Case Study: Te Waihorotiu Railway Station Seismic Considerations for the Oversite Development

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ABSTRACT

City Rail Link (CRL) engaged Link Alliance (LKA) to design and build all new station structures on Auckland's NZ$5.493B underground railway extension project. Te Waihorotiu is a midtown station south of Waitematā Station (Britomart). Constructed under Albert St, Te Waihorotiu is forecast to be New Zealand's busiest train station with entrances located adjacent to Aotea Square and Auckland's Sky Tower.

The Aotea Square Station Entrance Building (SEB) is a four-storey moment frame structure above ground with three basement levels connecting to the 350m long underground station. The design needs to cater for a future Oversite Development (OSD) that overlies the CRL building envelope and adjacent land.

Stability elements are shared with the main building core located adjacent to the CRL building and is not central to the building mass. The OSD building slopes from 24 storeys to 7 storeys which shifts the relative centre of mass from floor to floor. This paper looks at the design challenges and solutions to control torsional behaviour and large interstorey movements to achieve a robust design.

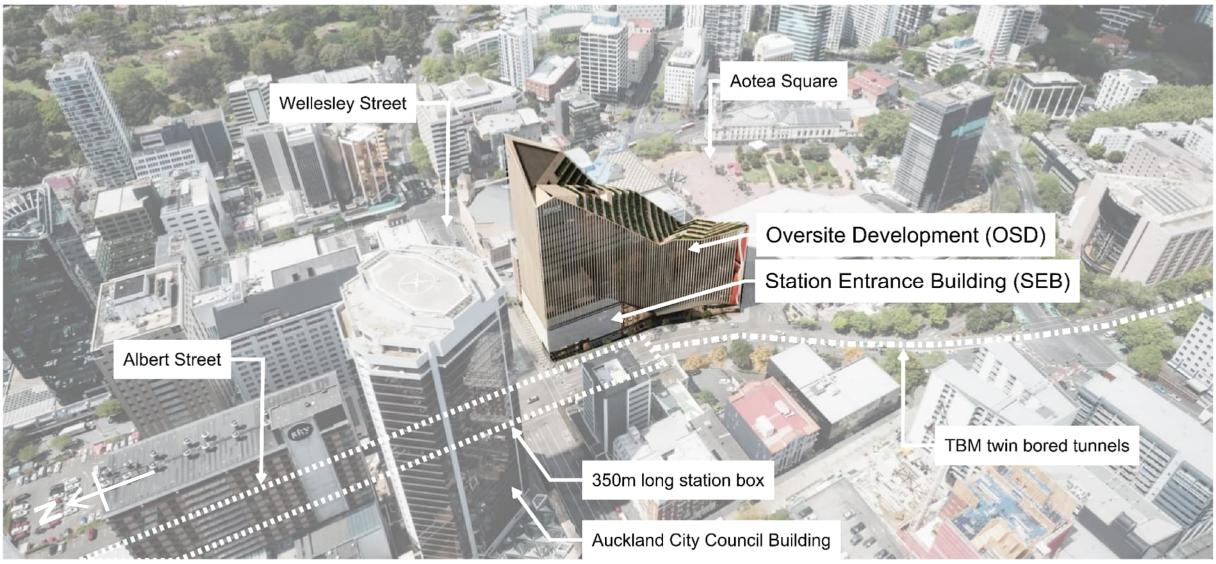


Figure 1: Oversite Development 3D Render

1 INTRODUCTION

At the completion of CRL, the train network will double its capacity to 54,000 passengers in peak hours. 3.45km of new railway line will be built, including 3.2km of tunnel connecting Waitematā Station (Britomart) to Maungawhau Station (Mount Eden) via 2 new underground stations - Te Waihorotiu (Aotea) and Karanga-a-Hape.

Te Waihorotiu's Station Entrance Building (SEB) shown in Figure 1 is located on the corner of Albert St and Wellesley St in the heart of the CBD adjacent to Aotea Square. The 4-storey building with three basement levels supports the front of house services but mostly houses plant for the functional requirements of the station.

The Oversite Development (OSD) otherwise known as the Symphony Centre is a 24-storey mixed use building that slopes down to 7 storeys on the south side to ensure Aotea Square is bathed in sunlight.

This paper outlines all the key seismic considerations that were taken to ensure that the SEB and its complicated interaction with the OSD met the project Minimum Requirements (MR's) and Building Code.

2. SEB/OSD INTERFACE

The requirements of the building layouts were set out in the MR contractual document and resource consent plans. The design of the OSD was awarded and developed separately with the overarching resource consent requirements and additional OSD Interface Design Report prepared by LKA and associated interfacing IFC drawings. The onus is then on the OSD designer, who will apply for consent last to demonstrate that they comply with the parameters established by the consent and LKA.

