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Wilshire Grand: Outrigger Designs and Details for a Highly Seismic Site

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Abstract

The 1100 foot [335 m] tall Wilshire Grand Center tower under construction in Los Angeles illustrates many key outrigger issues. The tower has a long, narrow floor plan and slender central core. Outrigger braces at three groups of levels in the tower help provide for occupant comfort during windy conditions as well as safety during earthquakes. Because outrigger systems are outside the scope of prescriptive code provisions, Performance Based Design (PBD) using Nonlinear Response History Analysis (NRHA) demonstrated acceptability to the Los Angeles building department and its peer review panel. Buckling Restrained Brace (BRB) diagonals are used at all outrigger levels to provide stable cyclic nonlinear behavior and to limit forces generated at columns, connections and core walls. Each diagonal at the lowest set of outriggers includes four individual BRBs to provide exceptional capacities. The middle outriggers have an unusual 'X-braced Vierendeel' configuration to provide clear hotel corridors. The top outriggers are pre-loaded by jacks to address long-term differential shortening between the concrete core and concrete-filled steel perimeter box columns. The outrigger connection details are complex in order to handle large forces and deformations, but were developed with contractor input to enable practical construction.

Keywords: Tall building, Core, Outrigger, Belt truss, Buckling Restrained Brace (BRB), Nonlinear Response History Analysis (NRHA), Performance Based Design (PBD), Seismic design

1. Background

The Wilshire Grand Center includes a 1100 foot tall [335 m] mixed-use tower, currently under construction in downtown Los Angeles, California, that will be the tallest US building West of Chicago upon completion in 2017.

Even though it's located in the 'City of Angels,' when it comes to building design the saying, 'The devil is in the details' still applies. The project occupies a full city block, replacing a 900-room hotel previously on the site. Based on local zoning rules, owner/developer Hanjin/Korean Air was permitted to replace those 900 rooms and build an additional 400,000 square feet [37,000 m^2] of leasable office space, plus required parking spaces. After considering various building massing and phasing options, the design selected was a single 73 story tower placed at the prominent corner of Wilshire Boulevard and Figueroa Street, with hotel functions occupying the upper 2/3 of its height, office space below, and parking in a five-story-deep basement covering the whole site. Consistent with Korean Air's goal of providing a high-end hotel experience for travelers, the floor plans and core layout were optimized for hotel functions. That favored a relatively long and nar-

Figure 1. Rendering of the Wilshire Grand tower by architect AC Martin. Note belt trusses, expressed at bottom outrigger level and behind glazing at top outrigger level. Courtesy AC Martin.

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Figure 2. Typical hotel floor plan showing rooms arranged around a narrow core. Courtesy AC Martin.

row core encircled by single-loaded corridors and guest rooms following defined modules, as shown in Fig. 2.

At office levels, the floor area was modestly increased to provide more desirable leasable space by shifting columns outward on one long tower face and by sloping columns at the narrow ends of the building. The combination of building height and core plan proportions meant that in the narrow direction the core alone would be too slender and flexible for practical or efficient lateral stiffness and strength.

The building illustrates many key outrigger design and construction issues outlined elsewhere (Choi, 2012). Outriggers consisting of steel trusses extending horizontally from the cast-in-place reinforced concrete core to perimeter columns were proposed to improve building transverse

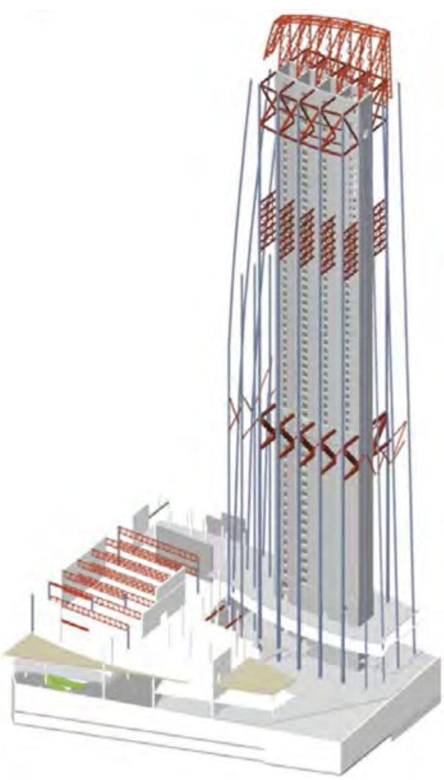

Figure 3. Isometric showing outriggers at three levels. Courtesy AC Martin.

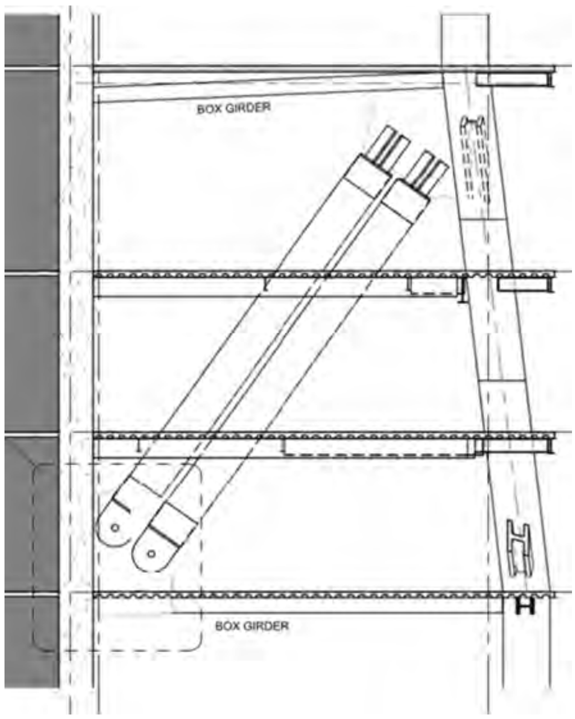

Figure 4. Bottom outrigger extending through three stories.

stiffness, shown conceptually in Fig. 3. At three discreet locations in the tower, the outriggers occur on five column lines to link five core 'web' walls to ten perimeter columns. The bottom outriggers are three stories tall and extend from Level 28 to Level 31 through office space which prominently displays the dramatic members' presence as well as through mechanical equipment space, as shown in Fig. 4.

