Seismic Design of the New Zealand International Convention Centre (NZICC)

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ABSTRACT: The New Zealand International Convention Centre (NZICC) will be a premier event, exhibition and convention centre in Auckland and New Zealand, featuring 8,100m² exhibition floor, a 2850-pax plenary hall and 2700m² of meeting and breakout rooms. The architecture concept for a vertically stacked and flexible spaces and the need for long span structure over double-height, column free spaces resulted in a very challenging structural design. The building is located on a sloping site between Hobson and Nelson Streets with nearly 8m retained soil, adding out-of-balanced seismic loading into the design consideration. The overall building comprises four levels of basement below Nelson Street and six levels of suspended floors.

The structural response to the design brief was to have long span steel trusses on tall columns over the plenary and exhibition levels. These deep trusses and varying floor plates across levels create a vertical irregularity and a potential column-sway zone in the Level 3 floor. Buckling-restrained braces (BRBs) frames are selected as the primary ductile lateral load bracing system, acting in parallel with the moment-resisting frames formed by the primary cruciform columns and storey-deep trusses. Several reinforced concrete core walls support the northern circulation building, which is tied back to the main structure at Level 3 and 6.

This paper describes the seismic design challenges and innovation used in the NZICC structure. A direct displacement-based design (DDBD) approach was used in parallel with a more convention code-compliant force-based design for the superstructure lateral load resisting systems. Given the vertical irregularity and the capacity design approach – a displacement-based approach allowed for more in-depth analysis of the proposed structures and an improved understanding of their inelastic seismic behaviour, particularly the interaction between the BRB and truss portal frames. The specific challenges and design methodology used for the dual BRB connections, transfer diaphragms, basement and superstructure shear walls are also discussed.

Keywords: Buckling-restrained braces, displacement-based design, seismic design, deep basement, dual system

1 INTRODUCTION

Beca Ltd (Beca) has been commissioned by New Zealand International Convention Centre Ltd (NZICC Ltd) to undertake the design and documentation of the Structural and Geotechnical engineering works associated with the New Zealand International Convention Centre (NZICC) on a city block between Hobson St, Wellesley St, Nelson St and the TVNZ building on Victoria St.

This paper describes the seismic design challenges and innovation used in the NZICC structure. A direct displacement-based design (DDBD) approach was used in parallel with a more convention code-compliant force-based design for the superstructure lateral load resisting systems. Specific challenges and design methodology used for the dual BRB connections, transfer diaphragms, basement and superstructure shear walls are also discussed.

2 DESCRIPTION OF THE PROJECT AND STRUCTURE

2.1 Overview

The New Zealand International Convention Centre (NZICC) will be the premier event, exhibition and convention centre in Auckland and New Zealand, featuring 8,100m² exhibition floor, a 2850-pax plenary hall and 2700m² of meeting and breakout rooms. The architecture concept for a vertically stacked and flexible spaces and the need for long span structure over double-height, column free spaces resulted in a very challenging structural design.

The building is located on a sloping site between Hobson and Nelson Streets with nearly 8m retained soil, adding out-of-balanced seismic loading into the design consideration. The overall building comprises four levels of basement below Nelson Street and six levels of suspended floors. The structural response to the design brief was to have long span steel trusses on tall columns over the plenary and exhibition levels. These deep trusses and varying floor plates across levels create a vertical irregularity and a potential column-sway zone in the Level 3 floor.