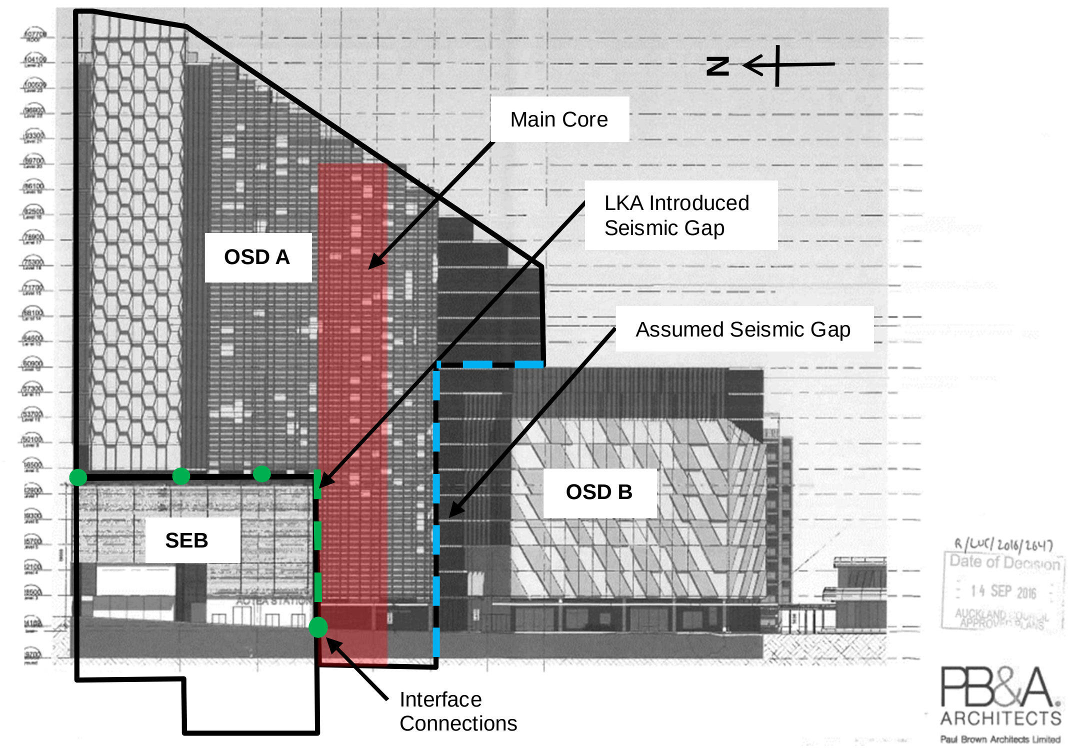


Figure 2: Resource Consent Drawings West Elevation

Based on principles set out in the consenting documents LKA have assumed that a seismic gap with double columns is located between the OSD A and OSD B buildings and therefore LKA have only modelled the OSD A building in the structural analysis model without any additional mass applied from OSD B (Fig. 2).

The OSD building envelope encompasses the SEB and the area immediately south of the SEB of similar footprint. As seen in Figure 3, the floor plates do not align at the horizontal interface and there are numerous openings. As such, LKA's designers decided to add another seismic gap above L0 to the underside of the OSDs level 8 which reduces the number of structural interface connections and transfers via columns and shear core.

The OSD connections have been catered for in the SEB design with 12No. SEB roof connections directly above columns, a 9m x 600 shear wall above the southern capping beam and another 2No. L0 column connections (Ref. green dots in Figs. 2-3). The SEB roof connections were designed so that the HD40 reinforcement bars in the column are anchored before the OSD is built and act continuously with the OSD after it is built using a combination of couplers and terminators (Fig. 3).

