Signature Roof Enhances Basketball Arena

A feeling of excitement permeates the design thanks in part to 65-ft. cantilevered overhangs at both ends of the building

By Karen Houghton, Ph.D., P.E.

The Rose Garden Arena is the most visible element of the new Rose Quarter development project located on the east side of the Willamette River across from downtown Portland, Oregon. More than just a new NBA arena, the project creates an entertainment district including the existing arena, outdoor public space, restaurants, office/commercial facilities, parking and bus/light rail transit facilities in a planned environment. The Rose Quarter is the first major example of a sports facility being used as the catalyst for the financing, planning and development of an urban planned district.

The Rose Quarter development, which opened in October 1995, was completed at a total cost of $262 million. As comprehensive as the project is, it is only the first phase of what is envisioned as a much larger development along the east bank of the Willamette River for commercial, residential and entertainment uses.

Signature Roof

The Rose Garden's roof symbolizes the energy and excitement inherent in an entertainment oriented building type. The simple curved form has its edges sliced in response to predominant street grids and view corridors. The 65-ft. overhangs at the north and south ends add a sense of tension and anticipation to the composition, heightening the pedestrian's awareness of the excitement to be found within.

The resulting roof design looks sophisticated while successfully achieving the intended architectural objective. However, the successful execution of this sophisticated "signature" roof created many exciting challenges for Ellerbe Becket structural engineers and the construction team headed by joint venture contractor Drake/ Turner. Structurally, the roof encloses the 375 x 575-ft. Rose Garden and has the capacity to carry not only the catwalks, an 80 kip retractable high tech video score board, various rigging load configurations associated with concerts and special events, and a patented acoustical cloud system; but also 80 future private suites.

The roof transfers gravity and lateral loads to the reinforced concrete superstructure of the arena below by utilization of a ring of steel columns and four steel Special Moment Resisting Frames (SMRFs), respectively. To make the structural design and construction even more challenging, the Northridge Earthquake occurred after the erection of the roof 's framing had begun but prior to its completion. The widespread reports of poor performance of SMRFs during the Northridge Earthquake caused much concern for all parties: Ellerbe Becket, the owner, city engineers, and the contractors as well. Unless a rational and reliable justification for utilizing the SMRFs could be established, the construction of the roof would be brought to a halt, resulting in costly delays and potentially redesigning the structural system all the way down to the foundation.

Structural System

The structural system of the roof utilizes ten one way bowstring trusses to carry the gravity loads of the 375 x 575-ft. Rose Garden enclosure. A series of propped cantilever secondary trusses framing into the outboard primary trusses support the 65-ft. roof cantilevers at the north and south ends of the arena. The cantilever areas at the northwest and southeast ends account for almost one quarter of the total roof surface area. Two levels of intermediate beams framing between the primary trusses top and bottom chords are required to carry the various loads associated with a

multipurpose arena. The intermediate beams framing into the top chord truss panel points provide bracing for the truss top chord and support the 3-in. 20 gauge acoustical metal roof deck, open web steel joist, and carry a portion of the future suites. The intermediate beams at the bottom chord support an extensive network of catwalks, acoustical clouds, and various rigging loads as well.

Open web steel joists with spans of 30 to 35 ft. are supported by the intermediate beams located at each truss top chord panel point and support the roof deck above. Due to the roofs curvature, Ellerbe Becket engineers elected to have the joists radially fabricated to match the 599-ft.-1-in. radius of the roofs curvature. The sloping curvature of the roof results in varying joist seat depths at 17½-in. deep to 2-ft. deep. The webs of the intermediate beams supporting the joists are orientated vertical; thus, all the joist seats are sloped in order to bear level on the top flange of the support beams. The 20 gauge 3-in. type N galvanized metal deck spans between joists in a perpendicular direction to the roof's curvature since it is bent about its weak axis.

A ring beam encompassed the outer perimeter of the ten primary trusses serving as a collector element for the outward thrust of the roof trusses. Early in the schematic design phase, Ellerbe Becket engineers conducted a cost study to determine if pot bearings permitting relatively free horizontal displacement of the trusses, would be a more economical support condition for the primary trusses. It was concluded from the study that pot bearings would not be a more economical support condition for the primary trusses as opposed to using a circumferential ring beam in conjunction with the flexural stiffness of the supporting columns.

