

# Wellington Airways Control Tower – Structural Design for Resilience, Case Study

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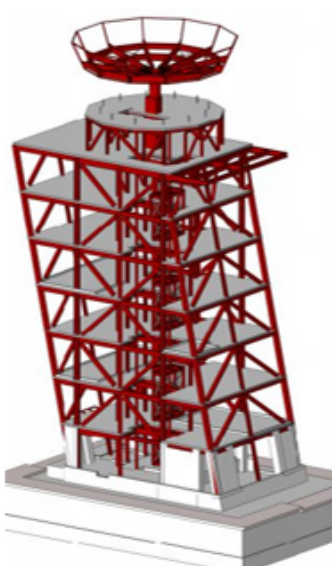
**ABSTRACT:** The new air traffic control tower for Airways Corporation of New Zealand (Airways) is located at Lyall Bay, Wellington. The nine storey structure, references windy Wellington and has been designed to lean into the prevailing northerly wind by architects Studio of Pacific Architecture.

The building's structure has been designed to meet onerous performance criteria, befitting the buildings designation as a critical post-disaster IL4 facility. The building has also been designed to meet client specific wind vibration criteria and to withstand tsunami inundation. Architectural requirements, including the 12.5 degree lean and the column-free tower cab add further to the structural challenges of this project.

The adoption of a base isolated solution allowed far greater freedom to consider more dramatic architectural forms. The leaning structure and single central cab column could not have been achieved without a base isolated solution. At the same time, the cost of the base isolation solution was mitigated by savings in lateral structure and foundations. Careful collaboration between structure and architecture has been vital to ensure a rational structural form is maintained and to avoid numerous transfer structures.

Tsunami inundation was considered a significant risk and a level of structural tsunami mitigation was deemed important to the client. Coastal Engineering advice from Tonkin & Taylor as well as international design guidance, ASCE 7-16 Chapter 6 – draft 2015, Tsunami Loads, helped to inform this process.

The resulting building incorporates a unique structure that addresses the natural hazards of this exposed Wellington site, whilst addressing the complex functional and urban design objectives.



## **1 INTRODUCTION**

The Airways Wellington Control Tower (WCT) project is a new control tower for Airways at the Wellington International Airport. The project has been designed as an Importance Level 4 facility, due to the critical function the control tower will serve in guiding air traffic following a disaster event, such as a large earthquake in the Wellington region.

The current air traffic control tower for the Wellington International Airport is over 50 years old and is reaching the end of its useful life. The new tower is intended to be a landmark structure for Wellington that will provide a high level of resilience against natural disasters, meet the requirements of the New Zealand Building Act, and serve the functional requirements of the airport and the air traffic controllers.

The site is located on the northern side of the corner of Tirangi Road and George Bolt Street in Rongotai. The site is owned by Wellington International Airport Limited (WIAL), and will be leased by Airways for the purposes of constructing and operating the tower.

The tower site is positioned on the southern edge of the Rongotai Isthmus, approximately 250m from the Lyall Bay waterfront. The surrounding landscape is coastal in nature. Below the current car park are soil layers of imported fill, sandy soil, and marine deposits.

With little existing natural or artificial shelter, the site is highly exposed to the weather with prevailing northerly and southerly winds. The winds are laden with salts and residual sand, which creates a harsh environment for built structures. The tower is located in an active seismic zone, and may also be subject to tsunami events due to the close proximity to the coast.

The selected design is a 9-storey (7 office levels, subcab, and cab) Importance Level 4 (IL4) building. The lower 7 floors have an overall footprint of 13.6m x 10.1m. Each floor is offset 0.725m to the North relative to the floor below to create a building lean of 12.5 degrees to the vertical.

The cab level is a 12-sided structure with sloped glazing on each face. To minimise the visual obstruction to controllers in the cab, the roof is supported on a single structural steel tube in the centre of the cab. The roof beams radiate out from the central column and support the head of the cab glazing. The result is a 360 degree structure-free exterior at the cab level.