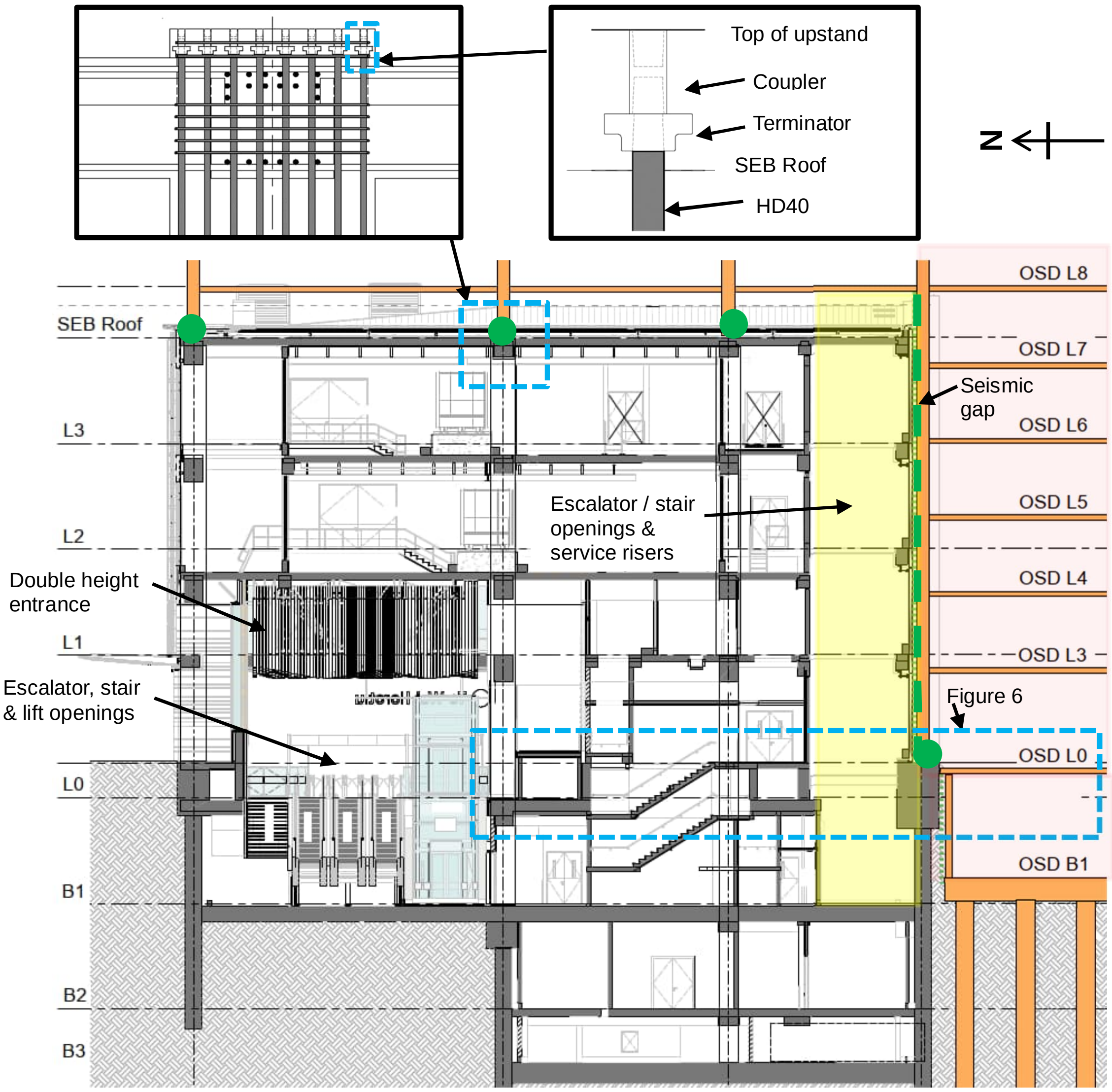


Figure 3: Architectural Interface Section Looking East

3. LATERAL RESISTING FRAME

Frame Without OSD

The SEB is a reinforced concrete frame building built top down with diaphragm walls and plunge columns. The superstructure floor plates comprise of prestressed precast planks with insitu topping (Fig. 4).

Lateral loads arising from wind and seismic actions are transferred from the floor plates acting as diaphragms to the lateral resisting frames consisting of a moment frame above ground and diaphragm walls acting as shear walls below ground. On the northern side there are three large blade columns and deep beams creating a stiff moment frame resisting lateral load from the OSD steel braced frame above.

The presence of the blade columns on the northern elevation means that the centre of stiffness is offset to the north, which causes some eccentricity above L0 for ground accelerations in the east west direction.

