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# Atlantis The Royal The Palm, UAE

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**Abstract** The 43 story, 178m tall Atlantis The Royal Resort and Residences tower nearing completion on the Palm in Dubai is a unique project in terms of its overall scale, geometry and form. The tower superstructure has a long narrow curved floor plan, measuring 400m in length above the podium with significant long span bridge structures forming voids and cantilevered blocks. This paper describes the innovative structural design solutions adopted and the construction solutions adopted by the Contractor to realize the Architectural vision.

**Keywords** Tall Building, Wind engineering, Strand jacking, Cantilever, skybridge, long span, composite construction

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## 1. Introduction

Atlantis The Royal is an iconic project, contemporary in style, rising 178 m above the Palm in Dubai. Its subtle 'S' shaped building footprint, is 25 m wide, extending around 500 m in length at the base. The development houses 795 new guest rooms and suites, and 231 luxury residences. The approximately 400 000 $m^2$ development is located on the crescent of The Palm and adjacent to the existing Atlantis. The Resort will stand 43 stories tall and feature private gardens, infinity pools and unparalleled views across Dubai's skyline.

Working alongside Kohn Pedersen Fox Associates, as design architects and IBI as the executive architect, WSP has provided multi-disciplinary engineering and environmental consultancy services for Atlantis The Royal. This paper focusses on the unique Structural Engineering design, construction aspects and creative solutions required to realize the architect's vision.

The building is divided into two main components, a hotel and a residential building. These two buildings, featuring a series of large openings throughout, create a transparent silhouette linked together at the podium levels, which serve as a shared lobby and amenity space for both. The two towers are also connected by a 50-meter-long central skybridge that links the 18th through the 22nd floors of the two towers, forming a grand arch, Fig. 1.

Each one of the two main towers consists of 3 sub-towers connected by a series of sky bridges that are suspended in the air and link the sub-towers together. The six sub-towers vary in height with a maximum hotel tower height of 43 stories (178 m) and a maximum residential tower height of 38 stories (159 m). The towers are interconnected by 3 and 4 story sky bridge blocks while also supporting cantilevered blocks off the east and west sides of the buildings.

The building structural system was developed in coordination with over 50 international consultants covering all the various disciplines required for such a project, however the main design was done by KPF New York with interiors by G.A Design International-London for the hotel and Paris-based Sybille de Margerie (SM Design) for the residential tower. Dynamic Engineering Consultants is the Architect of Record.

WSP Structural Engineers were able to leverage its global expertise on all aspects of the design of the project. The project was delivered through the collaboration of WSP structural teams from the UK, New York and Middle East, working on a combined analysis model as well as BIM models. The BIM model worksets were organized so that the work could be performed simultaneously. Considering the time difference between New York and Dubai, this meant that we were able to provide almost 24/7 work on engineering and document production.

## 2. Foundations

The Palm was the first of the Palm Islands to be



**Figure 1.** Overall view showing Hotel (left), Residential (right).

reclaimed. Its reclamation was completed in 2003 and involved the placement of 5.5 million $m^3$ of rock armour for the outer edge of the crescent, and 94 million $m^3$ of carbonate sand dredged about 11 km away from the seabed of the Gulf. The reclamation fill was then vibro-compacted in order to increase its relative density and thus improve its performance with respect to bearing capacity, settlement and to reduce the risk of an earthquake-induced liquefaction potential. The compacted fill was verified by undertaking extensive cone penetrometer testing (CPT), both pre- and post-vibro-compaction, to evaluate the ground improvement.

The generalised ground profile is comprised of 15m of ground improved fill over 15-25 m of Sandtone/Calcarenite on Conglomerate/Siltstone, Fig. 2.

Given the magnitude of the structural loads, reinforced concrete bored piles into bedrock were the required foundation solution for compression, tension and lateral loading.