The building design incorporates low damage design principles—in particular, base isolation—to protect the structure and its contents in very strong seismic activity. Base isolation was chosen for the benefit of providing the utmost seismic protection both for the structure itself and the contents within. The structure above the isolation plane is designed to remain elastic (no permanent damage) in a 2500 year earthquake, a performance which is in excess of the code requirement for a ULS event, but deemed appropriate due to the critical nature of the structure and the geometric irregularity inherent to a leaning building.

Tsunami resistance was also part of the project requirements and so the tower has been designed with concrete shear walls and concrete-filled steel tubes at the ground floor level to provide resistance to a 2500 year tsunami event.

This paper summarises the completed Structural Design of the Wellington Control Tower project. The project is currently in construction with primary structure installed up to Level 6 and steelwork fabrication nearing completion.

## **2 STRUCTURAL DESIGN LOADS AND CRITERIA**

Structural design actions for the WCT project have been determined in accordance with the loadings standard AS/NZS1170 and with a minimum design life of 50 years as required by the Building Act.

## 2.1 Design Earthquake Loads

**Table 1: Design Limit States for Site Specific Uniform Hazard Spectra**

Design Limit State	Return Period	R
MCE	7,500 year*	2.34
ULS	2,500 year	1.8
SLS2	500 year	1.0
SLS1	25 year	0.25

†see site subsoil class below

\*see MCE return period factor derivation below

### 2.1.1 Site Subsoil Class

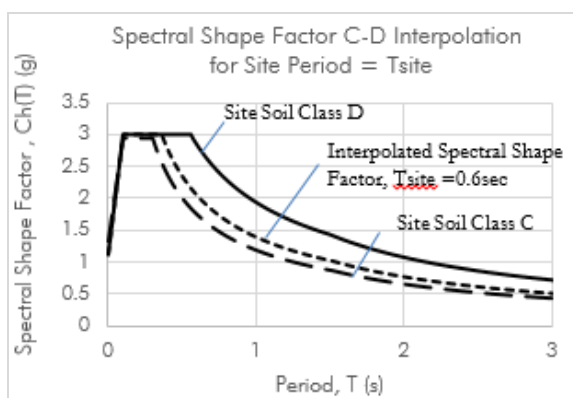
The site subsoil class was assessed by Geotechnical Engineers, Tonkin and Taylor in accordance with NZS1170.5:2004 section 3.1.3.7 to be “Class C” with a site period not exceeding 0.58 seconds.

Tonkin and Taylor recommended adopting an interpolated subsoil class for a site period of 0.58 seconds in accordance with the paper Site-effect terms as continuous functions of site period and Vs30 (McVerry, 2011).

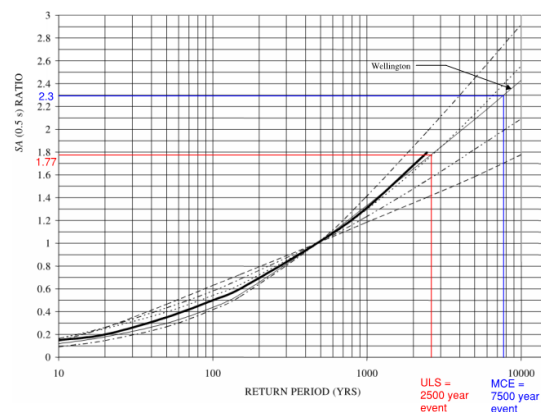
Holmes Consulting adopted an interpolated subsoil class for a site period of 0.6 seconds for the Detailed Design. The interpolation results in a spectral shape factor 17.5% greater than subsoil “Class C”.

### 2.1.2 MCE Return Period Factor

A return period of 7500 years was considered reasonable for the Maximum Credible Event (MCE) for design of the control tower and this was discussed and agreed as part of the Peer Review process. Figure 2, reproduced below, indicates a return period factor of 2.34 and MCE/ULS ratio of 1.3 for the Wellington region.