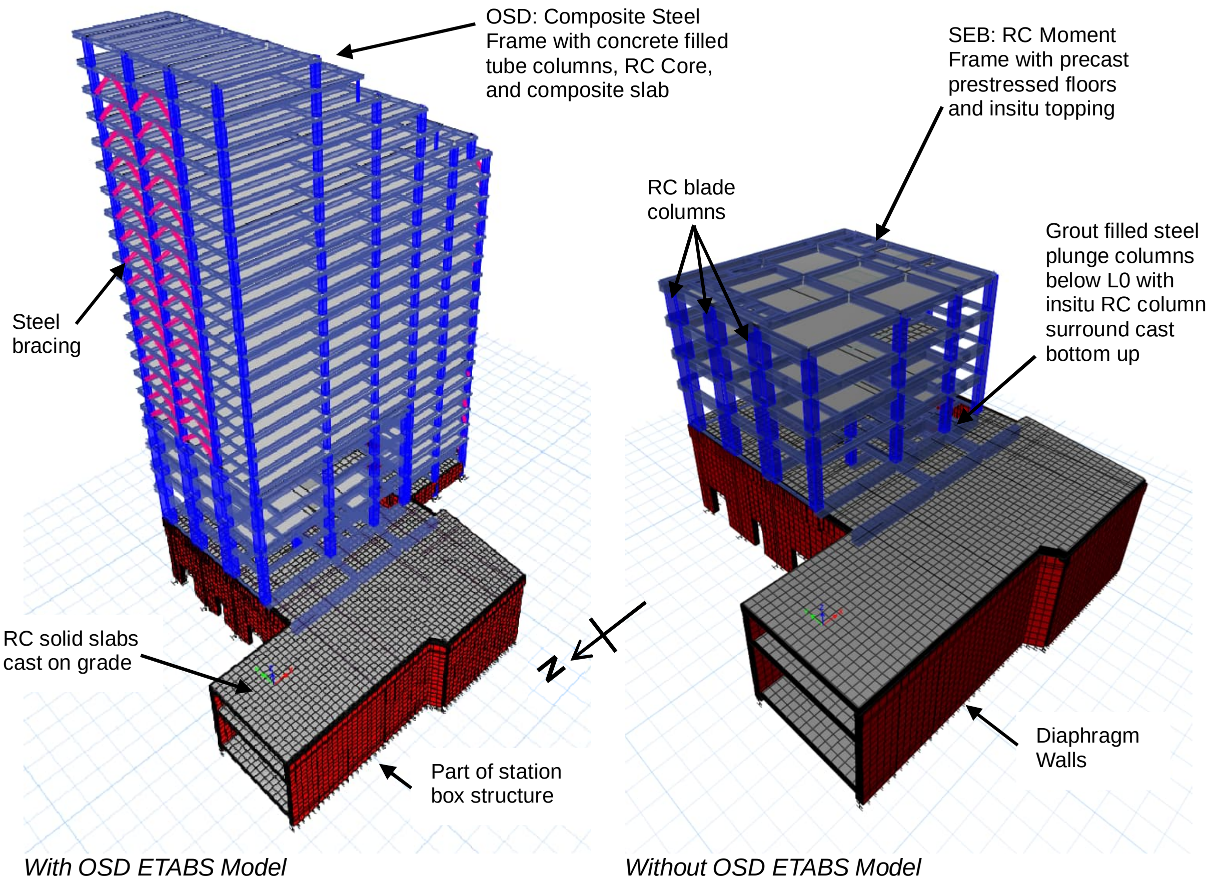


Figure 4 – Full 3D ETABS analysis models

Frame With OSD

The OSD is assumed to be a steel composite construction using concrete filled tubes for the columns and composite one-way spanning slabs on steel beams. The main core will be reinforced concrete supported on a large capping beam supported on piles. Concentrically braced steels are adopted for the perimeter bracing (Fig. 4).

The main stability element in the combined scheme is the large core that sits partially on the southern perimeter diaphragm wall and mostly in the OSD only zone from L0 to L19. As this core is not in the centre of the combined building another bracing line was added at the northern elevation. A perimeter moment frame was also assumed in the OSD that creates a dual system to better resist torsion arising from the offset in stiffness and mass centres at each floor of the sloped building (Fig. 5).

The inertia loads from the OSD above L8 will transfer to the structure below on either side of the structural gap. The distribution of actions depends on the relative stiffness between the SEB and OSD structures. At the northern end of the SEB, the reinforced concrete moment resisting frame (MRF) is enlarged to "blade columns" to provide enough capacity under the large seismic demand from OSD concentrically braced frames (CBF) above L8.

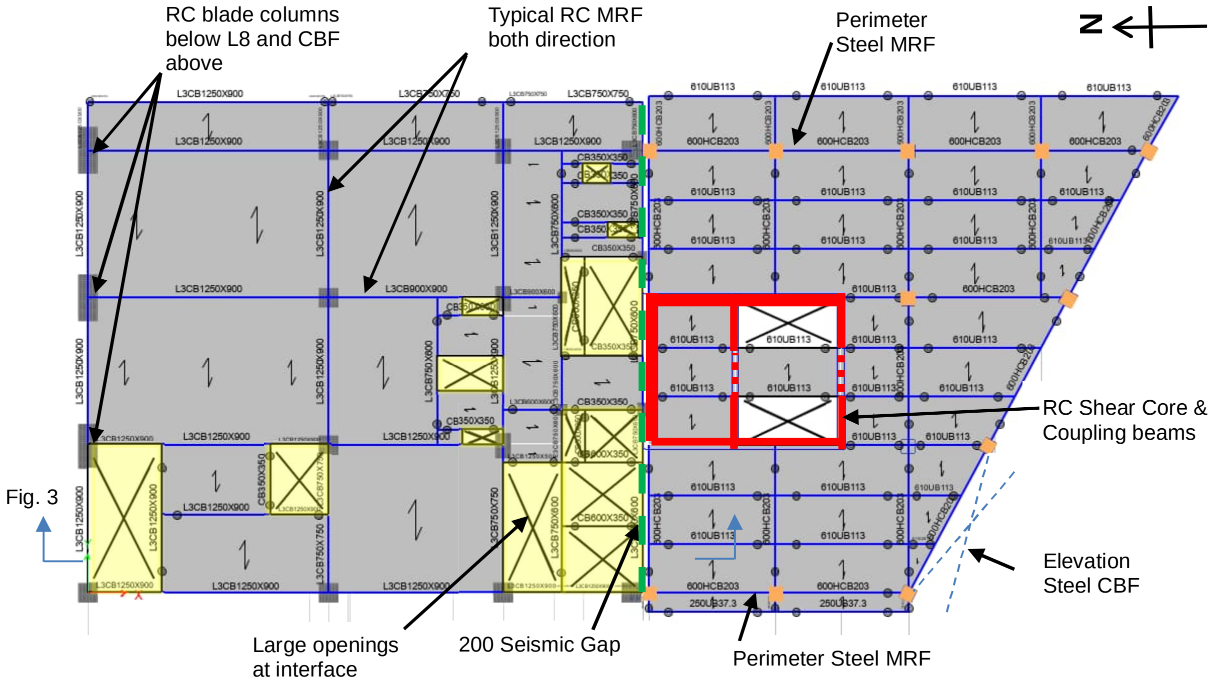


Figure 5: ETABS Model Plan L3 (OSD L6) Showing Stability Elements

4. SEISMIC DESIGN CRITERIA

Te Waihoritiu Station is required to be an importance level 3 building with a 100-year design life, which means the ULS seismic design needs to cater for a 1/2500 annual probability of exceedance which is the highest level in New Zealand according to AS/NZS 1170.0 and 1.8 times higher than a typical building designed for a 1/500 annual probability of exceedance.

SLS1 annual probability of exceedance is 1/25. Under an SLS1 earthquake no repairs should be required, which mostly effects architectural and services isolation details. Key seismic data is provided in Table 1.