In order to deliver the lateral loads of the roof to the concrete dual super structure below, Ellerbe Becket's engineers found that steel SMRF's best met the structural design requirements. The primary advantage of the SMRF system was that the R_w for seismic design was equal to that of the concrete moment frame and shear wall system below. It was found that four steel SMRFs, three bays each, controlled the roof drift and were capable of transferring the lateral loads to the concrete system.

The four SMRFs are positioned directly above the four full height shear walls of the bowl and lateral forces transferred from the SMRF columns into the shear wall by using a steel beam just above the base of the SMRF column. A plate is welded to the bottom flange of the beam and a WT welded to the plate. The flange of the WT is flush with the top of the concrete shear wall. The load path is completed via shear studs welded to the flange of the WT embedded into the top of the concrete shear wall. This beam assembly effectively resists the SMRF column base moments and shears and provides a transfer mechanism for the forces into the concrete shear walls, in lieu of a complex and congested column base anchor bolt assembly.

Fabrication and Erection

AISC-member Canron Construction Corp. performed all fabrication and erection of the structural steel. Detailing was done by, N.C. Engineering, Burnaby, British Columbia.

Since the fabricator/erection team was selected during design development, Ellerbe Becket

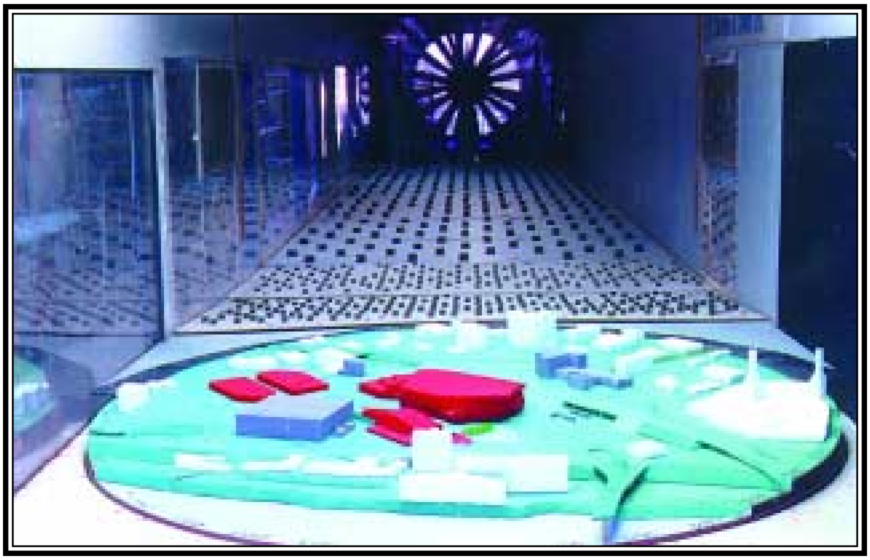
Engineers, Drake/Turner, and Canron were able to establish a team relationship unlike that which is typically achieved with such a large scale project. This early teamwork proved to be economically advantageous to the project. Input from all parties was utilized throughout the design process to reduce material, labor, and erection. This was achieved by discussing issues such as connection design, bolting versus welding, slip critical versus bearing bolt connections, availability of different steel grades and sizes, and Canron shop standards. The two most significant savings were achieved by using A490-X bolts in the primary truss connections and the utilization of 65 ksi steel for the bottom tension chords of the primary trusses.

At one of the preliminary meetings between Canron and Ellerbe Becket Engineers, it was widely agreed that a reduction in connection weight could be achieved if bearing bolts were used as opposed to slip critical with over sized holes. The construction issue associated with the lower erection tolerances of bearing bolts was easily resolved since Canron agreed to preassemble the trusses in the shop in order to ensure fit-up and avoid construction problems due to holes and bolts not aligning in the field. The use of A490-X bolts and resulting smaller gusset plate sizes reduced the connection weight of the primary trusses to 10% of the truss selfweight as opposed to 25% to 35% of the selfweight which is typically associated with connections utilizing slip critical bolts in oversized holes.