**Figure1: Spectral Shape Factor Results**



**Figure 2 - Comparison of Proposed R-Factors for New Zealand with Hazard Curves for 0.5s Spectral Accelerations**

### 2.1.3 Structural Performance Factor/Ductility

A Structural Performance factor ( $S_p$ ) of 0.7 (0.85 effective for NLTHA) has been used for the

foundation raft and isolators. Displacements to assess the moat requirements have been multiplied by  $1/S_p$ .

The structural ductility,  $\mu = 1.0$  (elastic) and  $S_p$  of 1.0 have been used for the superstructure design.

#### 2.1.4 Analysis/Design Methodology

The process, summarised below, has been used for the design and verification of the control tower;

1. Preliminary Design of isolation system to achieve a target isolated period and damping was completed using a single degree of freedom spreadsheet.
2. Preliminary Design of superstructure was completed using a base shear specified by ASCE 7-10 for isolated structures. This consisted of application of a static load distribution to a fixed base 3D ETABS model. Due to the significant vertical irregularity of the structure (approximately 60% of the seismic mass being lumped in the ground floor slab positioned immediately above the isolators, relatively low stiffness of cab structure relative to the tower), an approximation of the vertical distribution of the base shear was required.
3. A 3D analysis model was developed based on the Preliminary Design and analysed using NLTHA to develop a refined vertical distribution of base shear.
4. Developed and Detailed design was completed using the 3D ETABS model using the base distribution developed in step 3.
5. Finally the structure was verified using NLTHA.

##### 2.1.4.1 Earthquake Records

Initially the NLTHA was completed considering the maximum response of three earthquake records as required by NZS1170.5. The peer review process raised a discussion of whether the three records adequately represented the seismic hazard for a long period high importance level structure in Wellington, in light of recent research. This led to an additional seven earthquake records being considered. It was found that the initial three records bounded the seismic response of the structure.

## 2.2 Design Wind Loads

A wind tunnel study of the building was carried out by Windtech Consultants to verify that wind design brief requirements, specified by Airways, would be met. A 1:200 scale model including the land topography and surrounding buildings was developed and instrumented with 189 individual pressure sensors. Results of the wind tunnel tests showed that the structure complied with the design brief requirements of limiting wind-induced acceleration at the cab level to below 25 milli-g, and limiting the inter-storey drift under wind loading to 0.2%.

Results from the wind tunnel study were also used to define overall structure and façade design pressures.

## 2.3 Design Tsunami Loads

The current state of engineering practice typically considers wind and seismic as the predominant lateral loading scenarios. The New Zealand Building Code and Loadings Standard (AS/NZS1170) are both silent on specific reference to tsunami loading and/or inundation. However, recent experience, most notably Indonesia 2004 and Japan 2011 has highlighted the real impact of tsunami induced loading in coastal environments.

Significant progress has been made in the last decade toward identifying tsunami hazards and developing design standards, which may allow tsunami design to become a common part of structural engineering practice. Tsunami loading and design is not currently codified in New Zealand, and international guidance is limited but growing. GNS has compiled a report (Review of Tsunami

Hazard in New Zealand – 2013 Update) which identifies the height of the tsunami hazard around the New Zealand coastline for a range of return period events. This report confirms a known and quantifiable tsunami hazard, however, this report does not address any aspects of structural loading or design.

Given the known local tsunami hazard for the Control Tower site (GNS 2013), together with evidence from historic tsunami events across the Rongotai Isthmus and combined with the IL4 status of this building, a decision was made to include a specific tsunami loading criteria into the building design.

### 2.3.1 *Tsunami Loading*

The Design Tsunami loading has been determined in accordance with the 2013 GNS tsunami hazard report and the draft version of ASCE 7-16 Chapter 6, which was released for public comment in June 2015. The inundation level and velocity of the tsunami wave were determined as part of specific Coastal Engineering input provided by Tonkin and Taylor.