The piles were designed mainly as friction piles, in accordance with local authority recommendations and considering the weak nature of the bearing strata. This helped to minimize foundation settlements and to mitigate potential reductions in pile capacity, should weaker layers

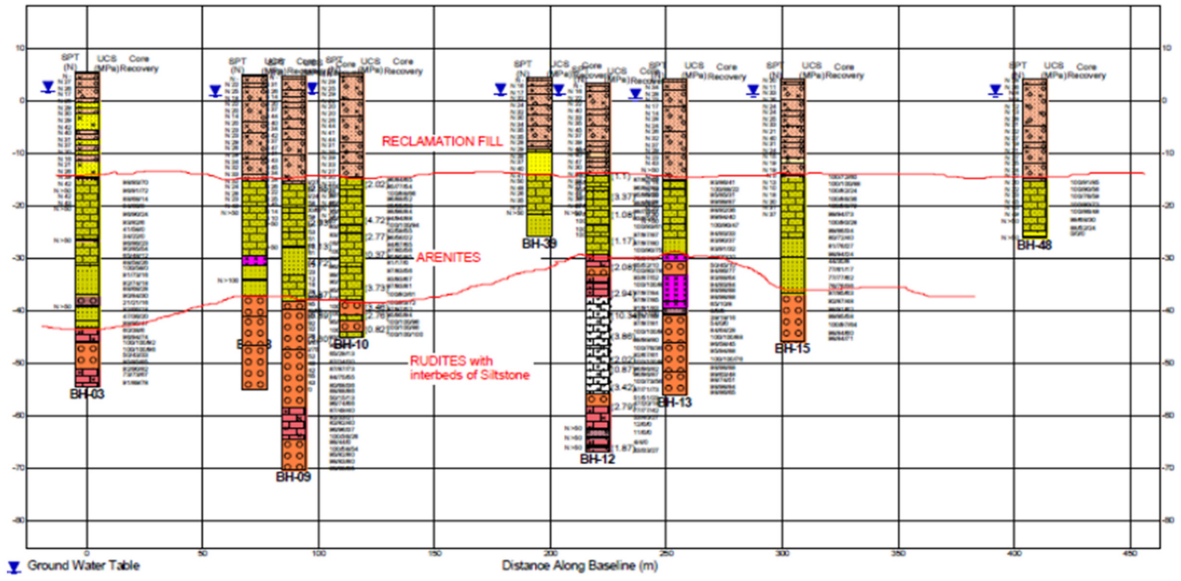



**Figure 2.** Extract from ground investigation report showing general soil profile. (Image courtesy of ACES)

exist close to the pile base. Similarly, negative skin friction caused by down-drag from the upper layers was considered in establishing the pile capacity.

At the early stages of the project, the design team had agreed that it would be beneficial to undertake a programme of pile load testing in advance of the final piled foundation design. This was instrumental in validating the design assumptions and to realize potential savings and efficiencies in the design.

A preliminary pile testing regime comprised of nine tests in total was specified:

- Six for bi-directional tests (compression and tension using Osterberg-cell) – Two 1500 mm, Two 1200 mm and Two 900 mm diameter piles
- Three for lateral tests – 1500 mm, 1200 mm and 900 mm – using pairs of piles.

All of the test piles were non-production piles. The piles subjected to bi-directional load were tested to 300% of the safe working load, whereas the piles subjected to lateral loads were tested to 150% of the safe working load. The maximum working (service) load requirements was 23 MN.

The availability of the pile test data during the Schematic Design Stage, enabled WSP to optimise the final pile design as more appropriate design parameters became available, including skin friction distribution with depth.

The tests were supervised by WSP and then back analysed to determine the required pile design parameters and also to derive the appropriate pile stiffness values to be adopted for the boundary conditions in the structural analysis model. Upper and lower bound values were adopted to ensure that the range of possible behaviours were covered.

The pile load test results were also used to update the ground investigation report recommendations regarding piled foundation capacities for submission to the local authorities.

## 3. Sub-Structure

The project has one subterranean level for parking. The foundation system is comprised of 2.0 m to 2.5 m deep raft foundations under each of the 6 sub-tower's core and tower columns, and localized pile caps under main podium columns and shear walls. Shallow pile caps support the basement slab within the underground car parking garage structure. Fig. 3 shows the overall view of the pile foundation for the project.

