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INDIAN WELLS TENNIS GARDEN, NEW STADIUM 2

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Abstract

The Indian Wells Tennis Garden (IWTG), located in Indian Wells, California, is home of the BNP Paribas Open, the biggest tennis tournament outside the Grand Slams. In 2014, IWTG had a major renovation adding a new 8,000-seat Stadium and an extensive site expansion. The stadium includes concession stands, three quick serve restaurants, and three fine dining restaurants facing the courtside, two of which offer a five star fine dining experience. The $95M project is the culmination of an innovative Design-Build approach.

The complexity of this project was heightened by a strict deadline. The project had to be constructed within 10 months and 10 days without interfering with either the 2013 or 2014 BNP Paribas Open Tennis Tournament. Objective that was accomplished by a close collaboration of the design team, architect, contractor, subcontractors and owner.

This paper describes the design, construction challenges, and lessons learned throughout the course of this project. Given the potential sensitivity of spectators to vibrations transmitted through the structure, vibration studies were undertaken to ensure that both the upper precast concrete bowl and the upper podium level steel framing were stiff enough to provide an acceptable level of performance. Theses vibration studies utilized finite element modeling and time-history analysis, combined with other methods.

Keywords: Stadium Design; Design Build; Vibrations; ASCE7-05

1. Introduction

The Indian Wells Tennis Garden (IWTG), located in Indian Wells, California, is among the premier tennis stadiums in the world. In 2014, IWTG underwent a $95M major renovation that included a site expansion and the addition of Stadium 2. The newly completed 8,000 seat Stadium 2 boasts three fast-casual restaurants, four concession stands, a new commissary and three gourmet restaurants, two of which have luxury suite views of center court. Renovation of the site included an additional 40 acres of new shade structures, a redesigned north gate with a standalone box office, four new practice courts, new onsite hardscape and new turf parking lots for over 2,000 additional cars.

The project had an aggressive construction schedule starting immediately after the 2013 Indian Wells BNP Paribas Tennis Tournament, and a deadline for substantial completion by beginning of February 2014. The structural design was completed within three months from the kick off meeting in December 2012 to the City approvals in February 2013. This paper describes key aspects of the design phase, as well as, key aspects during the construction of stadium 2.

2. Design Phase

The Stadium is located less than six miles from the San Andreas Fault (Fig. 1), known for its high earthquake risk in the region. This proximity to an active fault required special attention to the seismic design of all the building components. The design codes utilized were the 2010 California Building Code and ASCE7-05 “Minimum Design Loads for Buildings and Other Structures” (ASCE7). The seismic design parameters were obtained from United States Geological Service Design Maps, which for the Stadium location are as follows: a short period acceleration SDS of 1.0g and one-second acceleration SD1 of 0.6g. These parameters correspond to a zone with high seismicity in California, but they are far from the maximum in the State. The seismic design category is D and occupancy category is III.

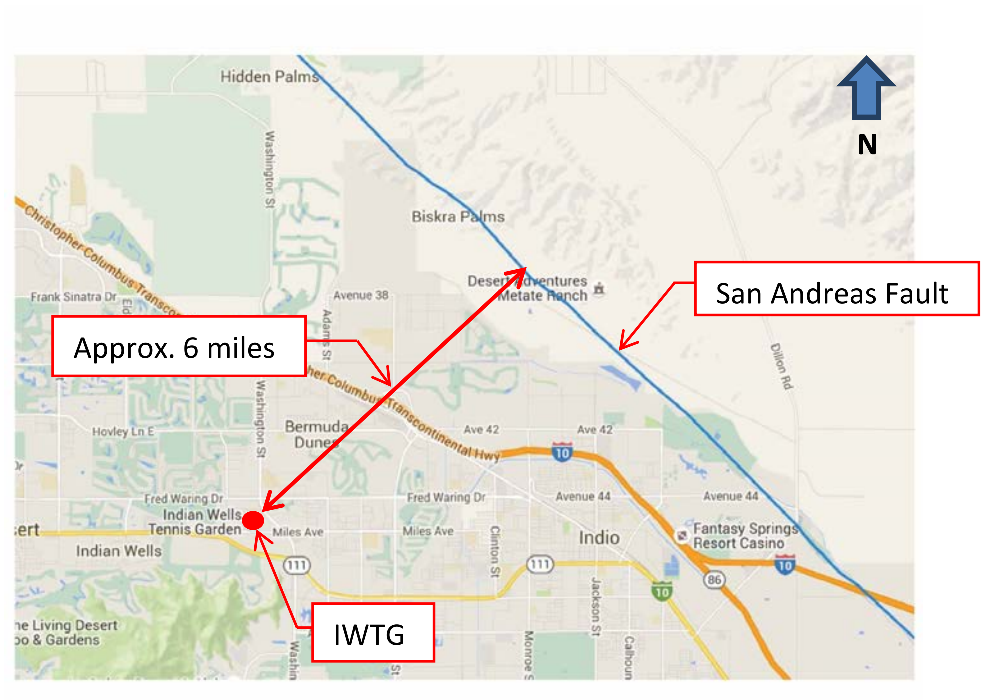

Fig. 1 – IWTG Location with respect to San Andreas Fault

Cast in place concrete was the preferred material for gravity and lateral system, following a similar construction of Stadium 1, but its long lead time for construction made it a non viable option due to the constrained schedule. The design team worked with the general contractor in the selection of multiple construction materials that allowed on site construction in parallel with offsite construction. .

