Building to Last

By Nathan T. Charlton, P.E.

One of the most unique projects being completed for the next century is the Church of Jesus Christ of Latter-Day Saints Conference Center in Salt Lake City. The 1.5 million sq. ft. complex is located one block north of Temple Square in downtown Salt Lake City, and occupies an entire 10-acre city block. The new complex includes a 21,000-seat theater/conference space, often referred to as the assembly building, a 900-seat regional theater and a 1300-car underground parking structure building.

The assembly building is a 600,000 sq. ft. structure in the shape of a quarter circle. The center stage is focused around two steel-framed balconies with terrace seating and two sloping cast-in-place concrete orchestra levels. Program requirements for theatrical elements include catwalks for light support, fly galleries for theatrical rigging and interstitial spaces within the roof truss depths for lighting control equipment. Five stories of support areas outside and adjacent to the main hall contain entrance lobbies, escalators and elevator lobbies, restrooms, mechanical spaces, broadcast and translation program space.

Given the unusual size required by the building program, the challenge was to produce a solution that would honor the physical, historical and spiritual significance of the Temple and its buildings. The solution, presented by Zimmer Gunsul Frasca Partnership of Portland, OR, was to transform the roof through landscaping by creating a series of terraces that respond to the sloping site including indigenous tress and shrubs. Furthermore, the four-acre rooftop incorporates planted gardens, plaza paved areas of granite stone with water pools and tunnels and skylights to admit natural light into the main lobbies and main hall. The stone mesas and vegetation of southern Utah provided inspiration for this natural design solution.

Three primary levels of the assembly building extend outdoors onto continuous terraces that offer dramatic views of Temple Square, downtown Salt Lake City and the mountains beyond. The building exterior incorporates granite from nearby Little Cottonwood Canyon, the same material used to construct the Temple, while the interior finishes feature light colored woods, stone tiles and carpeted floors.

Gordon B. Hinckley, president of the LDS Church, requested that the building be completed in time to hold the first general conference of the new century. The preliminary structural design commenced in the fall of 1996, and the general construction contract was awarded to Legacy Constructors, a joint venture of Okland Construction, Jacobsen Construction and Layton Construction, in early 1997. With the contractor's scheduling input and structural steel mill rolling dates provided by Schuff Steel, requirements for phased permitting, material procurement and construction were shaped. Ultimately, the general contractor would have 17 months to complete the construction of the assembly building after the issuance of the 100 percent construction documents.

Structural Design Challenges

The Church's program requirements dictated that the 21,000-seat assembly space be column free. The structure's quarter circle geometric shape required long span roof trusses up to 290' long. Furthermore, the roof top landscaped required trusses to be designed for superimposed loads between 250 and 525 psf.

The City of Salt Lake zoning and planning code limited the building height. This height limit required the building foundation to be "pushed" down to a maximum of 90' below existing exterior grade. Resisting external static and dynamic soil pressures at this depth became a monumental design and construction task. To further complicate the design challenge, all sides of the building are adjacent to the property line and street right-of-way, preventing the use of permanent soil retention. Excavation shoring was held back from the exterior walls for 5' for water proofing, precluding the use of the shoring system in the final foundation wall design.

The Conference Center is designed to the requirements of the 1994 Uniform Building Code (UBC) which places Salt Lake City in seismic zone three. However, the owner stipulated that the Conference Center have a design life of at least 150 years. The Church's program required the structure be designed for seismic forces well in excess of current code force levels. This included force levels meeting or exceeding those for seismic zone four with an importance factor of 1.25.

The lateral force resisting system for the building had to resist large soil pressures imposed on the "buried" structure along with the forces generated by the building inertial mass. A finite element program (ETABS) was used to perform a dynamic analysis of the building to resolve the distribution of lateral forces. Perimeter concrete shear walls and interior concrete shear walls up to 30" thick resist lateral forces.

Much of the assembly building is supported by the adjacent below grade parking garage. The building column grid and garage grid are different due to differing program requirements. This resulted in transferring the majority of the building's structural steel columns. Because transfer beams could not exist in the garage due to limited headroom, building columns were transferred at various floor levels as many as four times. Plate girders up to 9' deep and transfer trusses were used based on web penetration requirements.

Foundation Design

The parking garage foundation extends to a maximum of 45' below grade while the assembly building foundation extends to a maximum of 90' below grade. Retaining walls and foundations for each were treated uniquely. Excavation shoring for the project site encompasses the entire site perimeter. A conventional tied-back soldier pile shoring system temporarily supported the excavation.

The garage's exterior walls vary in thickness from 10 to 14" depending on soil pressures and in-plane shear stress levels. The walls span horizontally between piling. The walls "hang" off the piling by headed studs to provide vertical and horizontal support. Piling "reacts" at each garage floor. Exterior walls span floor-to-floor in absence of soil dynamic load.