There are several different loading scenarios that must be considered for a structure in a tsunami zone, as outlined in ASCE 7-16 Chapter 6, including but not limited to;

- Hydrostatic loading
- Buoyant forces
- Hydrodynamic/impulse loading
- Debris impact
- Debris damming
- Uplift forces/additional gravity load
- Foundation scour

A significant difference between seismic and tsunami load is that the tsunami loading increases much more rapidly as a function of return period than earthquake loading. The return period factor to scale from a 500-year to 2500-year earthquake is 1.8 as per AS/NZS1170. The base shear and overturning moment of a structure would typically scale by a similar ratio. Tsunami wave amplitude at the coast scales by approximately 1.5 times between a 500-year and 2500-year event. However most structures are built above mean sea level, which disproportionately reduces the effect of smaller tsunamis.

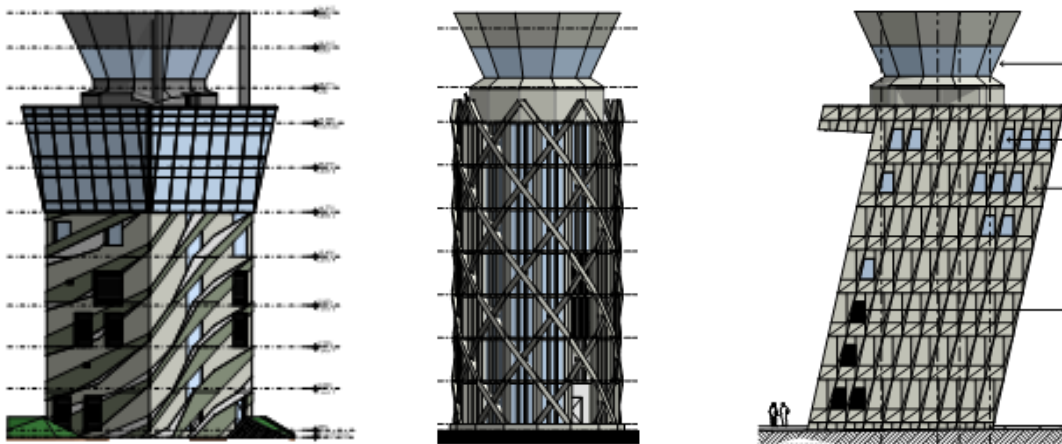
For the Control Tower, the tsunami impact under a 500 year event was low, as water level at the building was assessed to be less than half a storey height and flow velocities at this level were relatively low. For the 2500 year event however, water level impacted the bottom two storeys. Fortunately, for tsunami design, maximum flow velocity and maximum water level are not concurrent and so the governing tsunami load case was for a water level just above first floor. Seismic design loads still governed the global base shear and overturning demands on the structure and so tsunami design was focused on the hydrostatic and impact lateral loading of specific components.

Local member lateral force demands at ground floor level under tsunami loading were far greater than the capacity of the conventional steel braced frame system, and in particular for the longer length diagonal brace elements. As such a concrete blade wall solution was adopted at ground floor level in order to accommodate large impact force demands whilst maintaining continuity of the braced frame system from above. In addition, critical load bearing column elements at ground floor were changed to larger diameter concrete filled CHS columns (as opposed to steel UC sections above).