The middle outriggers occur roughly 2/3 of the way up the tower and extend vertically through six stories, from Level 53 to Level 59. These outriggers form 'braced Vierendeel trusses,' which are concealed within the walls separating hotel rooms. To keep the hotel corridor clear, 3 foot deep steel girders extend from the outrigger braces to the concrete core wall at each floor level, shown at left in Fig. 5. While steel posts share forces among girders, some posts are omitted to minimize restraint of core long-term shortening within the outrigger height.

The top outriggers extend from the Level 70 hotel 'sky-lobby' up to the Level 73 roof deck. They are exposed and architecturally featured for dramatic effect, as seen in the lobby rendering, Fig. 6. Steel belt trusses at the bottom and top outrigger levels, visible in Figs. 1 and 6, link the ten outrigger columns to all perimeter tower columns. By engaging all twenty perimeter columns, the stiffness of the lateral load resisting system is maximized, and differential vertical movements between columns are minimized. Belt trusses also act as 'virtual outriggers' reducing tower deflections in the long direction. The tower columns typically

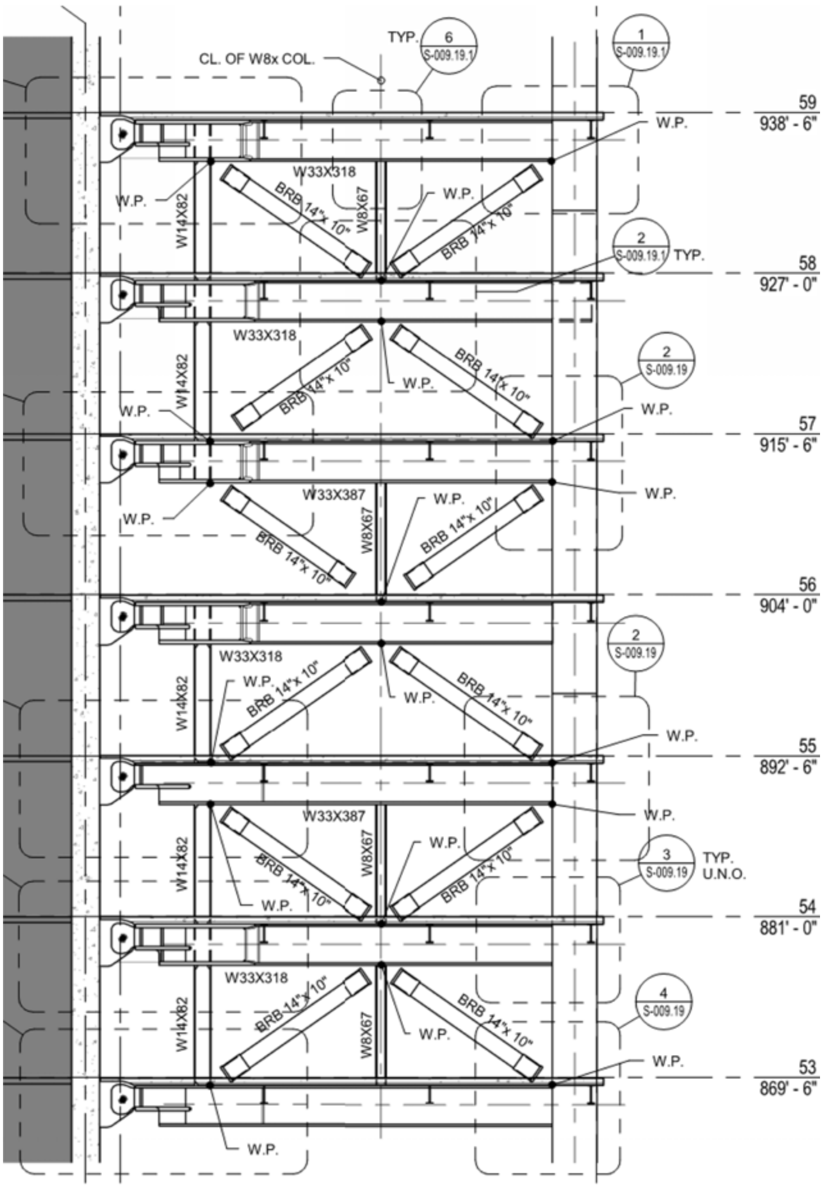

Figure 5. Middle outrigger uses deep girders to cross corridors at left. Beams are notched at corridors to pass utilities. Pins near column not shown. Courtesy Brandow & Johnston.

Figure 6. Top outriggers and belt trusses in the Level 70 hotel lobby. Rendering courtesy AC Martin.

consist of welded steel plate boxes which are filled with high strength concrete after erection. Column outer dimensions and box plate thicknesses reduce with height to reflect smaller demands and to maximize usable space. The steel box and concrete fill work together to provide axial stiffness when in compression for both gravity and lateral load cases, while the steel box alone acts to resist tensile forces during extreme wind or seismic events.