2.1 Stadium frame description

The Stadium has two major areas Lower bowl and Upper Bowl depicted in Fig. 2. The Lower bowl area is comprised of cast in place concrete on grade seating, retaining walls and a concrete utility tunnel. The Upper bowl is comprised of the steel frame podium level, restaurants facing courtside, mechanical and storage yards on top of the podium level, and the Upper bowl seating. Fig. 3 shows a typical cross section of the stadium.

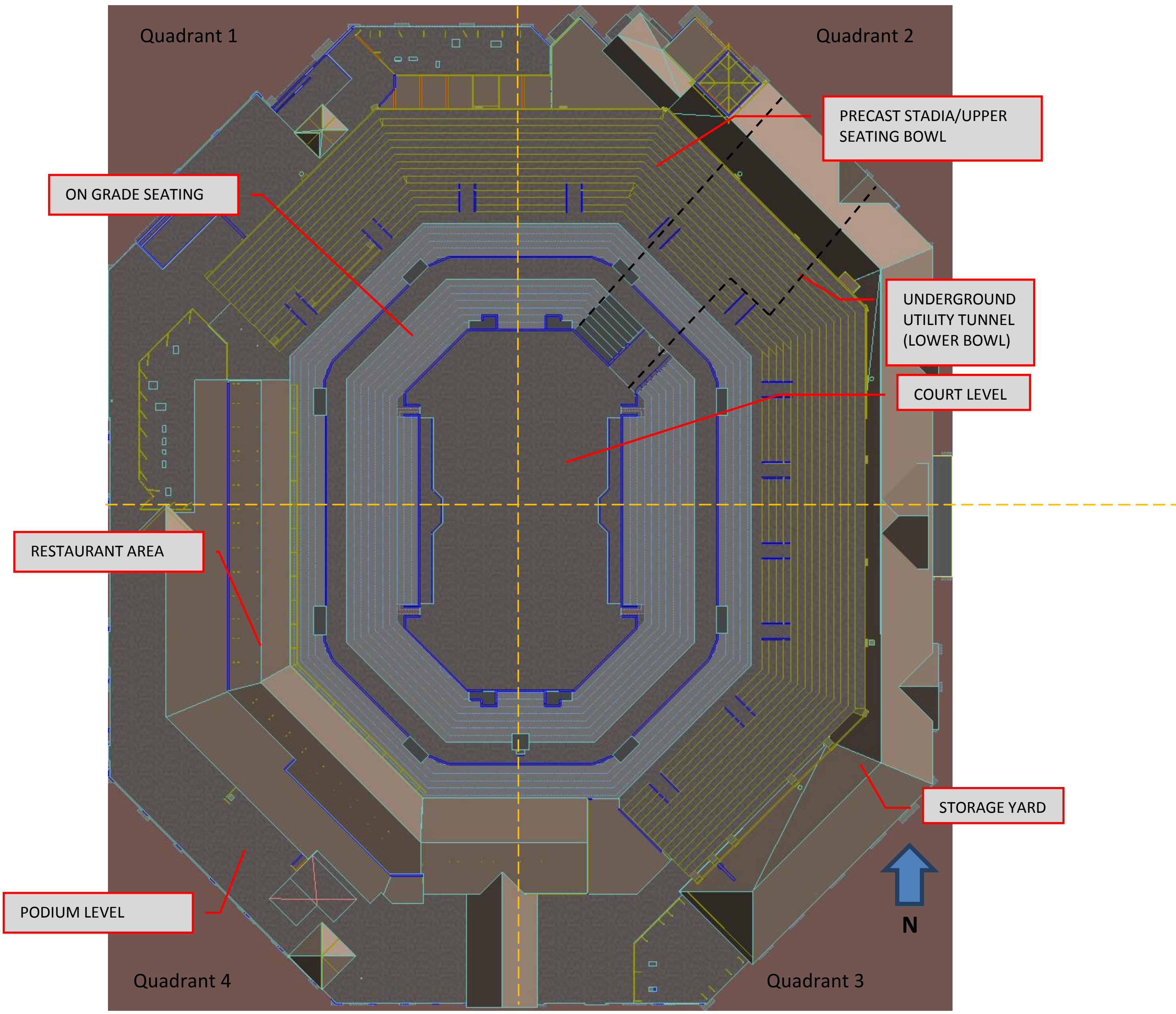

Fig. 2 – IWTG Plan view

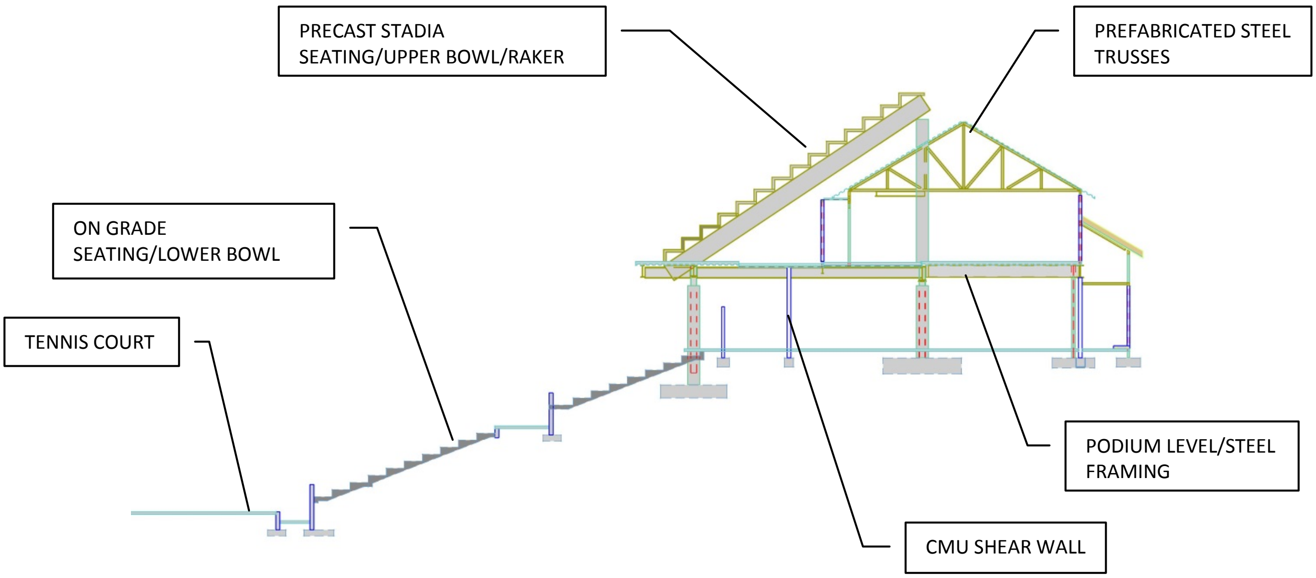

Fig. 3 – IWTG Typical cross section

The podium level consists of 4-1/2” of normal weight concrete over 3” metal floor deck supported by steel beams and columns. Non load-bearing concrete masonry unit (CMU) shear walls and concrete load-bearing shear walls serve as the lateral force resisting system. Supported above the podium level are light-gage framed upper level structures that utilize Sure-Board shear walls and Hardy Frames as their lateral force resisting system. These upper level structures have prefabricated light gage trusses for their roof framing, supporting metal roof deck.

The upper bowl seating consists of precast prestressed raker beams and stadium seating “riser” sections (Fig. 4) spanning to precast concrete columns, which are supported by the steel framed podium level below. Concrete load-bearing shear walls serve as the lateral force-resisting system for the upper bowl.