Given the large footprint of the building and the high ground water level, a key consideration in the structural design was to minimize the depth of the foundation systems, thereby minimizing the scope of the excavation, shoring and dewatering.

The 600 m × 150 m basement car parking raft, outside

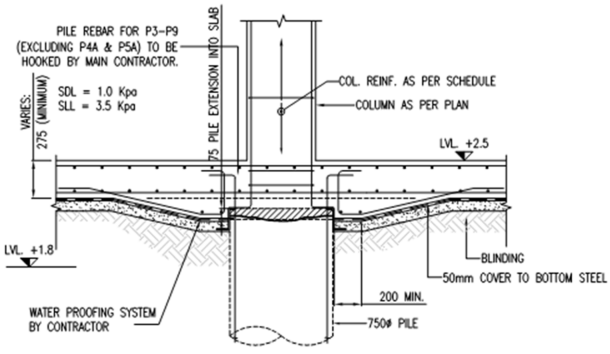



**Figure 4.** Shallow Pile Cap Detail was Developed to Preclude the Need for Widespread De-Watering Across the Site.

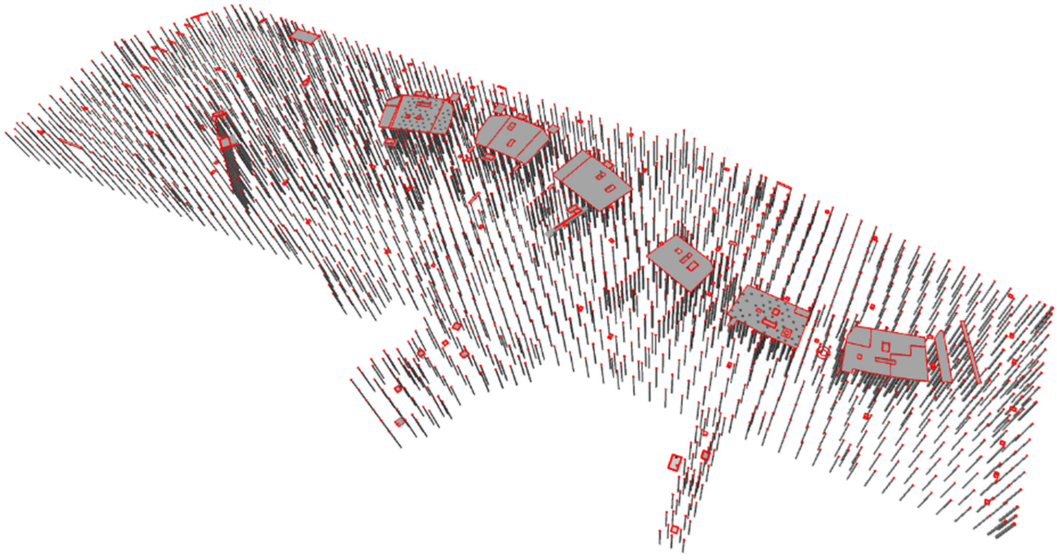


**Figure 3.** Overall View of Foundation.

the footprint of the main tower, was set very close to the design ground water level, hence a shallow integrated pile cap/suspended slab solution was developed to omit the need for a traditional deeper single pile cap, thus allowing the entire foundation slab to be cast just above the natural water table level as shown in Fig. 4 below.

The lift pits in the raft foundation were considered as openings in the design of the piles to minimize the depth of excavation. We were thereby able to adopt a simple removable sheet piled solution for the shoring, as opposed to folding the slab as is common practice in the region creating much deeper foundations and excavations.

## 4. Structural System

Each of the buildings is a combination of three sub-towers of various heights intermittently connected by a series of multistory skybridge structures placed between the sub-towers, creating multi-story voids within the buildings where the roof of each serves as a sky-court with various amenities.