2. First Steps of Analysis and Design

If following conventional prescriptive building code requirements, a tower of this height would require a dual system, such as a central Special Shear Wall core plus a perimeter Special Moment Frame. Such an approach was found to be undesirable and impractical for this project. Undesirable, because architect AC Martin wanted floor-to-ceiling windows, and the deep spandrel beams of a perimeter moment frame would block part of the view. Impractical, because a dual system does not provide the structural efficiency of a core-and-outrigger scheme; significantly thicker core walls, columns and moment frame beams would be needed to meet lateral stiffness goals for occupant comfort and story drift serviceability. While a core-and-outrigger system was determined to be the optimal scheme, it is not covered by prescriptive code requirements. The solution was Performance Based Seismic Design (PBD for short), an approach allowed within the building code. PBD uses Nonlinear Response History Analysis (NRHA) to simulate actual building behavior during realistic earthquakes, with acceptance criteria tailored to each type of structural element. This differs from prescriptive design that assumes good performance will result from specially-detailed members sized for artificially reduced seismic demands. Using PBD and NRHA is appropriate because outrigger behavior in an earthquake will depend greatly on the specifics of its design.

The design process began by sizing core walls, columns and outriggers for strength-level forces based on ASCE 7-10 wind loads, using 115 mph [51 m/s] 3 second gusts at 33 ft [10 m] for a 1700 year mean recurrence interval (MRI) based on the building's high occupancy. Dynamic properties were then checked for occupant comfort for a 10 year MRI, 63 mph [28 m/s] based on local wind climate data. With peak accelerations under 15 milli-g's at upper stories, the design met strict residential criteria, which is conservative for hotel rooms.

The preliminary tower design was then checked against linear elastic seismic loading assumptions to better estimate structural component demands. For overturning moments, the Response Spectrum Analysis (RSA) results were scaled up to provide base shear at least 85% of the base shear 'floor' equations in the prescriptive code, which governed considering a building period of 7 seconds in the transverse direction and local high seismicity. For shears, the scaled RSA results were then tripled to estimate Maximum Considered Earthquake (MCE) demands. This is because higher modes are major sources of story shear in tall, flexible buildings, a behavior that codes typically used for shorter buildings don't address well. These shear forces required thicker North-South 'web' walls of the tower core than prescriptive code formulas would indicate.

3. Buckling Restrained Braces

The outriggers posed a design challenge: a conventional outrigger diagonal sized for adequate capacity against buckling in compression would have considerable extra strength in tension. Connections to core walls and columns would then be sized to resist the largest possible tension capacity, a tremendous number. The horizontal components of that diagonal force would also induce a 'panel zone shear' force into the core 'web' walls at those stories which would greatly exceed shear demands anywhere else along the tower's height. Local conditions would drive overall wall thicknesses, or require costly and slow-to-erect embedded steel trusses within the walls at outrigger levels. Neither situation was desirable.

The solution was to use Buckling Restrained Braces (BRBs) for all outrigger diagonal elements. BRBs have a central steel plate core, carefully sized to yield at a specified value based on actual material properties, and shaped to maximize ductility by minimizing stress concentrations along the core and at connections. The core is coated with proprietary bond breaker material and slid through a shorter steel box or tube, which is then filled with concrete or mortar. Brace connections are made only to the core ends, not the surrounding tube. Tension forces on the brace that exceeding the core yield point cause it to stretch. Reversal of forces that put the brace core into compression cause it to 'squash' rather than buckle sideways, thanks to lateral stiffness provided by the concrete-filled steel tube. The resulting brace force-deformation relationship is a stable, repeatable and very well defined hysteresis loop. Loads on connections, columns and core walls were limited by establishing specific yield forces and considering the maximum forces that can develop when BRBs are strained past yield.

Each of the ten bottom outrigger braces contains four BRBs which function as a group to provide an 8800 kip yield capacity, because 2200 kips per BRB is about the maximum that has been fabricated and tested to date. No further full-scale laboratory testing was needed for this project. All four BRBs in a group have steel cores coming from the same steel 'heat' for matched yield behavior, like a well-matched four-horse team pulling a wagon together. The bottom connection for each of these BRBs is a high strength steel pin run through a clevis (fork) on the BRB and a locally reinforced gusset plate on the core wall, to provide an attractive appearance and compact size, relative to the huge loads involved. Fig. 7 shows the size of these connections.

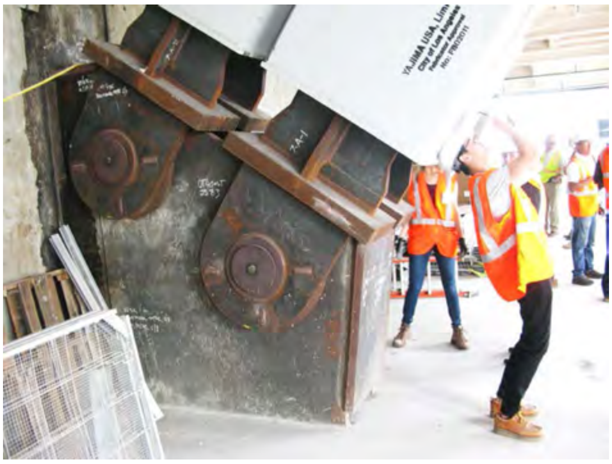

Figure 7. Quadruple BRBs pinned to gusset plates. Note visitors for scale.

The top connection for each BRB uses high strength bolts to connect to a gusset plate on a perimeter column as shown in Fig. 8. This provided some adjustability, since the BRB lengths are fixed during fabrication, and the column-to-core distance would be difficult to change during erection.

Figure 8. Quadruple BRBs bolted to gussets at upper connections. Note access to inner bolts.