Concrete shear walls structure the assembly building on all sides. Walls at each side of the house, dubbed the east and north house walls, and the 660' (plan dimension) curved rear house walls stand over 100' tall supporting the roof structure above. The east and north house walls are set in off the perimeter building retaining walls by 30 to 60'. The space between the house walls and building exterior walls is used as mechanical space, storage, loading dock and access/egress stair volume. Cascading planter structures cap this space.

The final exterior wall design incorporates heavily reinforced concrete (24 to 36") buttress walls up to 60' in height at 30' on center, extending 30' into the building space. Exterior retaining walls span horizontally between buttresses and vary in thickness from 24 to 30" in thickness. The buttress walls top out at or near the adjacent exterior grade, reduce in thickness, and stair step up to the roof, framing planter trays. Buttress walls are founded on massive 6' thick mat footings.

The 660' curved wall at the rear of the house integrates pilasters for the ten long span roof trusses and forms one side of the four full height mechanical shafts, which help to resolve forces from the cantilevered balcony trusses. The corners of the mechanical shafts are heavily reinforced "L" and "Tee" shaped elements. The shafts extend to the lowest level of the garage and contain large fans for garage exhaust. Spread footings for the curved wall are as much as 20' wide and 6' thick, supporting seven levels of structure and long span trusses.


110 ft. cantilevered balcony.

Long-Span Roof Truss Design

Ten long-span roof trusses ranging in length from 230 to 290' "fan" the hall configuration and span from the rear hall wall to a 152' transfer truss over the stage. Two concrete columns support the transfer truss and roof trusses at each side of the stage.

The design of the roof trusses required the selection and optimization of structural steel for the support of high, complex landscape loading. Further complicating the design were architectural depth restrictions, a function of interior space sight lines and roof elevation.

The high roof loading and geometric restrictions precluded the use of single rolled shape chord and web members. Truss members would require shapes built up from steel plates and a combination of steel rolled shapes. Various truss designs investigated included: concrete filled, built-up plate box top chord, bottom chord external post tensioning and double trusses bolted together.

The final design included; ASTM A709, 70 ksi [485 MPa] plate built-up box top chords, ASTM A913, 65 ksi [450 MPa] double wide flange bottom chords and ASTM A 572, 50 ksi [345 MPa] double wide flange web members. Trusses were pre-assembled in the shop then disassembled and sent to the job site for field assembly. All joints were detailed as bolted connections to limit time consuming field welding. Tension control indicators ensured proper tensioning for all bolted connections.

Truss deflection and axial shortening (and lengthening) needed to be considered. A large percentage of the total landscape load would be placed after substantial completion of the roof framing. If the trusses were "locked in" without the ability to move, undesired internal stresses would be induced into the roof diaphragm. Unidirectional bearing assemblies with a maximum rotational capacity of .04 radians accounted for this required movement, allowing for truss rotation and maximum of 2" of movement along the truss axis. The ultimate bearing capacity was specified as 100,000 psi with a working stress limit of 5,000 psi. This 20:1 factor of safety helps to minimize fatigue, limit vertical deflection and provide for optimum performance of these irreplaceable assemblies. Connections of truss bracing members affected by deflections and movement were detailed to be welded after the majority of the roof load was applied.

A "match drilling" process fabricated the trusses. Chord members were stacked with gussets aligned and a drill press with 72" long drill steel "match drilled" all four flanges. This ensured proper field fit-up of chord and web members and avoid extensive material handling required for drilling of individual pieces. Three gusset plates were used for the connection of the webs to chords. Connections were bolted with staggered ASTM A490X bolts. The ten roof "radial" trusses varied in weight between 215 and 300 tons.

Balcony Seating Design

Design of the 34-cantilevered balcony seating trusses presented a significant challenge that would ultimately affect nearly every superstructure framing system of the new assembly building. The 7,500

Field bolting match drilled connections.

seat elevated balcony level follows the quarter-circle curve of the rear wall of the house and projects over structured seating levels below with a maximum 110' horizontal cantilever span. Steel cantilever trusses are approximately 20' on center. The top chord raker beams support precast concrete seating modules.

The balcony truss top and bottom chords align with the balcony and mechanical mezzanine floor levels outside the hall proper, forming a 21' truss depth. Truss volume between top and bottom chords is framed for the support of mechanical equipment. As the top chord of each truss pitches downward with the seating the truss depths decrease from 21' to a point of convergence 58' from the house wall. From this "spring" point the raker beams project out completing the cantilever.

Live load deflections at the raker beam tips controlled selection of the cantilevered framing members. By analysis, live load deflections were limited to less than one inch.