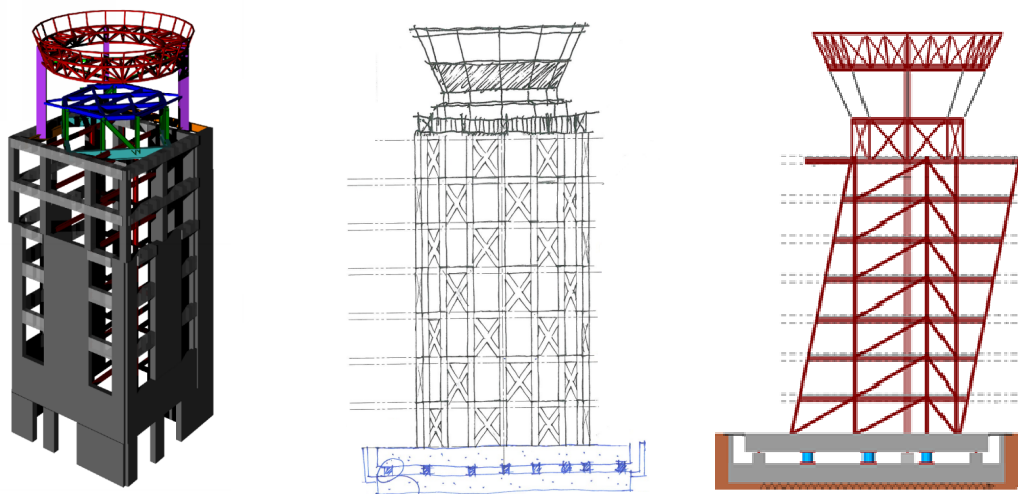
## 3 **STRUCTURAL OPTIONS**

Numerous options, both in terms of site location and building form were considered during the Feasibility and Concept Design phases of this project. Several early schemes drew heavily on project requirements and experience gleaned from the earlier Christchurch Control, successfully completed in conjunction with Hawkins Construction. Further schemes were subsequently

developed in part due to the requirement for a Resource Consent Hearing process, and further consideration of urban design imperatives. A selection of scheme options are outlined below.



**Figure 3: Selection of Architectural Schemes (courtesy of Studio of Pacific Architecture)**



**Figure 4: Corresponding Structural Concept Design Options (Holmes Consulting)**

The project started as a Design Build bid process with our project team led by Hawkins Construction. Early design schemes (not shown) developed at that stage and based on a prescriptive client brief, at arm's length from the client, were found to not meet preferred client outcomes. Airways subsequently changed the project team structure and entered into an Early Contractor Involvement (ECI) process with Hawkins Construction and their existing Design Team.

A more collaborative Client and Design Team relationship through this process focussed initially on space planning opportunities, with a conventional design (non base-isolated) square tower system with the three column cab roof support structure. Structural challenges with this design were significant and the extremely high seismic loads led to more structure, a stiffer building and consequently even higher loads. To compound problems there was little relationship between the three column supported cab structure and the square tower below leading to the need for significant transfer structure below cab level.

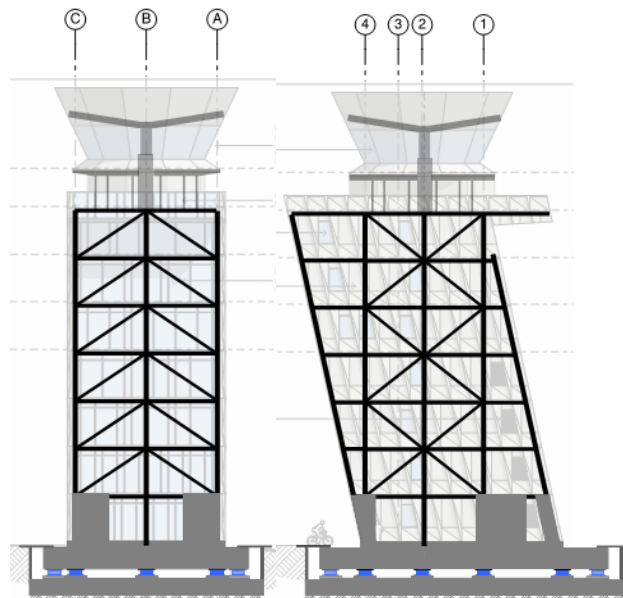
A subsequent design workshop focused more on structural and foundation drivers and opportunities and as part of this a base isolated solution was considered. Contrary to conventional thinking, it was shown that base isolation could be implemented at no additional

cost to the project. This was in part possible due to the fact that the required double layer of foundation structure could be positioned below ground in a zone of poor soil which was suited to a “dig out” solution.