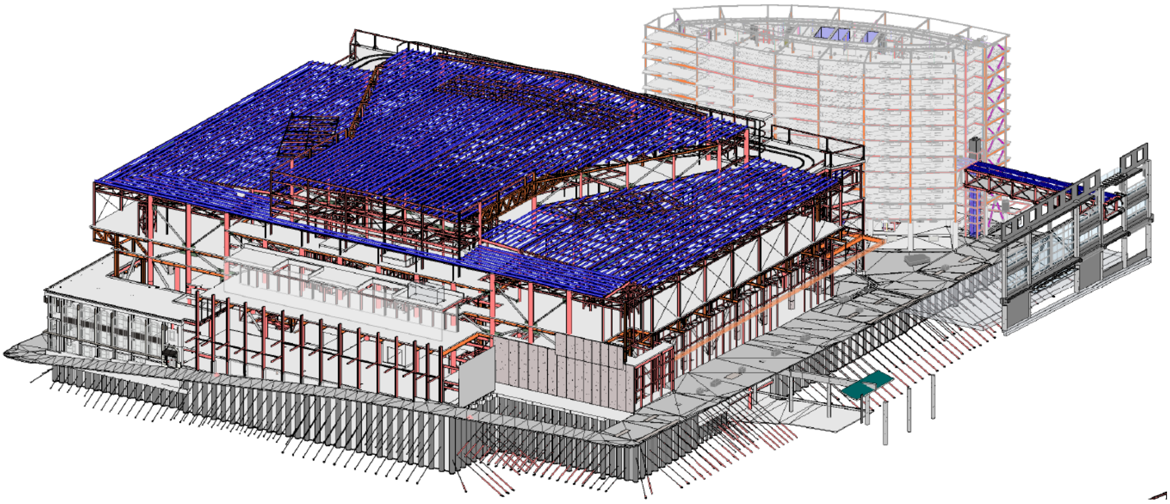


Figure 1: 3D perspective of NZICC Structure, viewed from the south-eastern corner.

2.2 Key seismic design principles

The lateral load system is required to resist loads from seismic (earthquake), wind and ground retention soil pressure in accordance to AS/NZS1170. For the superstructure above Level 3, seismic load is the governing lateral load condition while for the substructure below Level 3, the seismic load in conjunction with ground retention loading are the governing lateral load condition. Wind loading is important at serviceability limit state and for local structural/non-structural (e.g. façade) elements.

While a number of site and architectural constraints limit the selection and feasibility of the lateral load resisting systems, the seismic design of NZICC is guided by a set of key design principles:

- An efficient integrated bracing system to optimise functionality and space planning. This results in some degrees of vertical irregularity, the need for transfer diaphragms and the potential for concentrated ductility demand in the system.
- A well-defined and controlled seismic lateral bracing load path. Capacity-design principle is adopted to ensure the governing inelastic mechanism of the building is ductile and well-defined.
- Cost-effective lateral bracing systems, taking advantages of the ability to use most economical braced frame system in the superstructure and shear walls system in the substructure.
- A measure of resilience and robustness designed into the seismic bracing system.
- Aspirational low-damage and "replaceable" or "repairable" ductile fuses in the system. The dual elastic moment frame / wall and ductile BRB system provides a measure of re-centering capacity and damage mitigation to the superstructure as well.

2.3 Lateral Load Resisting Structural Systems

Buckling-restrained braces (BRBs) frames are selected as the primary ductile lateral load system, acting in parallel to the moment-resisting portal frames formed by the primary cruciform columns and storey-deep trusses. Several reinforced concrete core walls support the northern circulation building, which is tied back to the main structure at Level 3 and 6. The substructure lateral load resisting system comprises distributed internal reinforced concrete shear walls and the perimeter semi-contiguous bored pile retaining wall.

The superstructure systems are founded at a common seismic base at Level 3 (Hobson St), with the Level 3 floor slab acting as a transfer diaphragm to redistribute the seismic loads to the reinforced concrete walls below Level 3. The vertical discontinuity and the various levels of transfer diaphragms are illustrated in Figure 2. Figure 3 shows the schematic layout plans of the various lateral bracing elements and the seismic joints.