Welded connections of the precast seating to the rakers, rod bracing between every third bay of truss bottom chords and a continuous ring of framing beams at the cantilever tips help to provide stability to the rakers and resolve in-plane seismic forces.

Overturning moments from each balcony truss are resolved into a force couple between the top and bottom chords. The horizontal force from each truss transfers through the rear (radiused) wall of the house to the adjacent composite deck diaphragms at the balcony and mechanical mezzanine floor levels, accomplished with the use of heavy ASTM A572 W24x104 drag struts, which align with each truss chord. Struts extended into the adjacent diaphragm to sufficiently transfer the load through headed studs. Floor diaphragms react into four vertical full-height box shaped concrete airshafts and two hall side concrete walls. The box shaped airshafts are in themselves multi-span beams, which react into adjacent floors and the roof. The complicated load path and enormous total overturning force required unusually large steel to concrete connections where the trusses and floor diaphragms attach to the rear hall wall.

Horizontal forces from each truss top and bottom chord are transferred through the rear house wall by heavily welded connections to large embedded plates on each side of the concrete wall. A pattern of one inch and one and one half inch high strength bars join together each assembly of plates. Bars "punch" through the plates and are welded with partial penetration groove welds. This "sandwich" connection serves to transmit each truss top and bottom chord force to adjacent W24 drag beams. The drag loads are in turn transferred to the structural diaphragm. Ten primary collector girders at the floors adjacent to the balcony trusses resolve the diaphragm reactions and transfer them to four large concrete mechanical shafts and the north and east house walls. This complicated load path proved to be the most efficient way to resolve the enormous cantilever couple forces from the balcony framing. A partially redundant load path was created to transfer cantilever truss loads to the adjacent floor diaphragms (on the lobby side of the radiused wall) by using diaphragm reinforcing steel coupled through the radiused wall and headed studs on beam framing to transfer forces. A second partially redundant load path was created with the integration of a heavily reinforced corbel in the curved wall at the balcony top and bottom chord levels to serve as radial (arching) tension and compression members.

The cantilevered balcony truss analysis was further complicated by the lack of a continuous floor plate diaphragm between the four concrete shafts at the truss bottom chord (mechanical mezzanine) level. This required a three-span horizontal steel in-plane truss in the plane of the bottom chords to resolve forces in the absence of a continuous diaphragm. This 26' deep truss follows the rear wall of the house and reacts into the four box concrete mechanical shafts. Heavy W14x90 – W14x257 wide flange members form the truss diagonals and inside chord. The concrete radial corbel forms the opposite chord member. Truss node ASTM A572 gusset plates, weighing in excess of 3000 lbs., located on the top and bottom of each joint are bolted to provide member connection. Member gussets at the concrete radial house wall are welded to massive embed plates up to 9' long to resolve in plane truss forces.

Structural Diaphragm Design

Fourteen different floor and roof deck structural diaphragm types were used in the Conference Center structure to resist soil, seismic and wind lateral loads and the couple force created from the cantilever balcony structure. All diaphragms at floors

King truss— field welded splice of ASTM A709 30"x50" box - 2 1/4" plate compression diagonal.

and roofs consist of composite metal deck with concrete topping utilizing 1-1/2", 2" and 3" metal deck. Topping thickness vary from 2 to 16" and concrete topping strengths vary from 4,000 psi to 8,000 psi as a function of the diaphragm loads. Lightweight concrete, used as typical deck fill in most areas, provides the required two-hour fire rating. The code stipulates that lightweight concrete used to resist diaphragm shear forces be limited to 6,000 psi. In areas of high diaphragm shear normal weight high strength concrete was used.

A finite element model of the balcony, mechanical mezzanine and roof level diaphragms helped to better understand shear stress distribution. Primary stresses at the floor diaphragms are due to dead and live load overturning forces from the balcony trusses. Seismic stress also needed consideration. Large deck openings for escalators, elevators and mechanical shafts as well as reentrant building corner geometry greatly complicated the diaphragm analysis. Heavy diaphragm reinforcing resists balcony couple forces and limits creep from the sustained balcony cantilever load.

Conclusion

In conclusion, the Conference Center project is planned to stand well beyond the normal life span of contemporary buildings. This unique project has been given an unusually high degree of consideration during its programming, design and construction, and will stand for many decades.

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Structural Engineer:
KPFF Consulting Engineers, Portland, OR

Fabricator/Erector:
Schuff Steel, Phoenix, AZ

Software:
Dynamic analysis—ETABS
Diaphragm and long span truss analysis—SAP2000

Architect:
Zimmer Gunsul Frasca Partnership, Portland, OR

Construction: Legacy Constructors, a joint venture of Okland Construction, Jacobsen Construction and Layton Construction