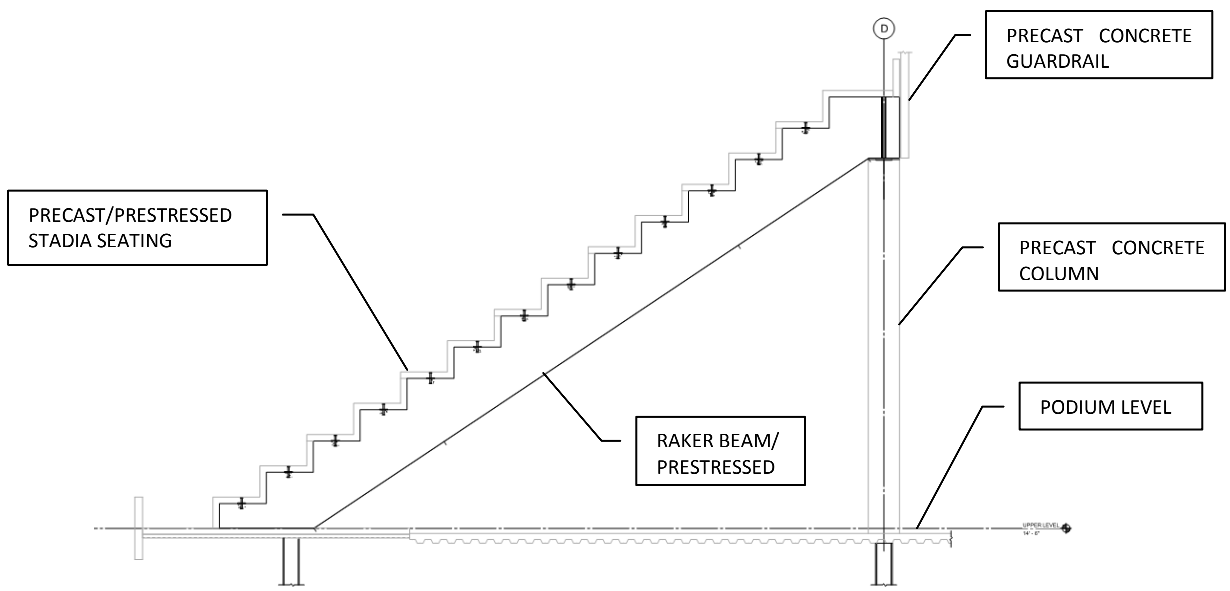

Fig. 4 – IWTG Typical stadia seating section

2.2 Key design aspects

2.2.1 Seismic design

As mentioned earlier the seismic design was done under ASCE7-05. The podium lateral system is a combination of bearing walls systems including special concrete reinforced masonry walls and special reinforced concrete shear walls. The response modification factor per ASCE7-05, Table 12.2-1 is R=5.0 for both systems, the overstrength factor is Ω_o=2.5 for both systems, and the deflection amplification factor is C_d=5.0 for concrete and C_d=3.5 for masonry. Per sections 12.2.3.1 and 12.2.3.2 of ASCE7 the values of R, Ω_o and C_d shall not exceed the lowest R that is used in the same direction at any story above that story, and the deflection amplification factor and the system overstrength factor used for the design of any story shall not be less than the largest value of this factor among the systems. Orthogonal effects were accounted as recommended by ASCE7-05, by applying 100 percent of the forces in one direction plus 30 percent of the forces for the perpendicular direction. This provision was used on the design of the upper stadia seating and the podium level, light-framed structures are exempt of this provision given that the roof diaphragm is considered as flexible. The seismic coefficient was governed by the ratio S_{DS}/(R/I) for each structure and the redundancy factor used was 1.3.