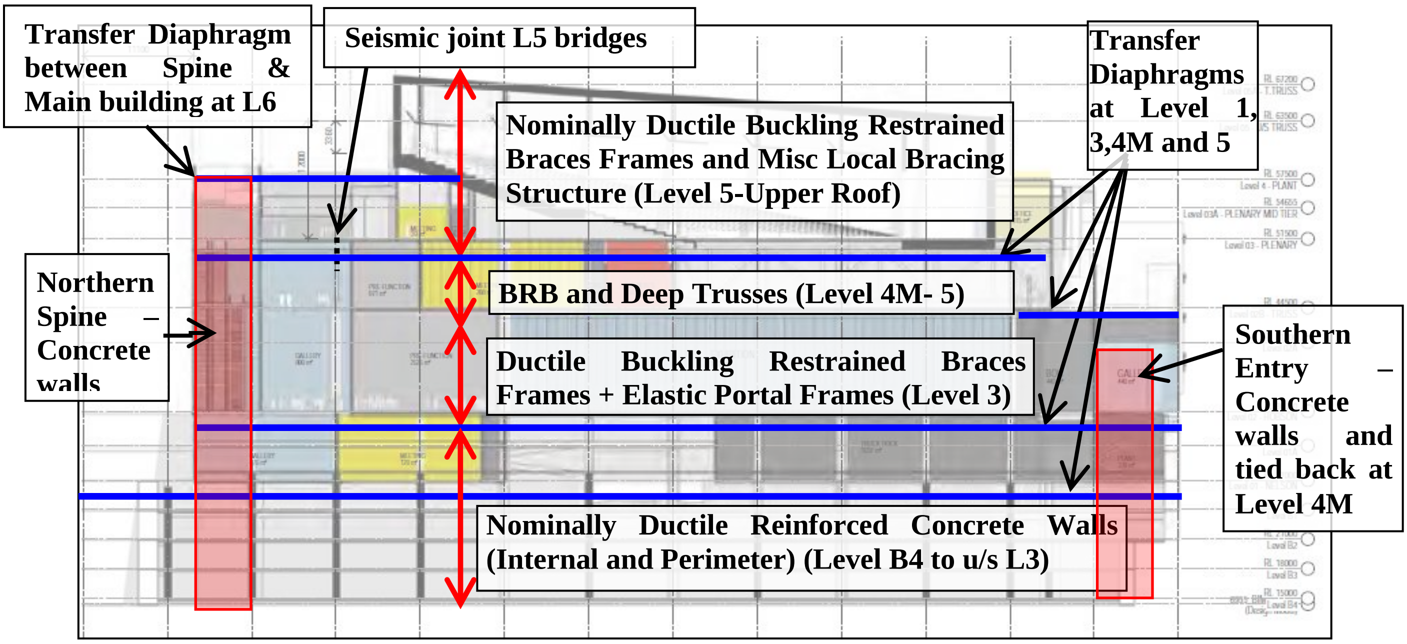


Figure 2: Schematic section of NZICC –illustrating the seismic resisting systems and movement joints

2.3.1 *Superstructure systems - Level 3 (Hobson St) to Upper Roof:*

The BRBs above Level 4M are expected to be responding in a nominally-ductile behaviour as most of the expected ductility demand are expected to be concentrated in the BRBs at Level 3 to 4M (see Figure 4). These braces are designed using conventional force-based design analysis for the effective ductility of the superstructure, with capacity design principles followed through to ensure any inelastic behaviour to be localised in BRBs' yielding core.

The cruciform columns and the connection primary and secondary trusses form moment resisting portal frames (MRF) which also contributes to the lateral load bracing system of the superstructure between Level 3 to 4M (see Figure 5). The trusses are capacity designed to ensure the potential plastic hinge zones are in the top and bottom of the columns, whilst the columns are designed to remain elastic at design level displacement.

Reinforced concrete walls along the Northern Circulation Spine provide some level of lateral bracing in both directions to the superstructure above Level 3. These walls are connected to the Main Building at Level 6 with a transfer diaphragm. These walls have been designed to be more flexible to minimise the displacement incompatibility with the BRB system. Ductile detailing are provided at potential plastic hinge zones at Level 3 to 4 though these walls are not expected to attract significant amount of seismic inertia due to the limited connection to the floor diaphragms.

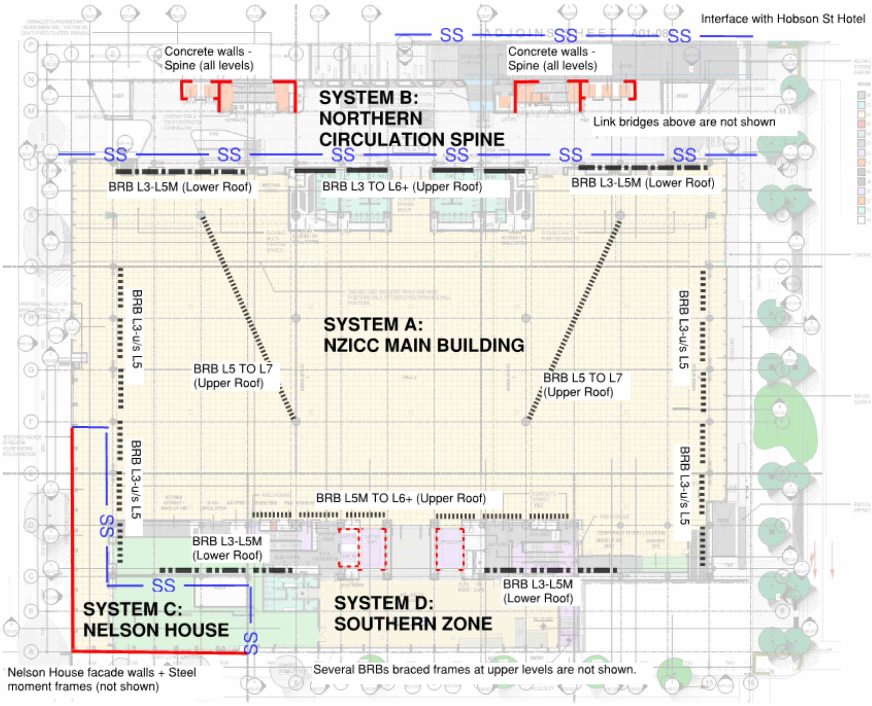


Figure 3: Schematic plan of NZICC Main Site – illustrating the four seismic resisting systems and movement joint detailing requirements