Each of the ten middle outriggers uses 12 single BRBs in repeating X patterns, with each BRB yielding at 800 kips. BRB ends connect to small gusset plates through steel plate splices field welded for length adjustments, visible in Fig. 9. The BRB end connections were sequenced by Engineer of Record Brandow & Johnston to minimize the risk of damage from a large wind or seismic event acting on partially-erected middle outriggers.

Each of the ten top outriggers uses a single BRB with 2200 kip yield capacity, in a rectangular outer tube sized to be as slender as possible. The bottom end connection, visible in the hotel lobby, uses a steel pin through the BRB clevis and core-mounted gusset plate. The top connection, concealed within a mechanical equipment level, uses splice plates with long slots for field bolts to allow jacks to widen the distance between the BRB and the top gusset by about ¾” to preload the BRB to 1000 kips compression once construction has topped out, as partial compensation for future BRB tension forces that will be generated by gradual concrete core shrinkage over the building's life.

Figure 9. Single BRBs at middle outrigger use field welded splice plate connections. The red X shows a delayed connection.

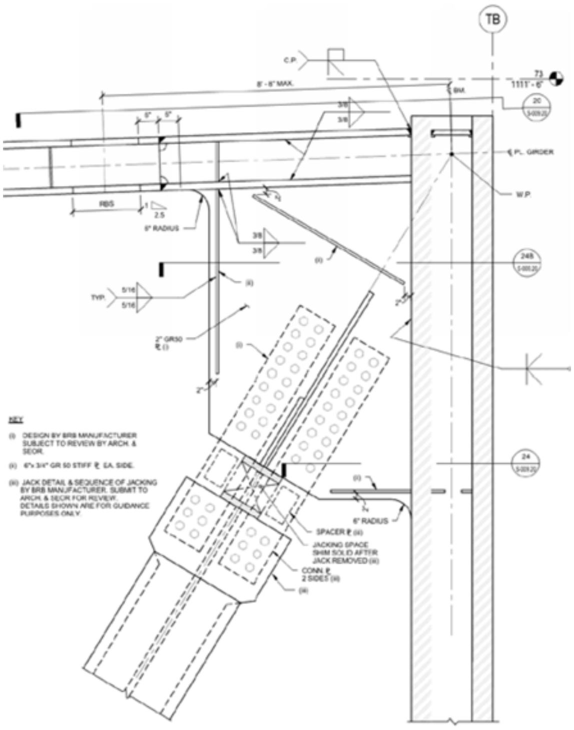

Figure 10. Top outrigger connection for jacking. Courtesy Brandow & Johnston.

After jacking, the bolts will be tightened for a friction connection.

4. Nonlinear Modeling

The commercial analysis program Perform-3D was used for determination of realistic deformations at displacement-controlled ductile elements, and realistic forces at force-controlled non-ductile elements. Core walls and piers used distributed fiber elements that reflected contributions to flexural strength and stiffness from both compression-only concrete and tension-or-compression vertical steel reinforcement, including steel yielding. Core wall shear stiffness was considered elastic, reduced from the theoretical value to reflect softening from cracking in a major quake. Coupling beams over core doorways are relatively deep compared to their span, so shear-beam model elements were appropriate. Building behaviors and member forces were affected very little whether columns were modeled using transformed areas or using separate fibers for steel and concrete. Most tower floors were modeled as rigid diaphragms for computational speed, with semi-rigid diaphragms used near outrigger and belt truss zones where load paths would be affected by diaphragm flexibility. The 'backstay effect' of basement floors and walls was considered by having three tower models bracket the likely range of interaction effects, with computationally efficient 'dummy frames' used in both directions to provide lateral stiffness based on unit-load deflections found by a separate basement model. Belt truss 'virtual outrigger' forces were bracketed by varying local diaphragm properties. BRBs were first modeled as steel elements with bi-linear properties in both tension and compression based on a set of reasonable assumptions. Once the BRB supplier was engaged, the property curves were slightly adjusted to reflect their proposed member geometries and lab test experience.

Geometry at the middle BRB was rather complex, due to offsets of work points and member hinge locations from member centerlines. Reflecting that level of detail within the main building model was not appropriate, efficient or necessary. Instead we created a detailed model of one outrigger tied to a core wall, and tuned simpler centerline elements in the full model to match its behavior. Pushover analysis of the detailed model was also instructive for connection designs: thanks to multiple large X's and posts linking floors, load paths change significantly as some BRBs yield before others. Designs of connections at posts, at core embed plates and at column gussets used the maximum forces found at any point in the displacement cycle, as shown in Fig. 11.

Demand inputs for the nonlinear model consisted of 11 sets of digital seismic time history pairs, selected for appropriateness to the range of local seismic hazards, and spectrally matched to site-specific Maximum Considered Earthquake (2475 year MRI) spectra in Fault Normal and Fault Parallel directions. The time history pairs were oriented so that 6 were in the higher Fault Normal direction for building transverse loading.