The podium level was divided in four quadrants as shown on Fig. 2. Each quadrant has CMU walls with a balanced distribution preventing horizontal torsional irregularities at each quadrant as well as the stadium as a whole unit. The required total wall length for the entire structure was approximately 466 feet. for each direction of analysis. The total wall length provided for the north-south direction was 1013 feet and for the east-west direction was 768 feet. The total base shear for the podium was approximately 5,000 kips and the average shear per foot in the CMU walls was 6.9 kip/ft.

The light-framed structures on top of the podium level were designed for the R=6.5 for light framed walls sheathed with steel sheets, the selected wall type was sure-board walls. The design of this light-framed structures for a higher R is supported by a two-staged equivalent lateral force procedure allowed by ASCE7-05, section 12.2.3.1; The requirements to comply with this section area as follows: The stiffness of the lower portion must be at least 10 times the stiffness of the upper portion, the period of the entire structure shall not be greater than 1.1 times the period of the upper portion considered as a separate structure fixed at the base. The rigid podium level was designed as a separate structure imposing the reaction of the flexible upper portion amplified by the ratio of the R/ρ of the upper portion over R/ρ of the lower portion.

A special consideration to the light-framed structures at the restaurants facing the courtside was given; the courtside consisted of long and tall windows to increase the visibility of the spectators from the restaurants to the courtside. In order to accommodate this requirement the structures were designed as a three sided diaphragm consisting of Sureboard walls running perpendicular to the courtside and “back of house” Sureboard walls running parallel to the courtside. The metal deck diaphragm in most cases is considered as a flexible diaphragm given the substantial deflections that the diaphragm experience with respect to the support lines. ASCE 7-05 defines a flexible diaphragm were the maximum diaphragm deflection is greater than two times the average drift of the vertical (lateral) elements. Considering this definition the diaphragm definition was evaluated and it was determined that diaphragm deflection was in the order 0.14-in versus 0.17-in of the wall deflection, which allowed the diaphragm as semi-rigid diaphragm, allowing its use for a three-sided diaphragm. Additionally, the final window layout allowed incorporating hardy-frame wall system on the courtside for redundancy.

The upper stadia seating is laterally supported by special concrete shear walls in the direction perpendicular to the stadia seating, and is supported by the podium diaphragm for the longitudinal direction. This configuration creates a three-sided diaphragm, and required to design the precast connections, between the stadia seating beams, with enough strength to make the stadia seating act as a rigid diaphragm an transmit the forces to the podium level as well as transverse walls by torsion. The upper stadia seating is comprised of 5 independent sections that are connected to the shear walls perpendicular to the stadia seating. Each section was analyzed independently and the shear wall demand was combined for the wall final design.

The concrete shear wall design was complex due to the high demands and the need of accommodating the construction sequencing and schedule. The construction sequencing required to erect all the steel framing prior to building concrete shear walls up to the podium level, and also required to erect all the precast elements prior to building the walls to support the upper stadia seating. Additionally, the seismic demands on these shear walls were high and required to provide special boundary wall elements as required by ACI 318-05. In order to incorporate the constructability and the seismic requirements the wall detailing included embedded steel columns acting as wall boundary elements for the section below the podium. The boundary elements above the podium consisted of the standard steel cage in one side of the wall and the precast concrete column at the opposite end of the shear wall. Figure 5 depicts a typical concrete shear wall elevation. The shear wall segment above the podium level was shotcreted, since the raker beam needed to be in place for the upper stadia seating erection. Steel cap plates with welded bars were provided at the top of the columns in order to splice the steel cages, and to connect to the embedded steel couplers at the precast columns. Horizontal welded bars at the steel columns and threaded bars at the precast columns were provided to develop the members into the solid concrete. As a result of these modifications from the standard shear wall detailing the erection of the steel framing and precast elements was speeded up, while maintaining required seismic detailing for the concrete walls.