At each end of the buildings there are 4-story overhang structures cantilevering about 12 m and attached at intermittent intervals along the height of the buildings, creating the void spaces in between.

The project has an overall plan dimension of about 25 m deep and more than 400 m wide above ground.

The structural material is primarily concrete with steel systems strategically introduced due to their advantages in load transfer and constructability. The structural system is primarily comprised of concrete slab, columns and shear walls. However, the cantilever blocks are designed to be steel structures for constructability while minimizing the weight of the overhang structure. Internal multistory bracing systems are utilized to enable the 12 m cantilever span to be efficiently supported by the tower columns. While columns are typically reinforced concrete, composite columns using steel core are utilized at certain locations, especially for the support of the skybridge.

At the main skybridge, a one-story deep steel truss system is introduced to span 50 m between the two main towers. At typical skybridges, a series of steel plate girders, spanning 30 m between the adjacent towers, are introduced to support the multistory suspended structures.

The typical floor system is a combination of 200 mm conventional slab within the building core, 200 mm to 350 mm concrete post-tension slab within the residence area and 90 mm concrete slab over 50 mm composite metal deck for the cantilever areas.

Concrete strengths range from 50 to 80 MPa for columns and shear walls. Shear wall thicknesses vary from 200 mm to 500 mm depending on their level, function, and utilization. The coupling link beams are typically reinforced concrete beams with exceptions at certain locations where composite beams with steel built-up sections are utilized.

## 5. Lateral Force Resisting System

The tower lateral force resisting system is comprised of a series core shear walls around means of egress. Since each of the hotel or residential buildings is a combination

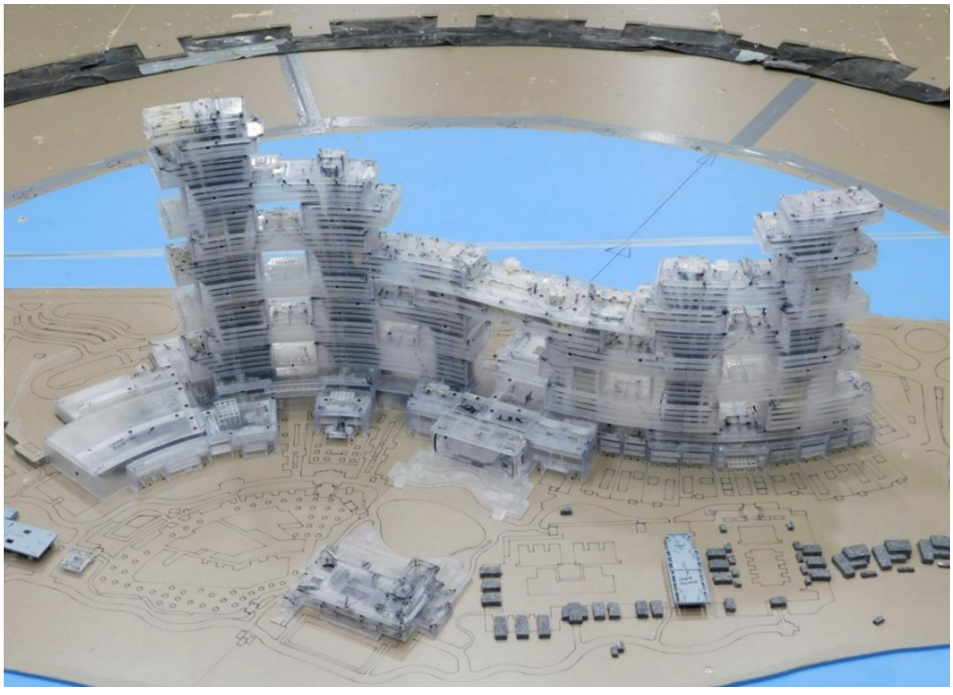


**Figure 5.** Wind Tunnel Model. (courtesy of RWDI)

of three sub-towers of varying heights, each tower has its own core walls, interconnected via floor slab diaphragm action.