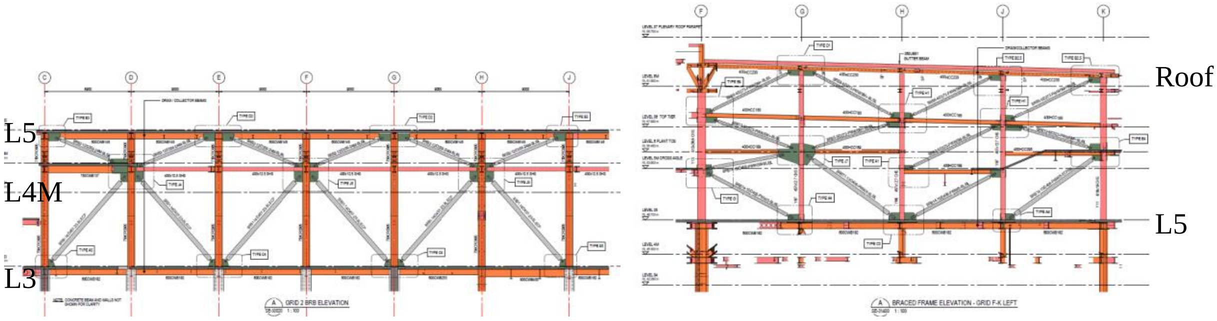


Figure 4: BRB Frames resisting seismic loading in the North-South direction: a) Grid 2/15 Level 3 to 5; b) Diagonal Grids East and West – Level 5 to Roof.

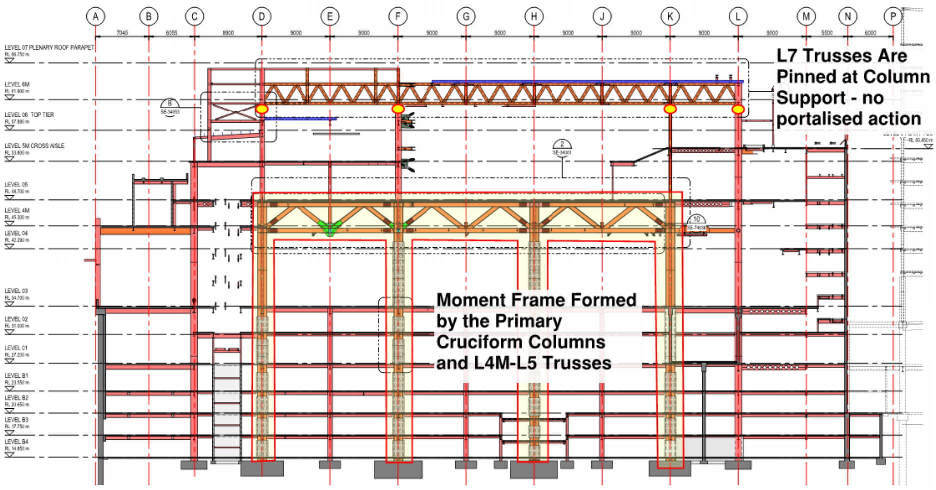


Figure 5: Moment Resisting Frame (MRF) formed by the Cruciform columns and deep trusses

2.3.2 ***Sub-structure and basement structures*** *- Level B4 (foundation level) to underside of Level 3.*

The primary lateral load system incorporates reinforced concrete shear walls to resist the lateral loads from out-of-balance retained soil, seismic inertia from basement levels and over-strength loads from the superstructure. These walls are designed for nominally ductile action from Level 1 to 3 inertia and over-strength actions from the superstructure above Level 3.

At Level 1 to Level 3, lateral loads are resisted primarily by reinforced concrete shear walls distributed on plan and around the perimeter of the building. Frictional and passive soil resistance contribute to some lateral load resistance along Hobson and Wellesley Street where applicable.

Below Level 1, lateral loads are resisted by a combination of retaining walls along the perimeter of the building and frictional and passive soil resistance. The walls are designed to resist out-of-balance passive soil pressure arising from the level differences between Nelson St and Hobson St.