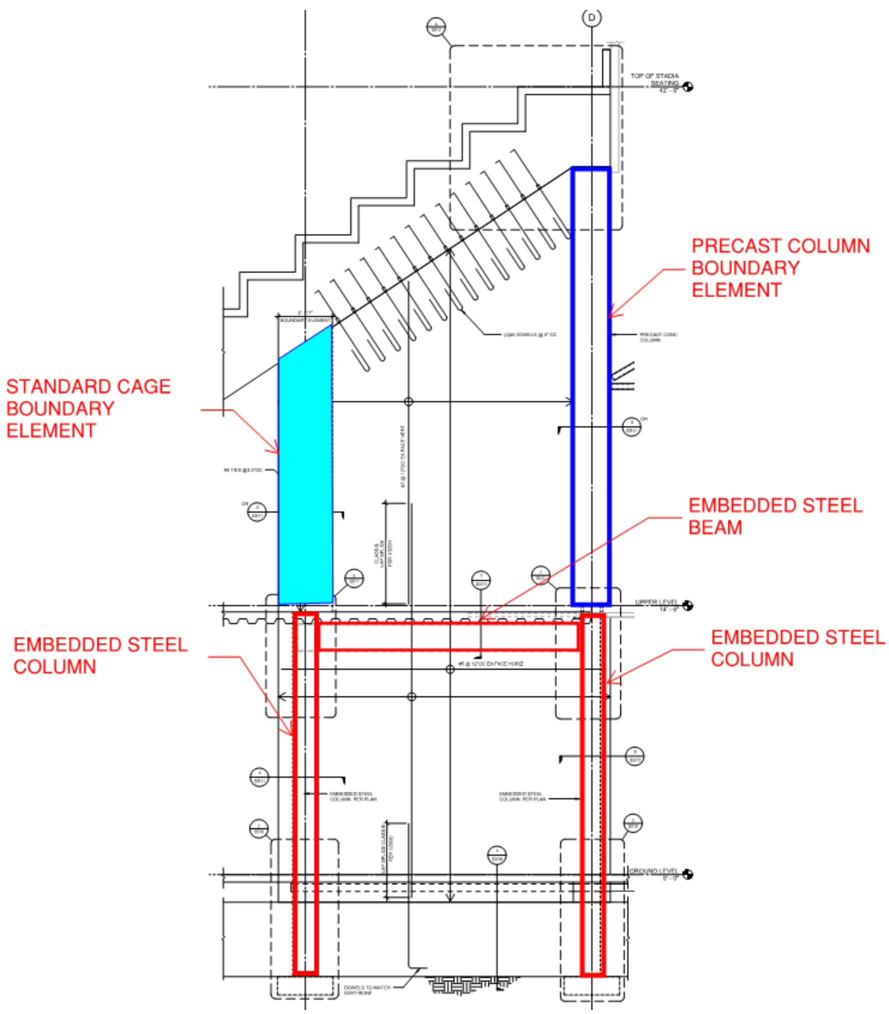

Fig. 5 – IWTG Typical Concrete Shear Wall Elevation

2.2.2 Vibration design

The tournament did not have any specific guidelines or requirements for vibrations, but to avoid undesirable floor excitation for the spectators a detailed vibration analysis was performed for two areas of the project: the upper stadia seating and the restaurant area.

2.2.2.1 Upper bowl seating

The precast riser beams support the spectators, and span between the raker beams. The analysis focused on the riser beam with the longest span in the stadium. The predominant source of excitation is spectators moving on top of the beams. Only the vertical vibration response was evaluated, since the beams area is constrained against horizontal movement and torsional movement through their connections to the raker beams.

The calculated first natural frequency of the longest stadia riser beam was 13.3 Hz, and the second and third frequencies were 53.3 and 119.9 Hz. These natural frequencies indicate that the stadia riser beams are very stiff with respect to vibration performance. Typical walking frequencies range from 1.6 to 2.4 Hz, and it is a common for natural frequencies to be 2.5 times greater than the excitation frequency (Bachmann H., *et al*, 1997), this method is also known as the “high tuning” principle. For our case, the maximum excitation was 5.8 Hz, well below the first natural frequency.

The stadia raker beams vibration was also evaluated obtaining a natural frequency of 6.22 Hz. This is still above the amplified walking frequency mentioned above. The estimate of 6.22 Hz is conservative, since the raker beam was assumed pin supported, whereas that actual support condition are providing some restraint against end rotations. Additionally, the tributary weight to the raker beam is approximately 183,000 pounds. This mass is not likely to be significantly excited by a single walker over the raker beam assembly, as the excitation weight will be marginal compared to the structure weight.

2.2.2.2 Floor framing system at Restaurant area

The restaurant area framing is comprised of steel beams, columns and a metal deck with concrete fill on top, as described in section 2. The evaluation focused on the 9-foot cantilever section at the windows facing the court. The restaurant structure above the podium is comprised of light gage metal framing. The evaluation of this area consisted in a two-step check, the first one followed a similar procedure as the raker beam evaluation, and the second step consisted of a finite element analysis of the floor structure to calculate the vibration response, due to a simulated footfall excitation.

The finite element analysis with SAP200 (CSI) of the restaurant area predicted that the first three significant modes of vibration have frequencies of 5.7, 7.0 and 7.3 Hz. In restaurants, the typical maximum walking pace is likely to be moderate – about 2 Hz or less. Following the procedure described in section 2.2.2.1, the maximum excitation is 5 Hz, which is less than the first mode 5.7 Hz complying with the “high tuning” principle.

The second step consisted in a time-history analysis in which the forces induced by a typical walker were simulated, and then the maximum vertical acceleration was obtained. The forcing function used to simulate the walker was a time varying sinusoidal vertical load with an amplitude of + / – 90 pounds, and a frequency of 1.5, 2.0 and 2.5 Hz. The weight selected in our analysis was conservative, considering slightly higher amplitude as the one defined in AISC Design Guide 11 (AISCDG11). This guideline establishes the average walker weight of 157 pounds, and establishes the force exerted by a single step as 41 percent of the walker’s weight, or 65 pounds for 157-pound walker. Design Guide 11 also establishes limits on vertical acceleration that could result in discomfort of an occupant of the building. The upper limit for offices, residences and churches is 0.5g. The time-history analysis determined that the vertical accelerations at the middle of restaurant framing and at the tip of the cantilever beam were less than 0.1g. Fig. 6 depicts the different accelerations of the system for the selected forcing functions.