The tower construction at the skybridges is designed with no expansion joints. Therefore, the continuity stresses caused by gravity load, wind, seismic, temperature and volumetric changes are considered in the design. Similarly, at the skybridge, the one-story deep trusses supported by shear walls at each end, react to wind lateral forces that were accounted for in the design of the skybridge truss and its connections.

The unique geometry of the structure created both opportunities and challenges in the design of the towers for the wind and seismic force effects. The large openings at the sky-courts throughout the building reduced the wind effect on the towers as they allow for wind passthrough. However, the very long and relatively narrow floor plates with multiple diaphragm slab interruption and multiple core walls required several types of wind tunnel studies in order to capture the maximum wind effects for the design of the core walls as well as the diaphragm slabs and skybridges. The wind tunnel tests were performed by RWDI for a two-tower analogy (residential and hotel) interconnected by the skybridge, as well as a six sub-tower version interconnected by a series of skybridges. The latter approach provides more refined information for the floor diaphragm forces at the skybridges and throughout. A general view of the wind tunnel model is shown in Fig. 5.

Usually, the wind tunnel loads are provided in 24 load cases for the design of the structure, however, for this project it required a total of 64 wind load cases to capture the extreme wind effect on each of the towers skybridge diaphragm action. The one-year and 10-year wind induced peak accelerations and torsional velocities are well within the acceptable limits.

The site is situated in a moderate seismic zone. Spectral analysis was performed for the seismic design response.

The effect of the ambient temperature variation was also considered in the design of the structure. This was particularly important due to the 400 m-plus- long and unique geometry of the structure with no expansion joints above the podium levels. However, no floor plate above the podium floors was longer than 100 m due to the sky-court voids. The structure would be exposed to the full effect of the temperature variations during construction while the building envelope and the HVAC systems were not yet in place to provide a controlled environment.

The building axial deformation due to elastic shortening, long term creep and shrinkage effects, in light of the building complex geometry, was studied and setting compensation values were proposed to minimize the differential axial deformation.