2.3.3 ***Transfer, roof and normal diaphragms***

Due to the change in plan location of the vertical bracing systems at Level 1 (Nelson St), Level 3 (Hobson St), Level 5 (Plenary), and Level 6 (connection between Northern Spine and Upper Plenary), the slabs at these levels have been designed to act as transfer diaphragms to distribute the loads laterally between the vertical bracing systems. Reinforced concrete topping provides diaphragm action for typical floor slabs. Drag and collector beams are typically provided between the interface of the in-plane diaphragm and the vertical braced frame lines.

Steel diagonal bracings and struts are provided in the plane of the lightweight upper and lower roof diaphragms. These braces distribute the seismic and wind loads to the vertical bracing around the perimeter of the roof.

2.3.4 ***Local secondary structure.***

Secondary localised bracing systems are used to provide lateral bracing to floor or roof areas that are not designed to be connected to the primary floor plates.

3 SEISMIC DESIGN AND ANALYSIS METHODOLOGY

3.1 Direct-Displacement based Design (DDBD)

The derivation of global seismic design actions for the superstructure of the Main Building and the Northern Circulation Spine is based on Direct-Displacement-based Design (DDBD) approach (Priestley et al, 2007 and Sullivan, Priestley, and Calvi, 2012), as summarised in Figure 6.

However, DDBD is not currently documented in any of the Verification Method documents cited in the New Zealand Building Code such as the NZS1170.5 (New Zealand Loading Standards for Earthquake Actions). As with the BRB design, the use of the DDBD approach requires an Alternative Solution pathway to demonstrate compliance with the NZBC.

Whilst the DDBD approach provides a means to determine the overall seismic base shear and global load distribution, a 3D ETABS model is used to distribute the seismic forces to the superstructure. The DDBD-derived lateral loads are applied at the centre-of-mass of primary floor diaphragms based on appropriate inelastic deformed shape profiles.

3.2 DDBD for BRB-MRF dual system

There are two challenges in applying the general DDBD provisions from Priestley et al (2007) to a dual BRB-MRF system. Maley et al (2010) provides some guidance for these but the research was more applicable to multi-storey structure with regular inter-storey height and without irregularities as those in NZICC. The following paragraphs describe the approach adopted for NZICC.

3.2.1 *Inelastic deformed shape profiles*

The lateral load vectors in the DDBD, shown in Figure 7, are based on the appropriate inelastic deformed shape profiles:

- “Column-sway” and concentration on inelastic displacement demand at Level 3.
- First mode load distribution profile. This is also similar to the equivalent static load distribution.

The first deformed shape is generally more conservative and it has been assumed in the DDBD.

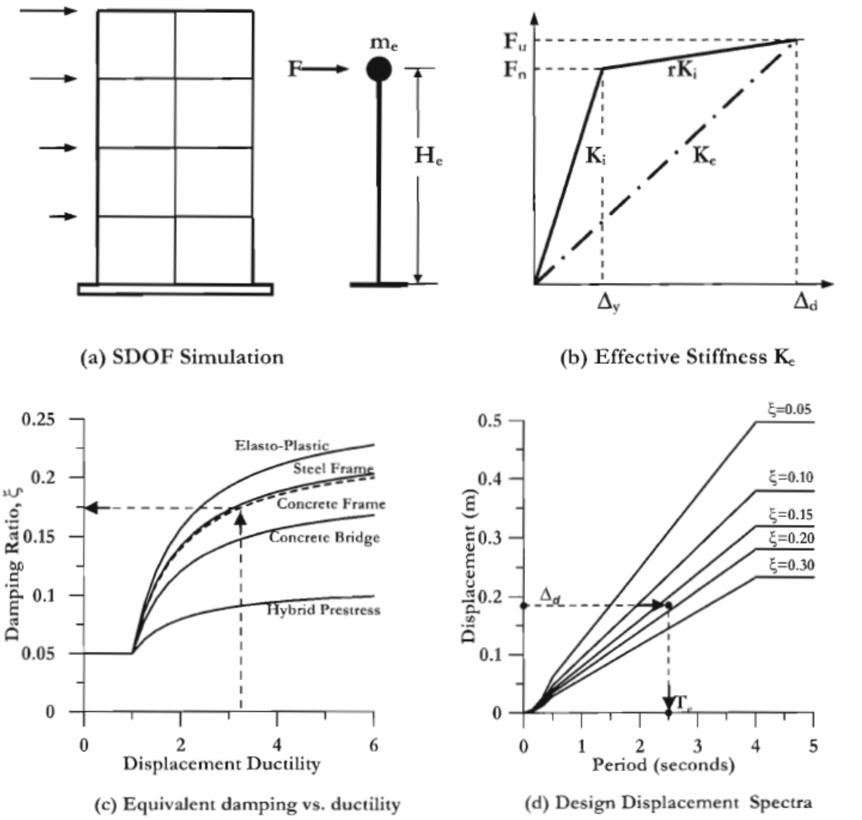


Figure 6: Fundamentals of Direct Displacement-Based Design (from Priestley et al, 2007)