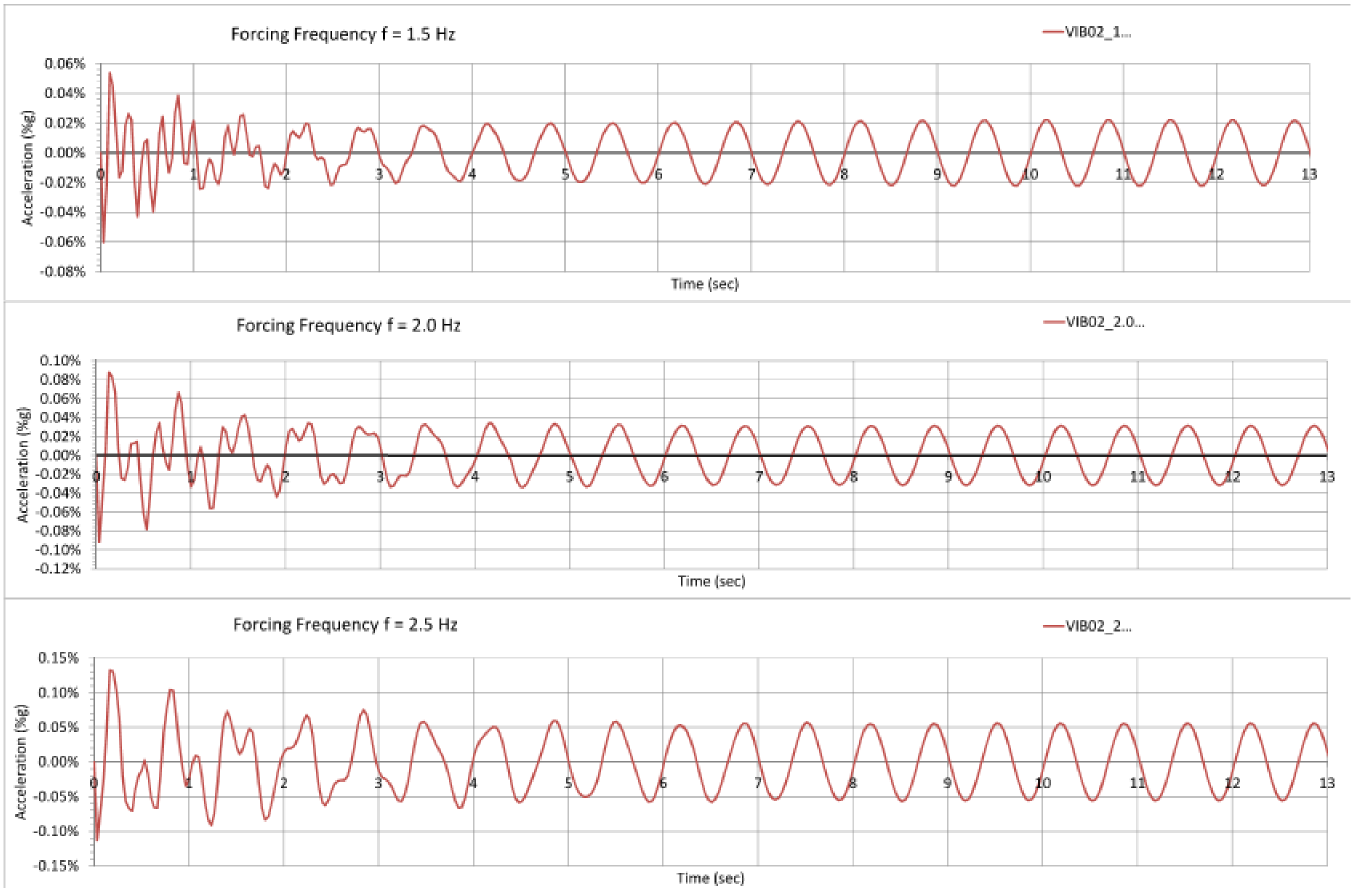

Fig. 6 – IWTG Acceleration response at restaurant area

2.2.3 Thermal joints

With a circumference of approximately 500 feet and subject to heat differentials exceeding 50 degrees (F), Stadium 2 required reliable thermal expansion joints. Four thermal joints were integrated into the upper (podium) level such that the concrete and steel could expand and contract within each quadrant, independent of adjacent quadrants. These joints were strategically located to minimize their aesthetic impact while maximizing their effectiveness, and were designed to allow the structural deck and supporting steel beams to expand and contract without creating seismic isolation between quadrants. During the course of construction, these thermal joints were put to the test and performed exceptionally well under real-time temperature swings in the range of 40-50 degrees (F). Fig 7 shows the thermal joints location and Fig. 8 depicts a thermal joint during construction.