**Figure 6.** Main Skybridge Structure.

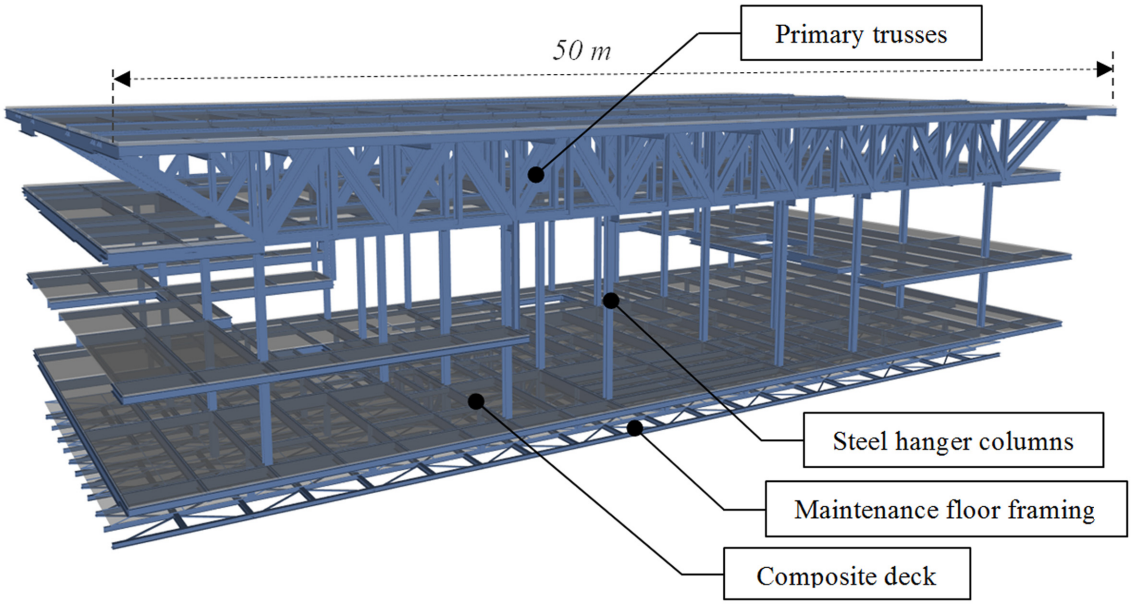



**Figure 7.** Main Skybridge Structural System.

## 6. Main Skybridge

At the center of the project, connecting the two towers is the signature skybridge, which features a grand space and amenity deck with a pool. The bridge spans 50 m between the towers at levels 18 through 22. A general view of the bridge is shown in Fig. 6.

A structural steel and composite floor system were chosen as a primary construction type for the long-span structure. The choice of material reduces the weight of the structure and greatly simplifies the construction process and logistics.

The primary structural system of the bridge is composed of four lines of steel trusses located between the upper two levels and steel hangers dropping from the trusses, supporting the steel floor framing of the lower levels as shown in Fig. 7.

The column layout of the typical bridge floor was seamlessly integrated into the architectural program to allow maximum flexibility and open space. All slabs are composite metal decks supported on steel framing, which has been designed such that the metal deck does not require shoring during construction. Connectivity and continuity of the bridge with the towers on each side is provided through the concrete floor diaphragms.

## 7. Interaction Between Towers

One of the main challenges in the design of the building is its continuous geometry. Each tower has three cores interconnected by skybridges and two towers that are interconnected by the main skybridge. Several options were considered in the early design stages with different jointing strategies that required control and accommodation of the tower movements under wind, seismic, temperature variation, and long-term volumetric changes attributed to the concrete material. The considered options included utilization of viscous dampers and locking devices to control movements across the joints. However, after numerous studies, a continuous structure without expansion joints was selected. Such a solution virtually eliminates the impact on architectural spaces and removes additional structural elements typically associated with joints.

Advanced analysis and design of the combined multitower/multicore system was based on the results from wind tunnel testing and a seismic site-specific study. A general view of the building's global finite element model and response of the combined system (one of the response modes) is presented in Fig. 8. Temperature deformations of the tower are affected by the irregular geometry of the tower (expansion and contraction cause changes to the length and curvature of the building). In addition to diaphragm stresses, the thermal deformations cause stresses in the steel elements of the skybridge.

## 8. Serviceability Considerations

Special attention was given to the overall bridge deformations from a serviceability perspective as well as the performance of the exterior cladding system and interior finishes. In a typical building design with standard spans, both absolute and relative deflection limits are considered. In the case of a long-span structure, absolute limits typically used in standard design practice might not be attainable, therefore a rotation or slope-based approach was used instead. Deformations of the bridge structure in the form of slopes and local gradients were evaluated in