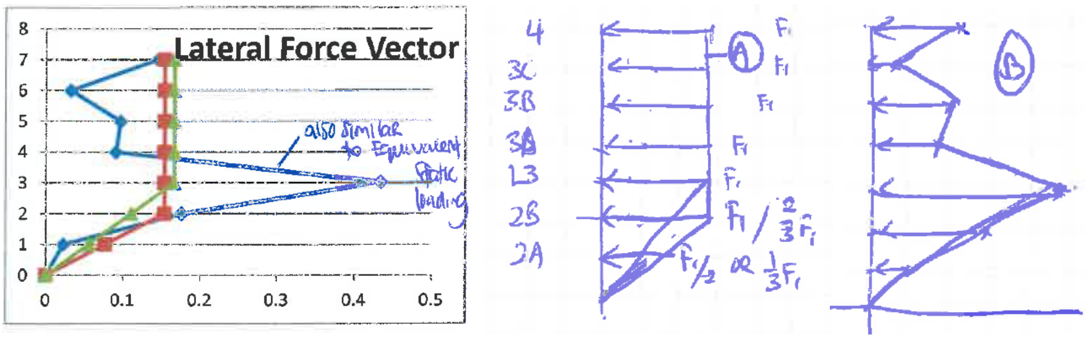


Figure 7: Assumed inelastic deformed shape profiles for DDBD: a) “Column-sway” at Level 3-4M. B) First-mode load distribution.

3.2.2 *System ductility and damping*

The following relationship of equivalent-viscous damping (ξ) and structural ductility (μ) is adopted for both BRB and MRF systems. The MRF however is designed to remain elastic at design level displacement, and therefore would only have 5% effective damping.

$$\xi_{steel} = 0.05 + 0.577*[(\mu-1)/\mu\pi]$$

The system equivalent viscous damping is therefore calculated using a base-shear weighted average:

$$\xi_{system} = \Sigma(V_{base,i}\, \Delta_i\, \xi_{subsystem,i}) / \Sigma(V_{base,i}\, \Delta_i)$$

where $V_{base,i}$ Δ_i and $\xi_{subsystem,i}$ are the design base shear, design displacement at effective height and sub-system damping respectively for the individual sub-system (MRF or BRB).

3.3 Hybrid of Force-based and Direct Displacement Based Design (DDBD)

In order to demonstrate compliance to NZS1170.5 and through discussion with the Peer Review team, a hybrid force-based and direct-displacement based design (DDBD) approach was used to derived the global seismic design actions for the superstructure (Main Building and the Northern Circulation Spine) and the global over-strength seismic design actions for the substructure (basement walls and foundation).

The general steps of this hybrid force-based and DDBD approach is as follow:

- Step 1- Undertake a DDBD design for a BRB-only system
- Step 2: Assess the MRF-only system using pushover analysis to determine its base shear as a function of imposed lateral displacement. An upper bound displacement demand from Step 1 is imposed on the portal frames to determine an upper bound elastic demand on the cruciform columns and trusses given a lateral displacement demand.
- Step 3: Undertake a DDBD design for a combined BRB – MRF dual system. Iterate for convergence of design displacement. The design base shear for the combined system and BRB-frame contribution is determined.
- Step 4: Calculate an equivalent NZS1170.5 force-based structural ductility factor μ and Sp that yields the same design base shear as per DDBD
- Step 5: Distribute the force-based seismic loading using ETABS model to determine axial demands on individual BRBs.

The design outcome of the DDBD is illustrated and summarised in Figure 8 for the dual BRB-MRF systems. The following are the various design response points:

a) **Point A:** DDBD design point for the dual BRB-MRF system, including consideration of BRB over-strength. This corresponds to a design base shear of 27.3MN at Level 3, which accounted for the strain-hardening of the BRB system and lateral load contribution from portal frames at the design displacement.

b) **Point B:** The equivalent force-based design point for the DDBD design. The design scenario above corresponds to NZS1170.5 force-based design and analysis to a selected structural ductility factor of 1.62 and Sp=0.814. The force-based base shear would be approximately 19.4MN for a structural period of 1.0s.

c) **Point C:** Force-based over-strength point for the dual BRB-MRF system, based on a global over-strength factor of 1.72. This over-strength factor is applied to the original yield force level (at Point B), and is derived based on the ratio of base shear at over-strength to yielding (33.3MN / 19.4MN).

d) **Point D:** An alternative over-strength point based on an upper-bound 1 in 2500 years return period demand. The base shear would be approximately 33.3MN for a corresponding displacement of 45mm.