5. Findings and Design Verification

Tower base reactions found made sense: shear forces were higher in the longer, stiffer direction under elastic

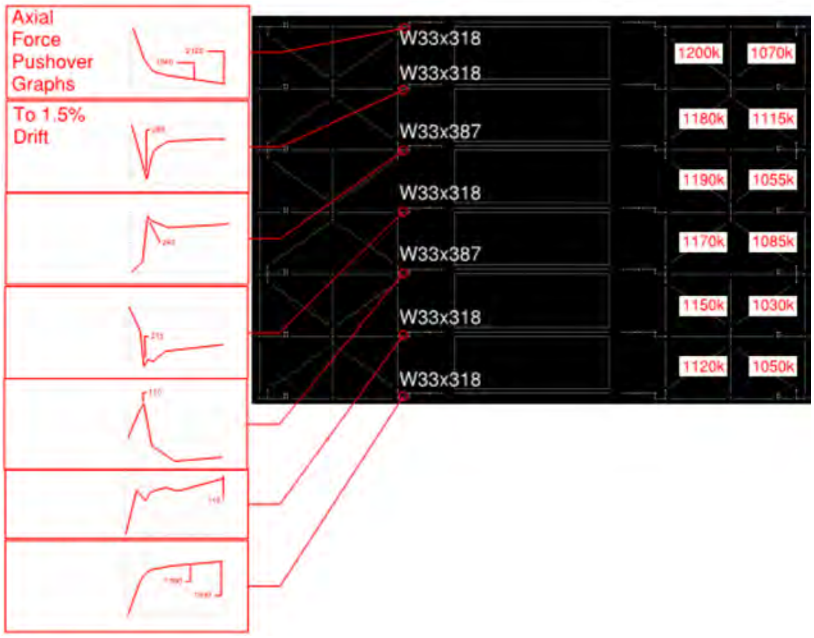

Figure 11. Middle outrigger girder axial forces vary during pushover. Note maximum forces are not all at maximum drift. Maximum BRB forces are shown at right.

MCE (scaled RSA shear forces), but dropped in the nonlinear MCE model as coupling beam yielding softened the response to many modes of excitation in this direction. No drop in shear forces was seen in the transverse direction since the 'web' walls are solid and higher modes don't yield the outriggers. Mean nonlinear MCE base shear is roughly triple 'code minimum' shear.

Base overturning in both directions shows large reductions from elastic to nonlinear MCE cases, thanks to beam yielding in the long direction and outrigger BRB yielding in the transverse direction. MCE base moments are of similar magnitude to 'design' RSA results scaled to code minimum base shear.

Overall building behavior was well within acceptance criteria established in the Los Angeles Tall Building Structural Design Council PBD Guidelines (LATBSDC, 2011)

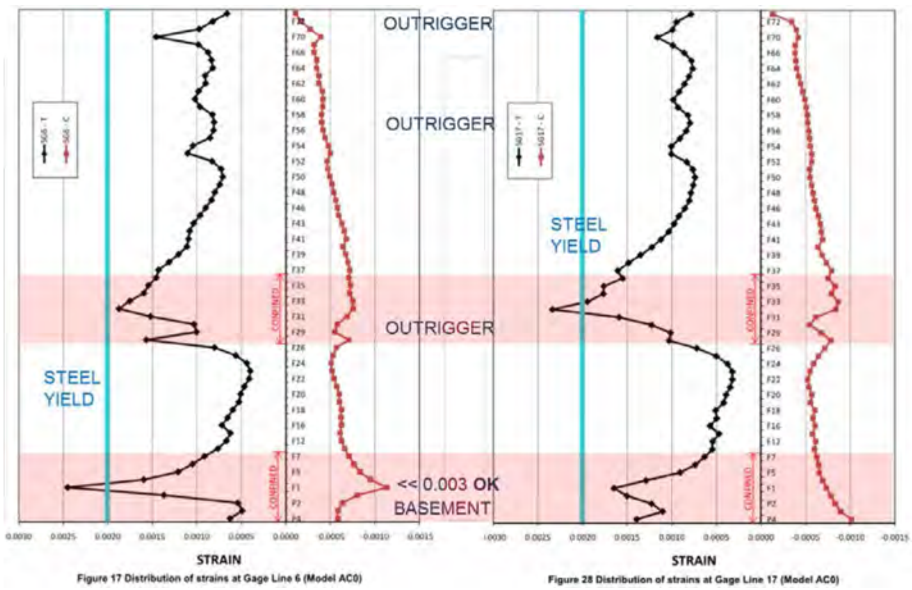

Figure 12. Mean MCE strains at two core locations show local peaks at potential hinge zones where confinement is provided.

and accepted by the Peer Review Panel retained by the LA Building Department, a requirement under the guidelines. MCE story drifts at mean and 84th percentile levels are roughly half the 2.4% limit for a high-occupancy building. Rotational demands for the coupling beams are within the performance limits indicated from literature on recent lab testing. Core wall strains show peaks at anticipated hinge zones above the basement and above the stiff bottom outrigger, as seen for two core locations in Fig. 12. Vertical core reinforcement required for wind strength shows seismic strains just slightly exceeding yield in these peak zones, well within acceptance criteria. Even though concrete strains are well below crushing limits, boundary element confinement reinforcement is provided at the two hinge zones for improved ductility.

Ductility demands on the outrigger BRBs, based on the mean of MCE peak strains in 11 time histories, tell an interesting story in Fig. 13. Expressed as a factor on yield strain, the bottom and top outriggers show small tension strains and much larger compression strains. This effect reflects the basic flexural behavior of the core: under seismic flexure the neutral axis will be close to the core compression face, where columns feel compression through BRBs acting in tension. In contrast, the distance from the core's neutral axis to the tension face and outrigger tension columns is much larger, meaning tension columns are 'working harder,' through BRBs acting in compression. The middle outrigger has BRBs as X's, so tension and compression effects are intermixed. Because the middle BRBs have yield lengths that are a smaller fraction of overall lengths than the bottom and top outriggers, they experience larger strains even for wracking displacements similar to those at the other outriggers.

At force-controlled elements, which have little ductility, the mean of 1.5×MCE peak demands for each time history was compared to expected capacities. Unsurprisingly, the core web wall shear forces are high but acceptable, and the effect of outrigger forces at core wall 'panel zones' is quite apparent in Fig. 14.

Demand/capacity ratios at the perimeter columns are also acceptable, being well below 1.0. Small net tension steel stresses, except at outrigger/belt locations, allowed for using economical partial penetration welds on some of the box column plates.