ETABS models have been built to design the SEB with and without the OSD subject to the above requirements. Due to the horizontal and vertical building irregularities, a modal Response Spectrum Method (RSM) approach was adopted. The base shears from RSM were scaled to at least 100% of those from the Equivalent Static Analysis (ESA) as per NZS 1170.5 Cl 5.2.2.2 and applied in two orthogonal directions of 100% and 30% as per Clause 5.3.1.2. P-delta effects have also been considered according to NZS1170.5 Cl 6.5.

The first period is 2.66s and 1.18s for the With OSD mode and Without OSD mode respectively, which means the floor accelerations are 2.2 times greater in the Without OSD case. The periods calculated by ETABS were also cross checked using the Rayleigh method according to NZS1170.5 Cl 4.1.2.1.

The building superstructure is designed as a nominally ductile structure with beams designed to μ = 1.25 and structural performance factor of Sp = 0.7. The columns and beam column joints are then assessed in accordance with NZS3101 Cl 2.6.2.2.2 to ensure that a strong column weak beam structure is maintained to avoid a soft storey mechanism forming during a significant earthquake event.

The minimum requirements document stipulated an elastically responding structure for the below ground elements but were silent on the impacts to the superstructure. Higher ductility levels were not considered for the superstructure as in the With OSD case potential damage rectification due to plastic hinging would pose an interface difficulty for a building of two owners.

Table 1: Seismic Requirements

Parameter	**SLS1 Earthquake**	**ULS Earthquake**
Site Subsoil Class	C	
Hazard Factor (Z)	0.13	
Design Life	100 years	
Importance Level	3	
Annual Probability of Exceedance	1/25	1/2500
Return Period Factor	0.25	1.8
Structural Performance Factor (Sp)	0.7	0.7
Near Fault Factor	1.0	1.0
Ductility Factor (μ)	1.0	1.25 (above ground) 1.0 (below ground)

5. DIAPHRAGMS

L0 Diaphragm

It was critical that the base slabs between the OSD and the Station Box were connected because the stability elements are shared. The main difficulties in connecting the two structures together were: 1) There are many large openings at the L0 interface (Fig. 5); 2) The SEB L0 slab steps; 3) Uncertainties over final OSD levels; and 4) Considerable kickback effects amplify the axial loads in the diaphragm due to the stiffness difference between above and below ground.

The solution was to 1) introduce a double slab system to maintain a constant lower-level diaphragm tied to the capping beams; 2) Allow diaphragm forces from the OSD to connect to the capping beam at varied levels by checking the capping beam for torsion; and 3) utilising the deep beams from the double slab depth to resist torsion (Fig. 6).

A strut and tie model of the L0 diaphragm was assessed by building a 2D grillage model with the software Space Gass. The inertia loads, ground pressure loads, and transfer loads applied to the model were based on equivalent static analysis scaled to the modal response analysis. The capping beams and other internal beams were denoted as struts together with other slab struts at around one-metre centres.

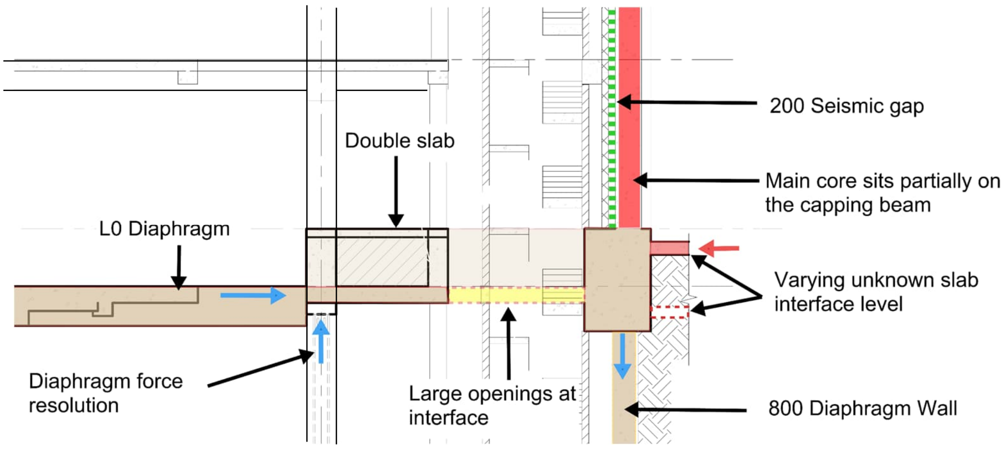


Figure 6: L0 Diaphragm Interface in Section (Detail from Fig. 3)

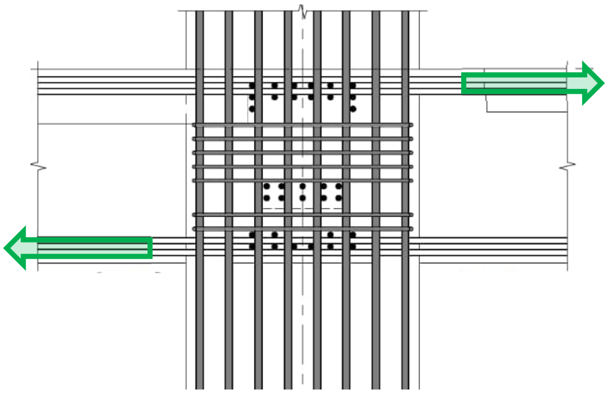
L1-L4 Diaphragms

The diaphragms above L0 were assessed using a Pseudo Equivalent Static Analysis (pESA). The storey inertia loads were applied to the ETABS floor plates to determine the shell stress under seismic actions. A grillage model was also developed to check the results.