This approach allows the design to take account of the nonlinearity and mixed-ductility nature of the system. The DDBD approach also provides a rationale mean to determine the level of effective structural ductility, over-strength factor, Sp-factor and thus the overall seismic base shear demand at

Level 3. This level of base shear is then converted back into an equivalent Force-based NZS1170.5 compliance seismic load case for analysis.

We have also assessed the implication of using the DDBD-derived lateral loads applied at the centre-of-mass of primary floor diaphragms based on appropriate inelastic deformed shape profiles.

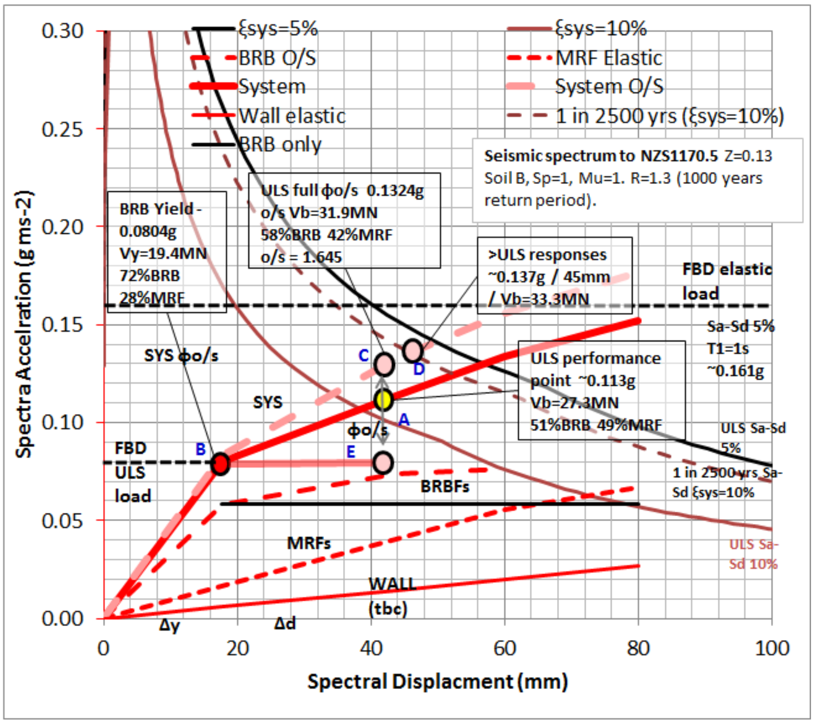


Figure 8: Direct Displacement-Based Design outcome: Global design and over-strength pushover capacity curves and seismic design actions for mixed ductility dual BRB and elastic moment frame/wall systems.

The ETABS analyses have also considered a range of loading scenarios in order to appropriately bound the design internal actions for sizing of the BRB braces above Level 3-4M, to ensure it meets the DDBD design requirements and also compliance with the intent of NZS1170.5:

- Equivalent static load distribution according to the DBD assumed deformed shape. This would be the approach for a full DBD design based on Priestley et al, 2007.
- Modal response spectrum analysis (scaled to similar base shear as derived from DDBD). This is a proxy DBD approach to account for higher modes and interactions between the different lateral load resisting systems (BRBs and walls).
- Modal response spectrum analysis (scaled to similar equivalent static base shear as derived from FBD, assuming the equivalent structural ductility of BRB system).

NZS1170.5 Section 8 Parts loading have been used to design the light-weight parts of structure such as the tertiary structure, localised diaphragm and localised bracing system.