Base isolation and the resultant significant decrease in seismic load demands on the superstructure, provided significantly greater architectural options and also allowed for a simplified central column support system to the cab roof, reducing the need for complex transfer structure.

#### 4 STRUCTURAL SYSTEM OVERVIEW

The selected structural system is a leaning base-isolated concentric steel braced frame structure. The predominant gravity load path is located within the central vertical portion of the building, directly below the cab structure. The primary lateral load resisting frames are located around the perimeter, with two bays of bracing on each face of the building. At the ground floor level the steel columns are protected by concrete shear walls which provide resistance to a tsunami event and also act as the main lateral load resisting system for a seismic event at this level.



**Figure 5: Arrangement of the primary structural system**

Ground conditions on site consist of deep marine deposits in various layers over bed rock at 45m depth. The top 3.5-4m of marine deposits were medium dense sands and gravels prone to liquefaction and considerable settlement. The presence of suitable founding material below that layer meant that a dig out of the top 3.5-4m of soil could be integrated well with a base isolation double foundation system. The significant reduction in seismic overturning demands associated with the base isolated structure resulted in a much smaller base raft slab than previously considered and so saving in the foundation size alone, essentially covered the cost of the base isolators.

Significant structural Design Features include;

- The superstructure is designed to remain elastic in a 2500 year earthquake (ULS) and uses a concentric braced steel frame system. Any ductility in the superstructure is precluded due to the leaning tower and the ratcheting effect that would otherwise occur with a ductile system.
- The structure is designed to resist 2500 year tsunami loading in accordance with the draft version of ASCE 7-16 Chapter 6. This led to a concrete walls and CHS column solution at ground floor.
- The building is base isolated with 13 lead rubber bearings, 1020mm diameter, in between the



raft foundation and an isolated concrete slab.

- Support and restraint of the cab roof was designed to limit obstructions and maximise views from the air traffic control room within the cab. Initially, the proposed system consisted of a single central column with tension rods around the perimeter. As the design progressed, analysis of the structure using a non-linear dynamic procedure identified the cab roof was sensitive to dynamically amplified and resonant torsional response. The size of the central column was increased to raise the torsion stiffness of the roof structure and reduce the resonant response. The larger central column also increased the lateral stiffness thus negating the need for the perimeter tension rods. Columns and braced frames below the cab level are on a radial grid, and sit on transfer beams to transmit loads to the orthogonal structure below. This presented a significant co-ordination challenge which was reduced in complexity following removal the perimeter tension rods to the cab roof.

## 5 CONCLUSIONS

The building form and design requirements have created a set of structural design challenges that are far greater than a typical commercial development of similar scale. Close collaboration with all project team members has been crucial to ensure that these challenges have been addressed and resolved through the design process.

The resulting design answers these challenges but in a very rational and conscious manner, resulting in a design that is both efficient and functional. We look forward to seeing the structure completed later this year.

## 6 ACKNOWLEDGEMENTS

The team at Holmes Consulting would like to acknowledge the client, Airways, for their willingness embark on this challenging design solution and in proactive in addressing challenges posed throughout the course of this project. Acknowledgement of the wider consultancy and contractor team is also warranted and without which this project would not have been a success. Key parties included;

Client: Airways Corporation of New Zealand

Architect: Studio of Pacific Architecture (with Paris Magdalinos Architects – cab design specialist)

Contractor: Hawkins Construction

Building Services: Beca

Fire Engineering: Holmes Fire

Structural Peer Review: Dunning Thornton Consultants

Quantity Surveyor: RLB

## 7 REFERENCES

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