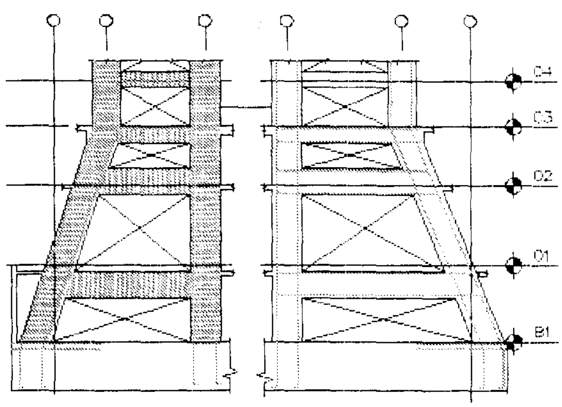


Figure 12: Sloping SMRF's from Level 3 to the Basement Foundation

While the slope of the two frames is not that great, 55 stories of eccentric column load transferred from the building above created a horizontal force component of significant magnitude. Fortunately, the frames produce horizontal loads in opposite directions under gravity loads. This gravity horizontal force component was resolved by building up the slab between the frames at level 3.

The real challenge was addressing the seismic overturning moments in each frame. The reactions at level 3 were determined by performing a response spectrum analysis under the 25% base shear model. In order to increase the confidence in the performance of the sloping frames, a capacity design methodology was used to resolve the horizontal earthquake force components. The code level horizontal components were increased by the over-strength factor and then the lower portion of the frame was redesigned to accommodate the additional loads.

Conclusions

While cast-in-place concrete has been the favored material for residential projects in many parts of the country for years, it has recently become cost effective for high rise projects on the west coast, even in areas subjected to the highest seismic demands. However, as these buildings are extended to new heights, stronger materials are required and extra steps need to be taken to insure proper consolidation. Welded Reinforcement Grid and structural steel link beams are two practical ways to reduce costs, speed construction, and simultaneously achieve the goal of better consolidation.

While this project will be the tallest reinforced concrete building in California at the time it is completed, it is likely to be surpassed within a matter of a few years. As buildings like it push further and further skyward, engineers will undoubtedly seek to use stronger materials. However, as this project has demonstrated, current code provisions regarding confinement for compression elements appear to limit the feasibility to about 10 ksi. Further research may be required in this area to reduce confinement steel requirements in order to allow the use of stronger concrete.

Acknowledgements

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References

ACI 318, 1999, Building Code Requirements for Structural Concrete and Commentary, *American Concrete institute, Farmington Hills, MI*

AISC, 2002, Seismic Provisions for Strucrtural Steel Buildings, *American Institute of Steel Construction, Inc.,Chicago, IL.* Part II

Bing, L., Park, R., Tanaka H., 2001, "Stress-Strain Be[illegible] of High-Strength Concrete by Ultra-High- and Norma[illegible]-Strength Transverse Reinforcements," *ACI Structur[illegible] Journal,* Vol. 98, No. 3, May-June 2001, pp. 395-4[illegible]

Grira, M., Saatcioglu, M., 1996, "Concrete Columns Confined with Welded Reinforcement Grids," Rese[illegible] Report, *Department of Civil Engineering, Univers[illegible] Ottawa, Canada,* Report OCEERC 96-05, Harries, K. A., Gong, B., Shahrooz B. M., 2000, "Behavior and Design of Reinforced Concrete, Steel, and Steel-Concrete Coupling Beams," *Earthquake Spectra*, November 2000, Vol. 16, number 4, Earthquake Engineering Research Institute, Oakland, CA

ICBO, 1997, *Uniform Building Code, Structural Engineering Provisions*, Vol. 2, 1997 Edition, International Conference of Building Officials, Whittier, California.

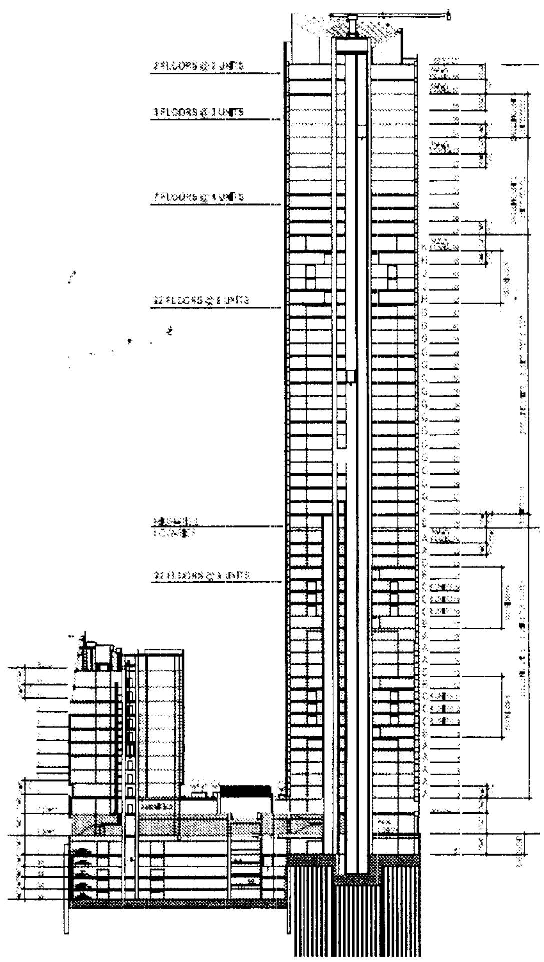

Figure 13: Building Section