Before the introduction of the seismic gap at the building interface, LKA considered to tie the building at all levels, but the OSD core had insufficient capacity to resist the shear loads imposed by the heavy SEB for the consented wall size and increasing the wall size further created other issues by changing the stiffness of the core and seismic behaviour of the building. Whilst some shear is transferring through the L8 diaphragm via the columns it is far less than would be the case if all levels were directly connected.

6. MOMENT RESISTING FRAME

The frame actions were assessed according to section 4 above. This required the plastic hinge zones adjacent to the columns to be detailed as limited ductile elements as listed out in NZS3101 Cl 2.6.7 and the columns/ beam column joints were checked for 1.15 x the moment capacity of the beam as amplified design actions. Furthermore, the beam capacity was increased by considering some contribution from the flange reinforcement in the slab in accordance with NZS3101 Cl 9.4.1.6



Beam Column Joints are checked for 1.15 x M_{cap} to allow for some plastic hinging in the beams

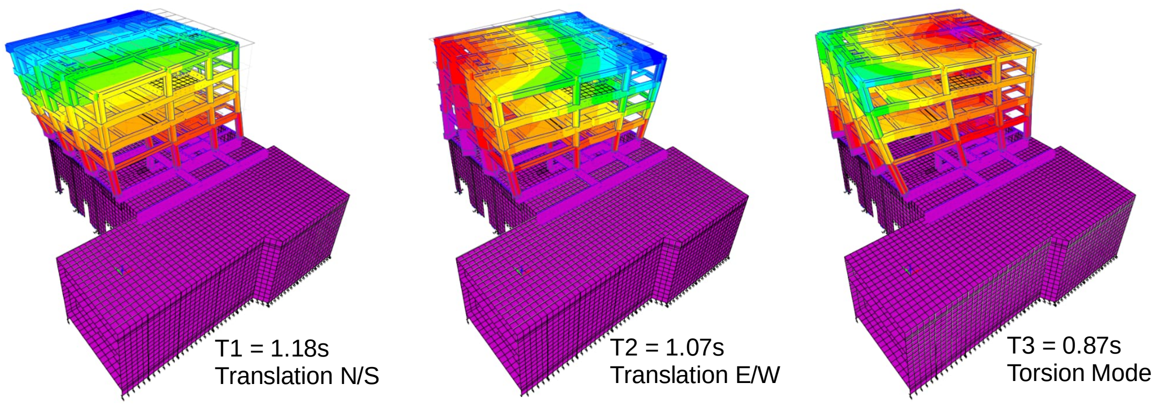
AND

The hinge zones are detailed as limited ductile beams

Figure 7: Beam Column Joints for μ = 1.25, Sp = 0.7

The reference design showed steel bracing instead of blade columns, but this was changed within the SEB by LKA to improve buildability of the reinforced concrete frame. The stiffness of the blade columns was carefully sized to be big enough to resist the seismic demand from the above OSD steel bracing but small enough to limit torsion in the Without OSD case. One check to determine if torsion is limited to an acceptable level is to ensure the first modal shapes are translational modes in both horizontal directions (Fig. 8).

Structural walls around the lift shafts and stair core at the southern end were considered but removed because they 1) attracted too much seismic load causing more eccentricity issues and 2) space proofing could not allow for the wall size needed to resist the demands. As such the walls around the southern openings are secondary blockwork walls.



First Modal Shape *Second Modal Shape* *Third Modal Shape*

Figure 8: Modal Shapes of the Without OSD Design

7. FOUNDATIONS

The building is founded on diaphragm walls that both retain the basement and transfer stability loads to the ground through a combination of bending resisted by passive earth resistance, side friction and end bearing. The toe depth of the panels varied significantly to cater for the loading requirements and diaphragm wall construction sequencing strategy. Bearing panels supporting the heavy column loads up to 45MN into rock were significantly longer than the retaining only panels. Loads from the columns are spread out through the capping beams. The toe of the bearing panels are also deep enough to prevent surcharge of the future North Shore line (NSL) tunnels and to cater for any softening after they are built (Fig. 9).

The diaphragm walls were modelled in ETABS as isolated walls with joints between panels down to a common level below the rock layer where out of plane bending would be negligible and they are supported on springs laterally, vertically, and rotationally. The spring stiffnesses were provided by the geotechnical engineers to account for passive earth resistance, side wall friction and end bearing resistance with settlement limited to 10mm under ULS vertical loads. This is more stringent than typical settlement limits to control differential settlement issues with the adjacent tunnel box which had settlement closer to 2mm. Verification of the spring stiffness was an iterative process and terminated when the previous analysis yielded reactions within 5% of the latest version. The OSD is assumed to be piled with some additional piles (from the consenting documents) under the core to reduce differential settlement issues in the "With OSD" model. The models were checked for variations in spring stiffnesses of +/- 25 % vertically and + 100% /-50% laterally.