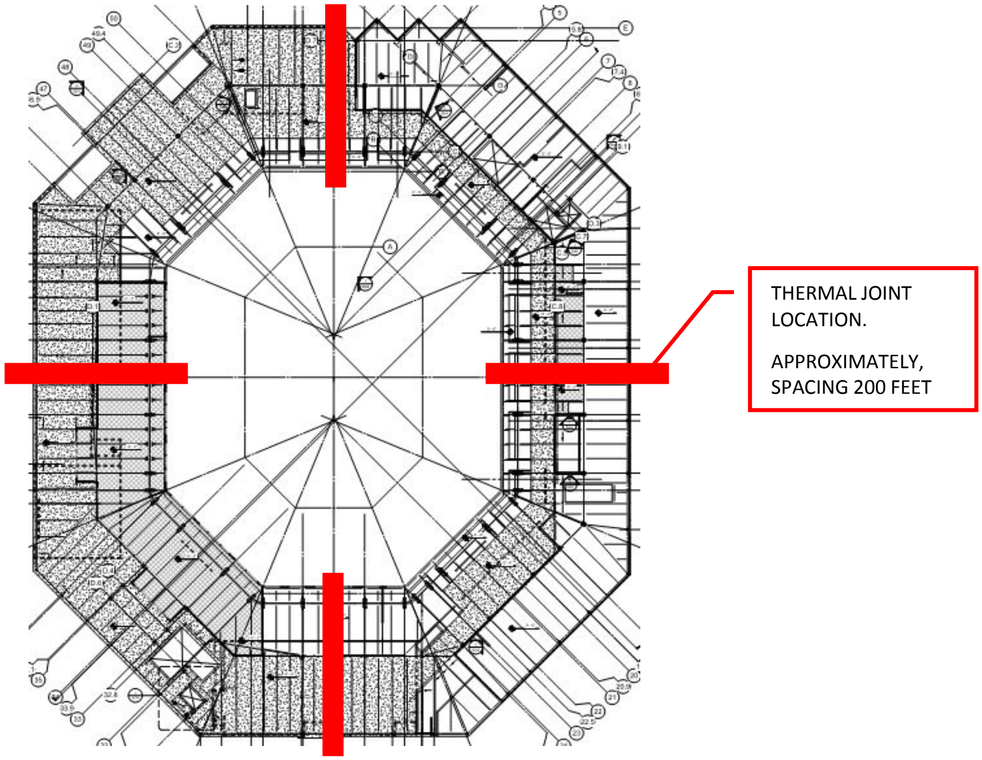

Fig. 7 – IWTG Thermal joint location

Fig. 8 – IWTG Thermal joint looking from top of podium deck

3. Construction Phase

The owner of the stadium stipulated that Stadium 2 and all associated site structures and improvements would have to be constructed between annual Tennis Tournaments without interfering with either event.

The delivery method chosen by the owner was Design-Build. This method ensured that the design team was in constant communication with the contractor, subcontractors and design consultants. This method allowed working collaboratively in the design of structural components, defining construction materials throughout the project, and resolving field issues with creative solutions due to the close collaboration of all the parties involved. This high standard of excellence and quality was reflected through the efficiency of the team and, as a result, the structural design was completed within a record time of 3 months with construction completed within 10 months and 10 days. Fig. 10 depicts an aerial view of finished stadium; photo is courtesy of Watkins Landmark Construction.

The coordination during design and throughout the construction was using a BIM model in REVIT 2013 (Autodesk). With a constraint schedule and multiple trades working in the field at different sections of the stadium, a 3D model for coordination during construction was a key element to anticipate potential field conflicts between structural elements and other disciplines. Fig 9 depicts a rendering of the structural Revit model.

Temperatures in City of Indian Wells during summer time can swing from low 40° to 120° Fahrenheit during the day. As a result of this, concrete placement started at 1:30 AM and normal work hours began at 4:00AM under lights. Special consideration was given to design of concrete mixes, as well as, concrete placement and curing techniques and quality control measures.

The construction sequence of each quadrant (see Fig. 2) played a significant role in the schedule. The service tunnel located in quadrant 2 was the critical path for construction, and dictated the access ramp location and construction sequence of the remaining segments. The steel erection started at quadrant 3, followed by quadrant 1, quadrant 4 to finally close with quadrant 2. Once the steel erection was completed, it allowed multiple trades crews to work under the steel structure to build CMU walls, concrete shear walls, install light gage partition walls, pour slab on grade concrete, place underground utilities.

The construction sequence of the upper bowl changed started in quadrant 3 in a counter-clockwise direction, starting with the erection of the precast stadia seating, as well as the supporting steel for the prefabricated trusses. This construction sequence allowed to incorporate the final restaurant interior design, and provided enough flexibility for the design team and construction team to incorporate revisions to the construction documents, accommodating each restaurant tenant need.