6. Differential Shortening

By design, outriggers provide stiff connections between different building systems. Even if connected vertical elements use the same elastic material, such as steel braced bays and steel columns, outriggers will end up transferring some of the gravity loads between elements based on differing stiffnesses and tributary loads. The amount of load transfer depends on the timing of outrigger construction; waiting until building top-out would minimize the transfer force, but that is rarely practical. Where connected vertical elements also have time-dependent deformations, such as concrete core walls and concrete columns, additional force transfers relate to differences in concrete mixtures, reinforcing ratios, volume-to-surface area ratios and exposure, in addition to gravity loads and construction timing. The Wilshire Grand tower posed an additional outrigger design challenge by connecting reinforced concrete core walls to concrete-filled steel box columns. The core walls will shorten slowly, but significantly, over the building's life due to creep and shrinkage effects, while the columns will creep and shrink much less because much of the load is carried by steel plates, and because creep and shrinkage of

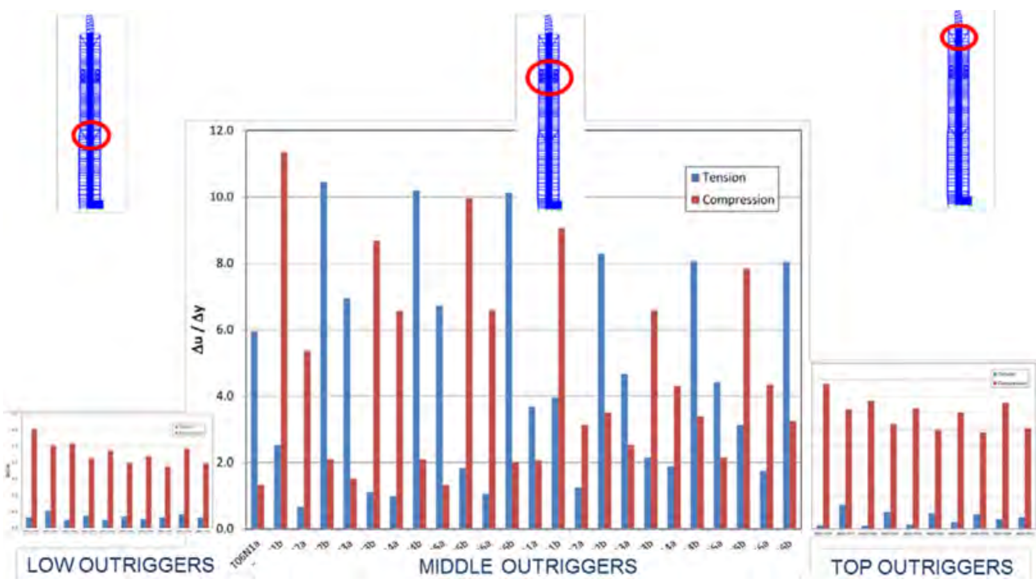

Figure 13. Displacement demands on BRBs vary by location. Note asymmetry between tension (smaller) and compression.

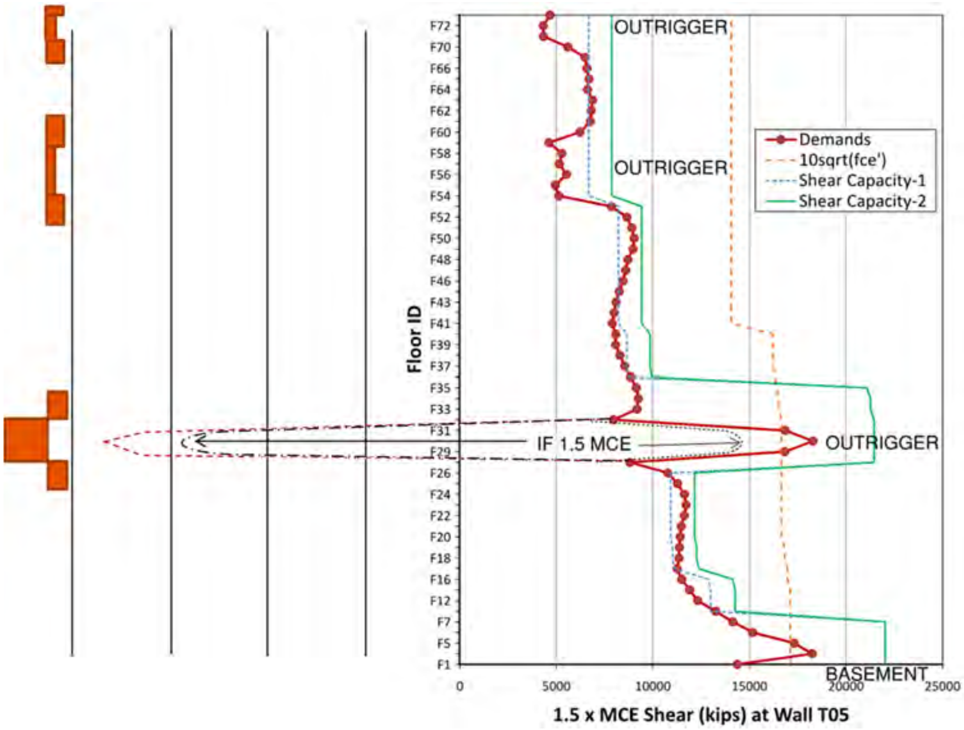

Figure 14. Wall shears at 1.5×MCE show 'panel zone' effects from outrigger force couples, reducing or even reversing wall shears as indicated by the core shear diagrams at left. Curves to left reflect reversal seen under static loading (solid curve to right shows 'directionless' results from dynamic analyses). Shear under mean of 1.5 MCE (inner curves) is less than that under 1.5 times mean MCE, showing force-limiting effect of BRB yielding.

the concrete fill is greatly reduced by being sealed inside the welded steel box columns.