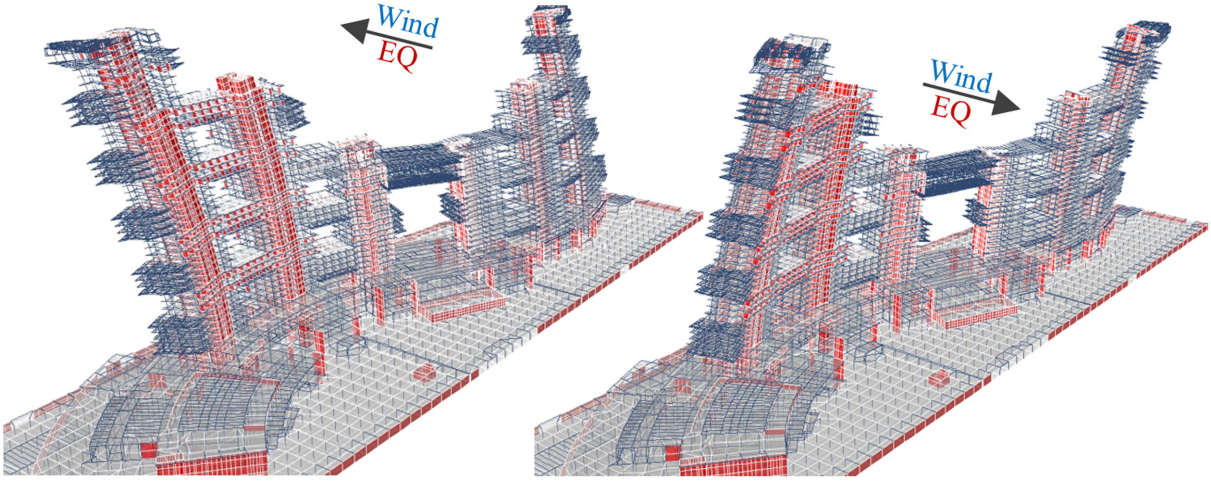



**Figure 8.** Response of Combined/Continuous Structural System. (Two towers interconnected by main skybridge)

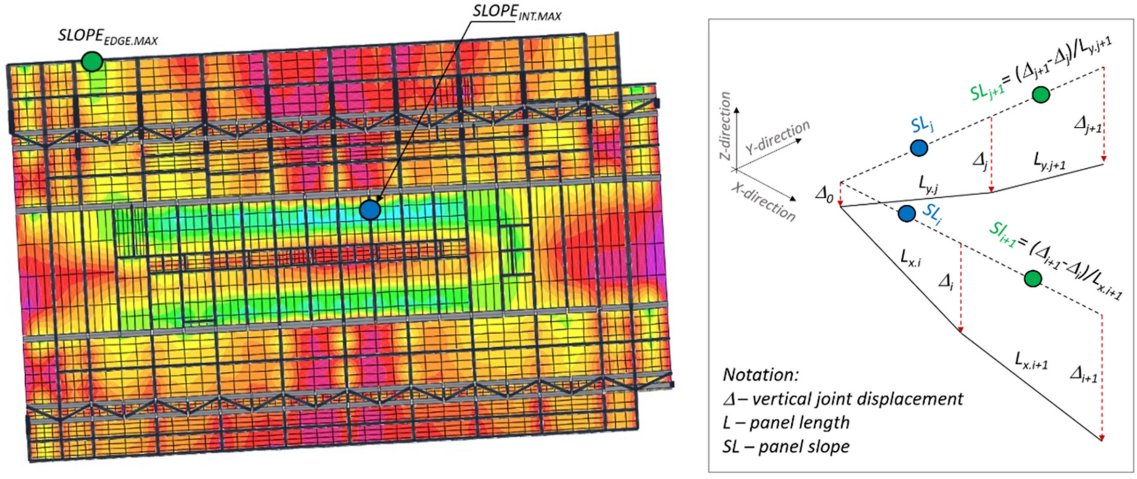



**Figure 9.** Serviceability Deflection Control.

the global finite element model with API customization. The skybridge was fine-tuned so that the slopes and deformations would stay within the cladding design guidelines. Fig. 9 shows a typical serviceability slope-deflection evaluation approach and results for the long span bridge structure.

Another critical serviceability design aspect is vibration assessment and human perception. The AISC Design Guide 11 was used for vibration evaluation and control.

## 9. Construction

The Construction of Atlantis The Royal presented unique challenges due to the presence of the cantilever blocks, skybridges and the main bridge. The primary Contractor was a JV between SsangYong and Besix with Eversendai as the primary steelwork fabricator and erector. Some of the key features are listed below.