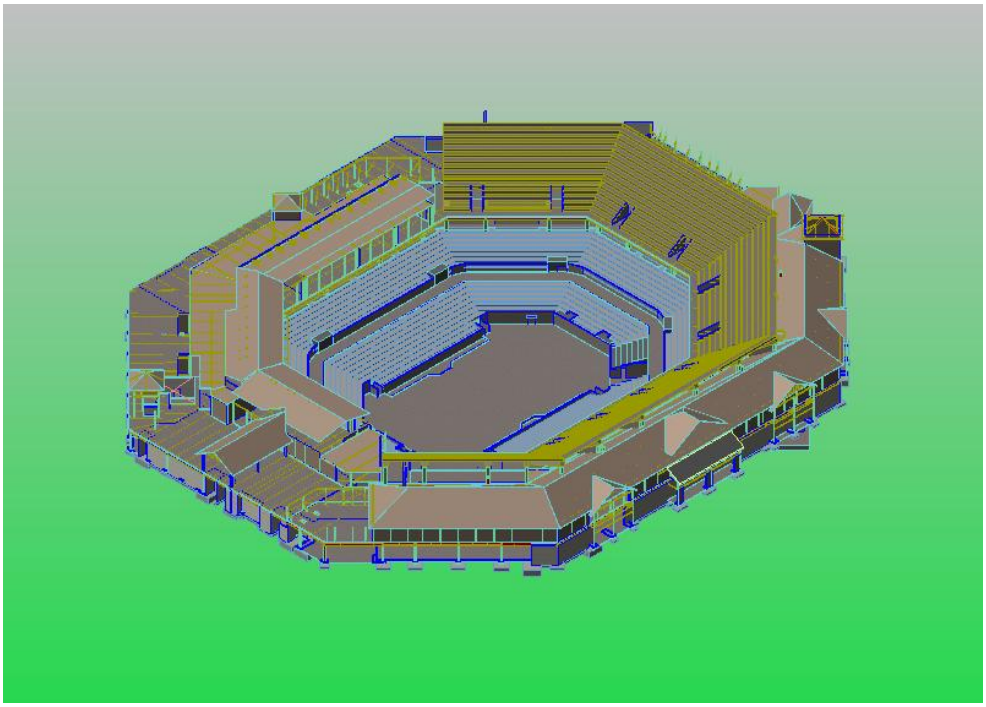

Fig. 9 – IWTG Structural Revit model 3D Rendering

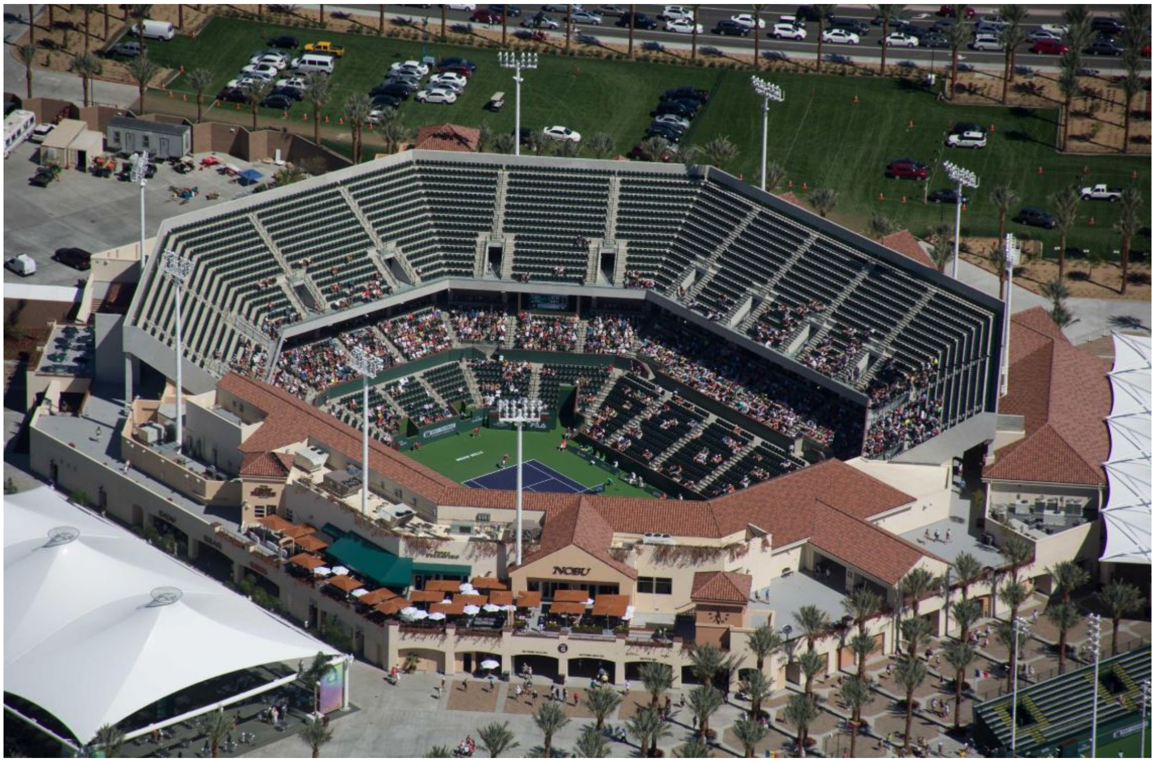

Fig. 10 – IWTG Stadium 2 at BNP Paribas Open 2014

4. Conclusion

Home to the second-largest tennis stadium in the world, IWTG boasts close-to-court seating with sight lines that surpass not only other tennis stadiums, but also most of the world's sporting stadiums. Upon the completion of this project, IWTG was named the 2014 Featured Facility Award winner - the USTA's highest tennis facility honor. That same year, the facility accommodated more than 431,000 fans for the 2014 BNP Paribas Open Tennis Tournament. With its state-of-the-art court construction and facility design, in addition to court amenities, this remarkable landmark sporting facility provides athletes, spectators and the public with an unparalleled state-of-the-art experience.

7. References

The following list contains some of the references that were used when performing this analysis.

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