The diaphragm walls were also checked to limit crack widths (except seismic/ transient load cases) in accordance with NZS3106 to enable the concrete to autogenously repair its cracks and thereby minimise water ingress. Out of plane bending demands from geotechnical Plaxis models considering the top-down staged construction were enveloped with the ETABS wished in place demands with some adjustment for creep effects. Seismic out of plane demands were also checked in the Plaxis models applying an allowance for the superstructure inertia loads from the ETABS model with guidance from figure 5.7 of the Bridge Manual (NZTA 2013).

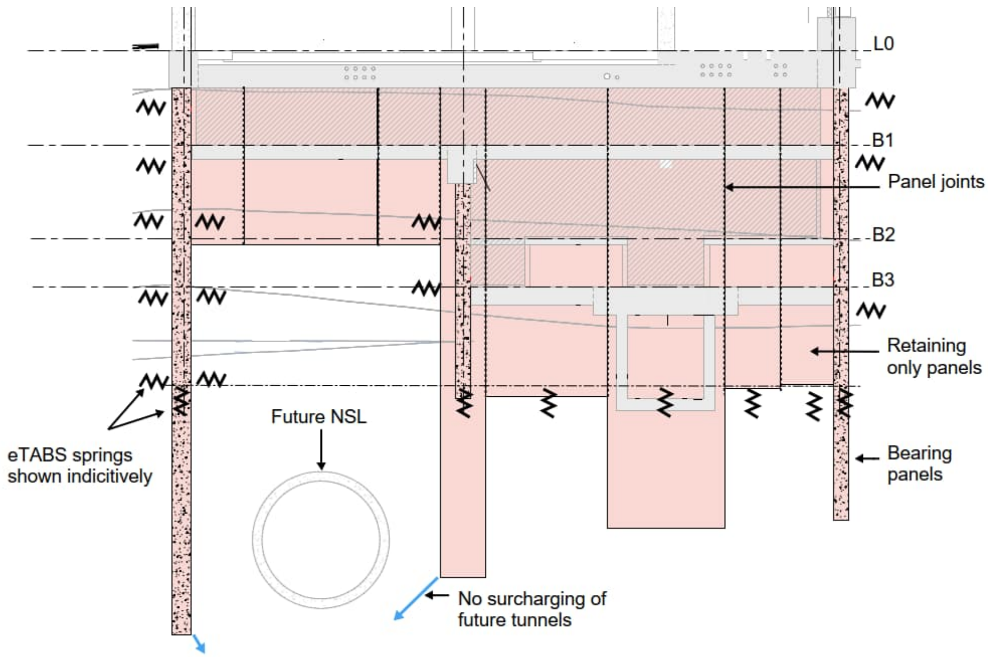


Figure 9: Eastern Diaphragm Wall Looking East

8. DISPLACEMENT CONSIDERATIONS

The strategy of dealing with displacements under seismic loading is to: 1) Isolate all primary structure, precast façade, blockwork, and life safety services for ULS movements or 1.5 times where NZS3101 Cl 17.6.2 applies. As the moment frame is not as stiff as a braced frame, ULS interstorey drifts are up to 70mm at according to NZS1170.5 Cl 7.3. Whilst this is a large movement, it is still well within the inter storey drift limits of 2.5% set out in NZS1170.5 Cl 7.5.1, which would be as high as 170mm. 2) No repairs are permitted under SLS1 movements. Architectural details like light weight partitions and all non-life safety services are isolated for SLS1. Some of the displacement impacted structural design elements are discussed below.

Precast floors

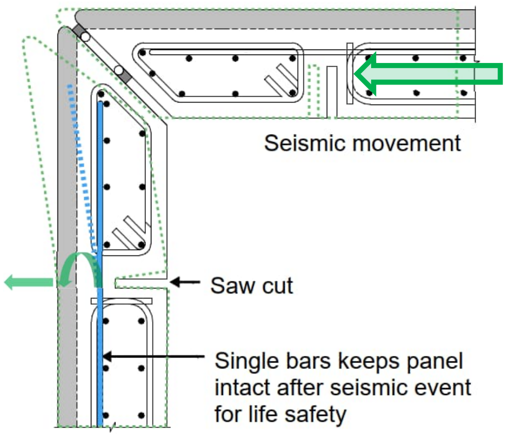
The prestressed precast floor seatings were checked according to NZS3101 18.8.1 ranging from 125mm to 175mm. As an Sp factor of 0.7 was adopted, the assessment included an estimate for plastic hinge elongation and beam twisting based on a limited ductile approach.

Blockwork Partition Walls

The blockwork was designed as partition walls with a fixed base and a free in plane pinned out of plane top connections via L brackets clamping the walls. However, due to the high levels of ULS interstorey drift there will be significant yield in the centrally placed reinforcement. As such it was designed using NZS1170.5 provisions for Parts & Components with limited ductility of $\mu_p = 2$. The base connection utilised well anchored HD10 bars lapping to HD16 bars above the plastic hinge zone.

Façade

The precast façade is articulated in the same way to the blockwork walls but using bolts in grout filled tubes at the base and oversized holes at the top connections to cater for both 1.5 x ULS movement and construction tolerances (Fig. 10). At two of the corners, to keep the joint size to a minimum, frangible panels were adopted that will bend under ULS movement but not collapse and cause harm to the public.