*Main Bridge Construction*

The main bridge was assembled just above the lobby steel trusses at level 04 with the main bridge trusses at Level 9, spanning between the 2 slip-formed RC cores on either side of the lobby 50 m apart. The top and bottom chords of the trusses were formed using 460 MPa steel to minimize steel tonnage. The entire main bridge was then strand-jacked up into position at Level 22, Fig 10. The key facts are listed below:

- Lift height – 50 m from the truss assembly point (9$^{th}$ level); assembly was on top of L04, while lifted to 17$^{th}$ – 22$^{nd}$ floor
- Jacking Trusses – strand jacking, jacks were supported on trusses cantilevering from the left and right core walls
- Total weight – 1350 tons, including the soffit cladding of the bridge structure
- 8 hydraulic jacks were used
- Each jack consists of 22 Strands (grade 1860 strands 15.2 mm diameter)
- The carrying capacity of each jack – 260 tons

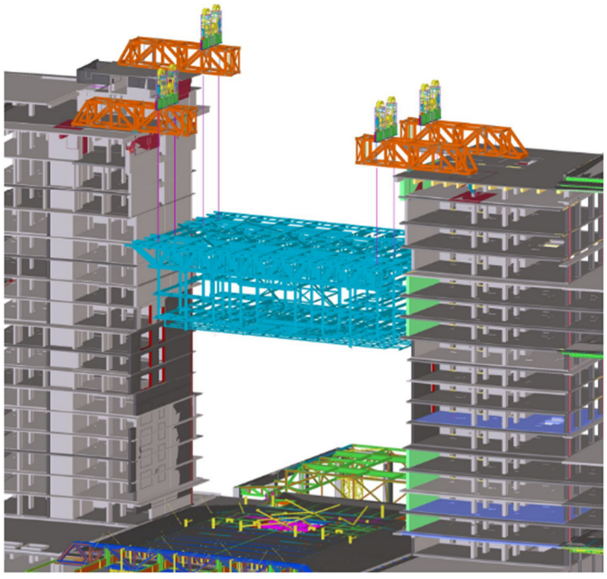


**Figure 10.** Image Showing the main bridge being strand-jacked into position.



**Figure 11.** Image Showing Propping to Lowest Deck of Skybridge Plate Girders being Installed.



**Figure 12.** Image showing Cantilever Blocks During Erection. Note the Main Skybridge being Assembled above the Central Lobby Trusses.

- The load on each jack varies from 160 to 205 tons
- Operation duration – about 10 hours duration. The leveling of the bridge is ensured by using laser distance measurements from the jacking points.

*Skybridge Construction*

The typical skybridges are comprised of steel plate girders up to 1.2 m deep, acting compositely with the lowest floor deck. Some of the heaviest sections had to be split into 3 parts to be able to be lifted into position using the tower cranes. These were held in position using diagonal raking props until the full section was erected in position, as can be seen in Figs. 11. The composite girders were pre-cambered to account for non-composite and subsequent composite behavior under dead loads from the 4 levels of the RC structure plus the sky-court roof to be constructed above.

*Cantilever blocks*

The Cantilever blocks were erected using diagonal raking props to erect the lowest level and support the blocks steel frame self-weight until the full height bracing

system within the block was installed in its entirety, Fig. 12. The raking props could then be removed, followed by casting of concrete floors. Steel presets were carefully denoted on construction stage drawings to define both presets and cambers to provide flat floors at the time of fit out.

*Main Lobby Components*

The main hotel lobby roof spans 50 m, similar to the main bridge above, to create a spectacular column-free space. This was achieved using 2.5 m deep steel trusses. The main lobby houses some remarkable features such as 'fire and water' glass enclosures around the main entrance façade, two circular glass elevators with cascading water on the outer glass surface, a suspended walkway spanning the full distance across the lobby. There are several large aquariums up to 10m tall formed with acrylic panels located throughout the lobby as a principle feature.

The pre-assembled acrylic fish tanks required a tandem lift with a 400-ton mobile crane combined with a 160-ton mobile crane both sitting on top of the suspended ground floor slab to lift and rotate the tanks; the total weight of biggest fish tank package was 68-tons; temporary steel trusses with mega shores were installed under the suspended slab to transfer the mobile crane outrigger loads directly to the foundation.

## Credits

Owner: ICD
Operator: Kerzner International
Design Architect: Kohn Pedersen Fox Associates
Executive Architect: IBI
Architect of Record: DEC
Hotel Interiors: GA Design International
Residential Interiors: Sybille de Margerie
Water Features: WET Design