Since the partnering between Ellerbe Becket Engineers and Canron occurred early, the use of 65 ksi steel was considered since ample time was available for shipping the higher grade of steel. It was concluded that ample savings could be obtained by using 65 ksi steel for the bottom chord tension members of the primary trusses. While the

tension members could benefit from the higher grade steel, the top compression chord of the trusses would not see a reduction in weight due to buckling issues. Typically A572 GR-50 steel was used elsewhere. Angles and tubes were 36 ksi and 46 ksi, respectively.

After shop fit-up the trusses were disassembled, shipped, and reassembled in the field in an upright position in special racks fabricated by Canron. This upright position was required due to connection plates at panel points extending out to receive the top and bottom chord intermediate beams. Each truss was erected in two components with one component supported by a shoring tower while the second component was lifted into place. The construction sequence for the primary trusses started with the placing of the first two North outboard trusses and erecting the secondary trusses supporting the 65-ft. cantilever. When one end was complete, the construc-



tion process was repeated on the opposite end. For the remaining inboard trusses, the erection process continued to switch back and forth with the two trusses above center court being erected last.

The roof's edge geometry proved to be the most challenging task for all parties involved. The framing of the 65-ft. cantilevers at the ends and the curved longitudinal edges required extensive work between Ellerbe Becket Engineers and Architects. Although the basic concept of the geometry was established early in the design phase, the precise locations of the fascia, soffit, and gutter were not defined until late it the production of construction documents. In order to keep the design of the structural steel on schedule, it was necessary to set structural work points before all architectural work points were quantified. Main structural work points were established at panel points on the primary and cantilever trusses. Architectural work points were established relative to the structural work points using a computer program written by Ellerbe Becket Engineers. Approximately 1,000 points were calculated and provided to the steel fabricator in tabular form.

The task of taking the construction drawings and turning them into shop drawings was taken on by N.C. Engineering. N.C. Engineering worked with Ellerbe Becket Engineers and Canron to produce the shop drawings associated with the complex geometry.

Structural Analysis/Design

Ellerbe Becket Engineers performed a three dimensional static and dynamic analysis of the roof structure utilizing SAP90 Plus. The extensive analysis was required as a result of several factors: The overall system irregularity in seismic zone 3, a need to evaluate the dynamic effects of the relatively flexible steel roof structure supported by a stiffer and more massive concrete super structure below, the three dimensional effects of the secondary propped cantilever end trusses which are supported off the one way primary bowstring trusses, and the effects of unbalanced loading due to the rigging loads, catwalk, and the acoustical clouds. A conventional plane frame analysis would exclude the influence of out-of-plane member deformations and potentially lead to misleading results for such a complex system.

A wind and snow study was performed by RWDI, Ontario, Canada. A scaled down model of the entire Rose Quarter Project and surrounding city terrain was built by RWDI based upon the architectural/engineering drawings and field information. The arena, roof and other new structures were instrumented for the collection of data in the study. The wind pressures were evaluated by utilization of a wind tunnel imposing different wind velocities and directions upon the model. The snow drift study was conducted by submerging the model in a shallow tank of water and flowing a light weight sand across the model from different directions and velocities. The resulting roof pressures recorded from the two studies were used in the three dimensional SAP90 Plus model.

The final SAP90 Plus model included all primary and secondary steel members with their corresponding material properties. Load combinations utilizing gravity, wind, snow, seismic and temperature were evaluated. The results of the computer analysis were then used for the final design of the roof members following AISC LRFD design criteria. In compliance with the City of Portland, Oregon, complete connection design and detailing was also done by Ellerbe Becket Engineers. Ellerbe Becket Engineers worked closely with Canron to determine efficient connection schemes in order to minimize costs.

Primary Bowstring Trusses

The primary trusses supporting the roof span up to 375 ft. with a midspan depth of 33 ft. in order to carry a tributary width of 36 ft. Due to the elliptical shape of the arena, the truss spans varied resulting in 5 pairs of trusses or 10 total primary trusses. The trusses are cambered for dead load, 4 in. at midspan tapering to 0 in. at the supports.