Frangible Corner Joint Detail

Typical Façade connections during installation

Figure 10: Façade Details

9 CONCLUSIONS

Te Waihorotiu Station's SEB and OSD building envelope is predefined contractually which complicates the seismic performance including 1) a sloping OSD building 2) different floor levels at the horizontal interface; 3) large openings at the building horizontal interface; 4) the main core housed outside of the station building; 5) the building needs to satisfy both with and without OSD modes; and 6) needs to satisfy the return period of 1:2500 years.

The response from LKA was to; 1) add a moment resisting perimeter frame within the OSD to resist torsion arising from the sloping building; 2) add a seismic gap at the horizontal interface to simplify shear flow down through the building; 3) utilise the stiff diaphragms at level 8 and level 0 with a double slab at the level 0 interface to control torsion effects; 4) change the braced frame on the northern side to a blade column Vierendeel system tuned to minimise torsional behaviour in the without OSD mode and; 5) remove the structural walls around the southern openings that were too stiff and overloaded with the space proofing available widths.

The eccentric behaviour effect in the Without OSD mode coupled with the greater building stiffness and lower accelerations in the With OSD mode meant that some of the beam bending moments are greater in the Without OSD mode. LKA has designed the two building modes for a nominal ductility and optimised the reinforcement by taking advantage of the reduced structural performance factor of 0.7 with limited ductility detailed hinge zones and amplified actions according to NZS3101 Cl 2.6.2.2.2 that ensure the strong column weak beam philosophy is maintained.

The design has required a rigorous and iterative process to achieve compliance of the building codes and robustness checks to prevent a soft storey from developing and ensure a watertight building.

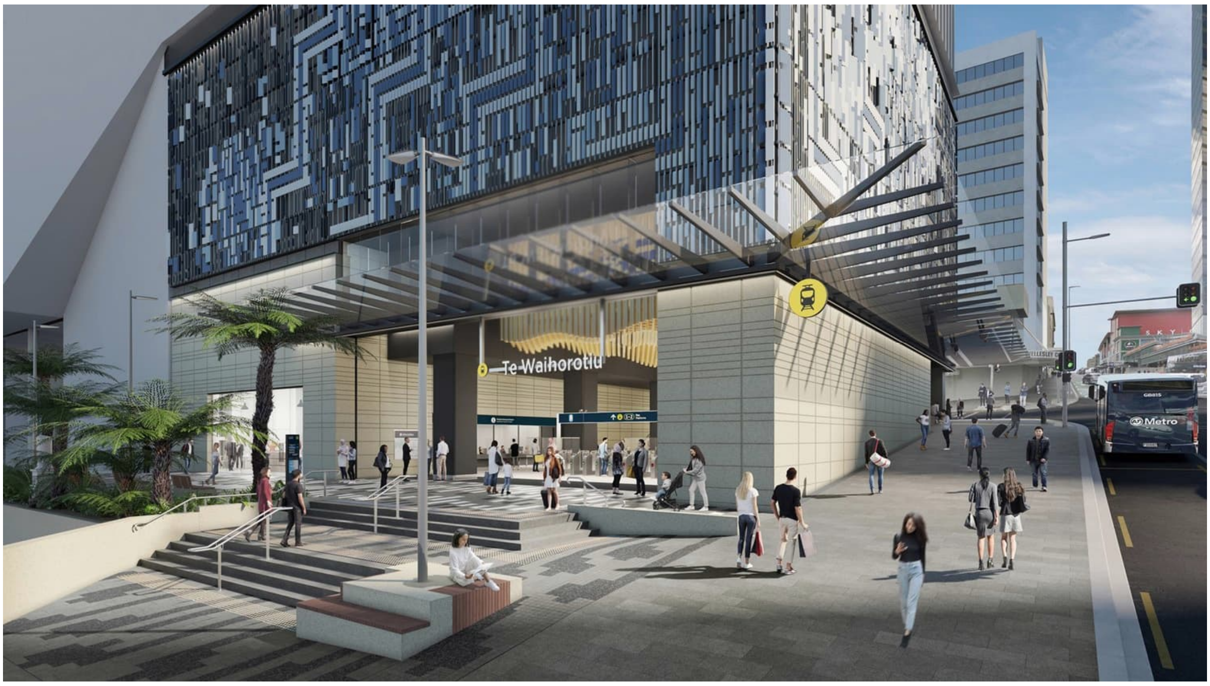


Figure 11: Te Waihorotiu Station CGI of Wellesley St Entrance

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REFERENCES

AS/NZS1170.0, 2002. Structural design actions, Part 0: General principles. Sydney/Wellington: Standards Australia Ltd/Standards New Zealand

NZS1170.5, 2004. Structural design actions, Part 5: Earthquake actions – New Zealand. Wellington: Standards New Zealand

NZS3101, 2006. Concrete structures standard, Part 1: The design of concrete structures, Part 2: Commentary on the design of concrete structures. Wellington: Standards New Zealand

NZS3106, 2009. Design of concrete structures for the storage of liquids. Wellington: Standards New Zealand

NZTA, 2013. SP/M/022 Bridge manual. Wellington: The NZ Transport Agency