The challenge of differential shortening was recognized at the time the structural system was selected, and was addressed six ways.

1) The age of core wall concrete testing was specified by the structural engineers and extended to 90 days rather than 28 or 56 days. This permitted use of low-water, lean cementitious material blends with lower shrinkage properties.

2) Numerous trial batches for shrinkage were performed by concrete supplier Catalina Pacific, to strike a balance between minimizing shrinkage and providing more economical concrete placement. While concrete mixtures using local Los Angeles area cements and aggregates are known for high shrinkage, including a Shrinkage-Reducing Admixture (SRA) lowered the predicted values to acceptable levels. This allowed local materials to be utilized in the concrete. Modified 28-day shrinkage tests by testing agency Twining used 4" [100 mm] square prisms to identify promising mixtures for further study. These prism tests were extended well past the normal 28 days to check that SRA benefits continued as drying continued, as this information was not in previous test data.

3) A formal, year-long creep and shrinkage testing program was run for the two selected concrete mixtures, one with 8000 psi [55 MPa] minimum concrete strength for the lower half of the tower core walls, and the other with 6000 psi [41 MPa] minimum strength for the upper core walls. Sealed and unsealed sample cylinders, loaded and unloaded, allowed determination of separate drying and creep effects. Information on specific creep and shrinkage parameters was provided in periodic progress reports from testing laboratory WJE. Shrinkage prisms were also 'baked,' driving off all free water, to establish ultimate shrinkage limits.

4) Proprietary shortening prediction software based on the B3 model used creep, shrinkage and 'baked prism' data for calibrating the concrete shortening model. The influence of longitudinal reinforcing, including bars in the core walls and steel plates of the box columns, was reflected by an age-adjusted modulus approach.

5) Time-related strains were modeled in elastic ETABS and nonlinear Perform-3D computer programs. In ETABS, artificially applied thermal expansion or contraction of the concrete core reflected construction timing by offsetting outrigger forces that could not exist prior to making the BRB connections. The timing was based on the construction schedule from Turner, the general contractor. A separate set of artificial thermal strains simulated the creep and shrinkage strains from our prediction software; the analysis program then showed how these strains were changed by outrigger participation. As Perform-3D did not have a thermal strain feature, comparable length adjustments between outrigger levels were made by converting core elements at selected stories into linear elastic elements and ‘squashing’ them with artificial pairs of forces. Using this model we demonstrated acceptable performance of the overall building whether an MCE quake occurs soon after construction, with negligible forces from creep and shrinkage, or many decades later under large induced creep and shrinkage forces.

6) Construction-phase BRB jacking is specified in the design to mitigate long-term shortening effects. In a large seismic event BRBs will be cycled through strains well beyond yield, in effect ‘washing out’ strains from differential shortening developed to that date. But yielding of the BRBs should be avoided during wind events. Long-term differential shortening will induce only small strains in the bottom outriggers, leaving plenty of ‘spare capacity’ for wind forces. Shortening-induced strains would be larger at middle outrigger BRBs, but wind loads would increase strains in half the BRBs and reduce strains in the other half, so total wind-resisting capacity is unaffected by shortening. Middle outrigger stiffness is slightly affected by shortening: the nonlinear elastic-plastic force-deformation curve of outriggers becomes more ‘round shouldered’ since some BRBs yield somewhat earlier, and others later, than in the no-shortening scenario. The top outriggers are most affected by differential shortening between the core and the perimeter columns, as they try to bridge the difference by pulling up on the core and down on perimeter columns. If simply connected at top-out, the top BRBs would start with no axial force, and experience 2000 kips [8900 kN] of tension several decades later. In that case, a major wind storm late in the building’s service life could yield top BRBs, affecting tower stiffness and force distribution. By preloading the top BRBs with 1000 kips of compression at top-out, ‘spare capacity’ of at least 1000 kips is maintained throughout the building’s service life, which is sufficient to handle large wind loads without yielding.

7. Outrigger Design and Detailing

The final outrigger designs considered strength, stiffness and ductility requirements for diagonals and chords, and for their connections to core walls and columns. Loads and deformations from gravity, self-strain (creep and shrinkage), wind and seismic conditions were key, along with effects of anticipated construction scheduling.

Each bottom outrigger connection has paired gusset plates linked for lateral stability. Each plate receives two BRBs in over-and-under configuration, pinned at Level 28 and bolted below Level 31 as previously described. Outrigger chords follow the paired-plate configuration, being designed as steel plate box members with each web merging into a gusset. Diagonal BRB yielding generates significant chord end rotations as a rectangular outrigger bay is wracked into a diamond shape. To minimize strains from rotation, chords are as shallow as practical, 24” wide by 16” deep [610×406 mm] while still providing necessary axial strength. For ductile beam behavior when strain exceeds yield, chord box plate thicknesses meet seismically