The chord and web members consist of W14s oriented with the webs horizontal. The W14 compared with other wide flange sections is the most efficient compressive section, and by orientating the web horizontally, the unbraced length in the weak axis was effectively reduced by the truss web members at each panel point. In addition, connections at the panel points were easily constructed by using bolted sandwich gusset. The top chord of the primary trusses consisted of W14x257s at midspan reduced to W14x233s at the ends. The bottom chords are smaller, taking advantage of 65 ksi steel, resulting in W14x145s at midspan and W14x90s at the ends. It is interesting to note that, unlike a conventional truss with a constant depth, the chord forces do not reduce dramatically when the top chord is arched with respect to the bottom chord. This is due to the fact that the effective depth of the truss decreases at approximately the same rate as the moment.

Initially, the truss top and bottom chord splices were located away from the panel points in order to reduce excessive material that would result in extending the panel point gusset plates the required length of the splice plates. However, it was later concluded that for the resulting chord and web forces, the more economical connection entailed the splice points of the truss chords to coincide with top and bottom chord panel points. This eliminated the labor and material associated with having separate connections for the panel points and chord splices.

Impact of Northridge

The Northridge earthquake occurred shortly after the erection of the roof SMRFs. The disturbing performance of steel SMRFs during this earthquake provoked much controversy within the engineering profession as to how appropriate the current building codes were with respect to designing and detailing this class of structures. Shortly following the Northridge Earthquake and the discovery of the Steel SMRF performance, the Uniform Building Code released a bulletin which effectively stated that the UBC code procedure concerning the connection design was not sufficient.

This event brought much discussion as to how safe or reliable the Rose Garden roof would be with it's four SMRFs if a similar event were to occur in Portland, Oregon. The owner's representative, Bob Collier, quickly sought an investigation into the seismic structural integrity of the steel SMRFs in the Rose Garden Project.

The challenge Ellerbe Becket Engineers faced was two-fold. One, the UBC code guidelines that were conventionally thought to be a sound rational engineering approach for SMRF design had just been omitted. Therefore, what guide lines could be used to ensure the performance of such structures during a seismic event? Secondly, the steel SMRFs erection was complete on the roof. If any retrofitting were to be done, it would have to be done in the field at a higher cost than if it could be done by the fabricator in the shop. In order to avoid costly delays in the construction phase, these issues had to be resolved in an efficient manner.

The approach to addressing the issues required exploring what alternatives were available to the dilemma of using Steel SMRFs. To change the roof system to a different lateral system would effect the ductility factor R_w. In turn, this would effect the design of the roof steel members and connections since the lateral loads would no doubt differ and result in additional costs since the fabrication was well under way. Secondly, the lateral forces on the concrete bowl below would have to be re-evaluated and members redesigned due to the incompatibility between the new Rw of the modified roof lateral system and that of the concrete dual system below. Again, the additional costs would be unreasonable. Therefore, changing the roof 's lateral system caused a domino effect in design, construction and material costs down to the foundation.

The next approach entailed considering means and methods which could be utilized reliably to reinforce the SMRFs moment connections. Ellerbe Becket engineers relied upon the ongoing research at the University of Texas which was being supervised by Michael Englehart. After numerous conversations with Michael Engalhart, it was concluded that the connections could be reinforced by following guidelines from his research. The estimated cost associated with this reinforcing procedure which would be done in the field was $280,000.

Before committing to such a construction task, Ellerbe Becket Engineers performed extensive analyses to determine what seismic force level the existing SMRFs could take. In short, the steel SMRFs had been governed by drift (stiffness) and not strength. When the capacity of the moment connections was evaluated and compared to the actual elastic seismic forces, it was shown that the connections could accommodate a higher force level than that expected from a seismic event. Therefor, to reinforce the connections would only serve to provide a joint capacity well beyond a reasonable seismic force level.

The final conclusion was not to reinforce the connections at this point since they had more than adequate strength capacity for the current code force levels. However, it was determined prudent to use a more resilient notch-tough weld material. In the future, if research concludes anything contradictory to this, the connections are accessible to reinforce later since the frames are not enclosed by architectural components. The project was spared $280,000, the City of Portland Engineers approved of this rational engineering decision, and costly construction delays were avoided.

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