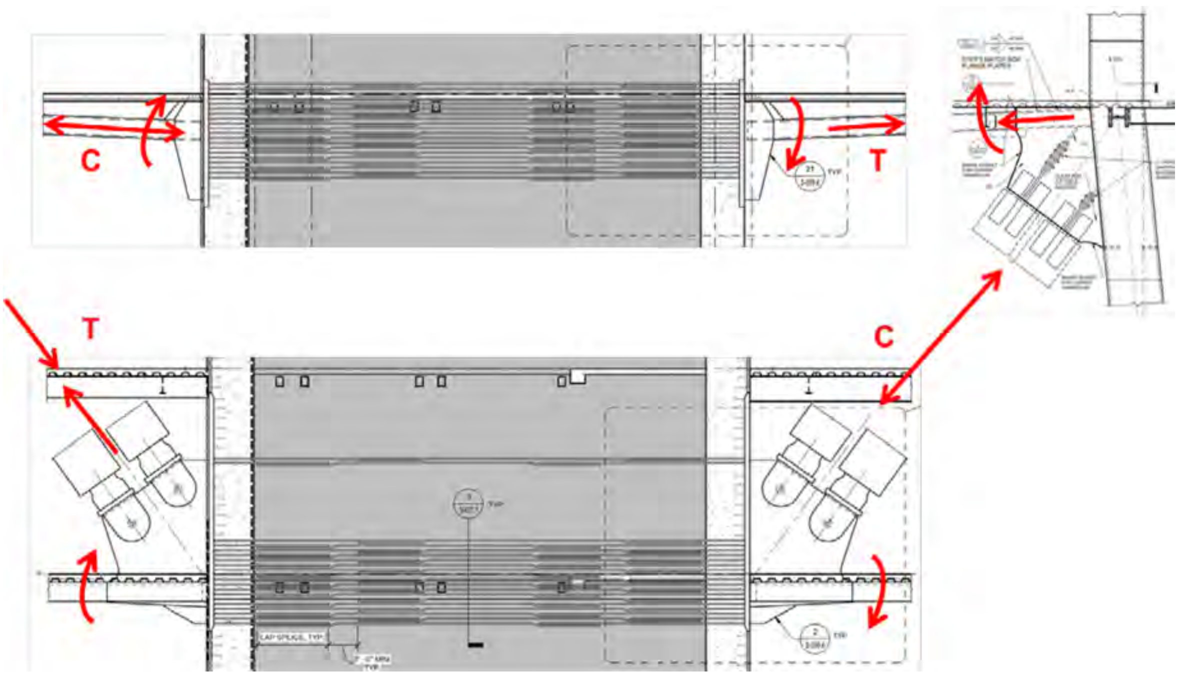

Figure 15. Simultaneous forces acting on bottom outrigger connections.

compact criteria. Chord ends and gusset plate intersections are also shaped to encourage hinging away from critical welds. The gussets themselves were checked for stability against buckling by SIE, Inc., the engineer for BRB subcontractor Mitsui Nippon, under simultaneous maximum anticipated BRB axial force, corresponding chord yield moment, and out of plane forces from seismic acceleration and interstory drift. See Fig. 15. Connection design forces were based on 1.1 times peak, but at least 86th percentile, MCE BRB axial forces, and 1.1 times corresponding chord plastic moments with Ry overstrength factor. At the bottom BRB, the forces are very large: BRB gussets connect to core walls by field welds to embedded steel plates with more than 100 #9 Grade 60 [29 mm, 414 MPa yield] reinforcing bars along the core web wall for the horizontal component and local moments, and hundreds of 7/8” [22 mm] diameter headed shear studs distributed along the 36 ft [11m] tall embed plate for the vertical component. The perimeter detail is simpler, with paired gussets, extending from box column faces, that have reentrant corners shaped to minimize stress concentrations. The box girder top chord connects to the core with a separate embed plate having a similar number of reinforcing bars but fewer shear studs.

Middle outrigger connections posed different challenges. Stiff, 36” [900 mm] deep girder chords are needed for effective Vierendeel action across the hotel corridors, so outrigger wracking would generate impractically large moments, shears and girder strains if rigidly connected to core walls and columns. Instead, true pins are used at brackets off core walls and columns, seen in Figs. 16 and 17. This enables maximum connection shear forces and moments to be controlled by well-defined BRB behaviors. Vertical W8 and W14 [200 and 360 mm wide] posts for sharing vertical forces among the seven girders are shallow enough that local yielding from direct connections is acceptable.

Top outrigger connections use single gusset plates stiffened for lateral stability, with BRBs pinned at Level 70 and bolted under Level 73 as discussed above. To meet the single gussets, chords are 16” [406 mm] deep I-shapes built up from steel plate; the web merges with the BRB gusset and flanges meet gusset stiffeners. For ductile behavior under large inelastic rotations the chord flanges have Reduced Beam Section or ‘dogbone’ shape cutouts and local stability bracing. Core connection design is similar in concept to the bottom outrigger, but for forces that are proportionally smaller based on peak MCE forces in the single BRBs. Column connection design is complicated by the need for shortening-compensation jacking at the BRB as discussed above and seen in Fig. 10.

Figure 16. Middle outrigger girder pin at core wall.

Figure 17. Middle outrigger girder pin at column.

8. Conclusions

Core-and-outrigger structural systems are becoming increasingly popular for tall building construction, providing gravity and lateral strength and stiffness efficiently without view-blocking perimeter moment frames or braces. Treatment of a core-and-outrigger system in a high seismic zone is challenging because this system is not covered under prescriptive code provisions. In addition, outrigger forces can be very large and difficult to transfer to other elements. Performance Based Design with Nonlinear Response History Analysis can be used to understand outrigger behavior and verify acceptability of a design. Where outrigger member and connection forces would be excessive in a linear elastic system, forces can be limited and predictable ductile behavior provided by replacing key elements with Buckling Restrained Braces. Outriggers are also affected by differential shortening between the core and perimeter columns, making this an important behavior for study and design. As it includes nonlinear modeling, BRBs, shortening effects and special detailing, the Wilshire Grand tower rising in Los Angeles provides excellent contemporary illustrations of all these points, setting a precedent for future tall buildings in high seismic zones.

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