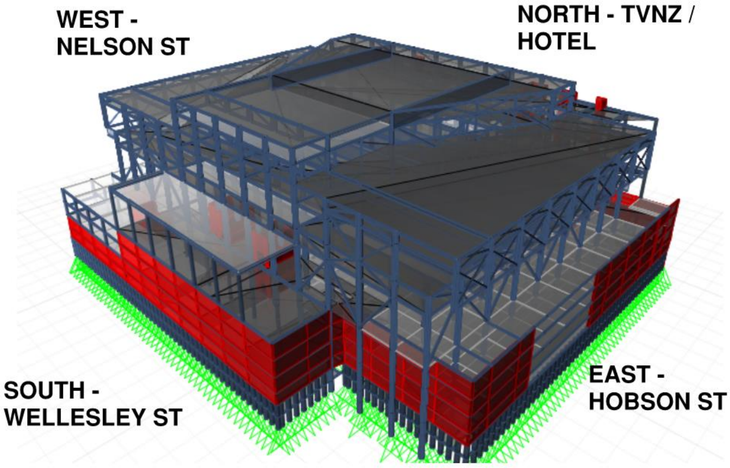


Figure 9: 3D ETABS analysis model

3.4 Rationale to use DDBD

The motivation to use displacement-based design in conjunction with force-based design was for the following reasons:

3.4.1 *Irregularity and potential for column-sway ductility concentration at Level 3 to 4M*

The superstructure is highly irregular in the vertical elevations due to the presence of the very stiff storey-deep truss zone (Level 4M to 5) and high inter-storey height of Level 3 (10.6m), resulting in a potential concentration of ductility demand from a soft-storey mechanism at Levels 3 and 4M. In such an irregular configuration, the inelastic response of the structure and the ductility demand distribution would not be well-predicted by a conventional force-based design approach or elastic analysis.

As such, the design inelastic response of the Main Building is selected to be a controlled and ductile "soft-storey" mechanism, with most inelastic displacement demand to be concentrated at the BRBs at Level 3 to 4. DDBD approach can explicitly account for this to allow appropriate structural design provisions to be made.

For the reasons above, a nonlinear method such as the DDBD approach is the most rational and suitable design approach, in the absence of undertaking a full non-linear analysis / design approach. It is noted that a non-linear analysis is generally used for design verification and assessment of existing structure, and is not generally an appropriate tool for design analysis. The level of uncertainties and effort to complete a nonlinear time-history for a complex building like NZICC would be very high.

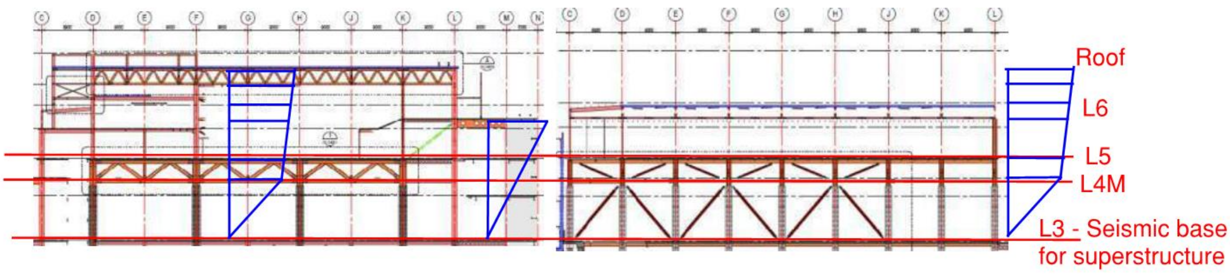


Figure 10: Schematic displacement profiles of dual DDBD and MRF/wall system

The blue lines in Figure 10 above show the assumed inelastic displacement profile with 80% of the overall displacement demand concentrated at the Level 3 to 4M. This is only an assumption for the DDBD of the critical ductile region; the overall displacement profile of the building is also assessed using elastic ETABS analysis.

In the revised DDBD design - the critical drift is calculated based on the achievable building base

shear at ULS SDOF displacement. This requires iterations as the portal frame and north walls base shear contribution is a function of the ULS displacement. The BRB contribution is a designer's choice.

The BRB contribution is optimised to ensure BRB contribution is at least 50% of the total base shear of the combined system; whilst meeting a specific target drift (~0.55%).

3.4.2 *Mixed ductility performance expected between the BRB and portal frame / cantilevered wall*

A direct displacement-based design (DDBD) approach allowed for more in-depth analysis of the proposed structures and an improved understanding of their inelastic seismic behaviour, particularly in respect of the interaction between the BRB and truss portal frames.

The portalised moment frame from the storey-deep trusses and cruciform columns at Level 3 to 4M and the reinforced concrete cantilevered walls at the Northern Circulation Spine are expected to respond elastically or with a very low ductility demand at the design level earthquake. The BRB system on the other hand is expected to yield relatively early.

A conventional elastic ETABS analysis will generally lead to over-prediction of the BRB post-yield stiffness and BRB post-yield strength contribution at the ULS and over-strength performance point. Table 1 below illustrates the difference in the strength proportioning based on initial stiffness and displacement-based approach.

The analysis method adopted achieves a load distribution closer to the non-linear strength proportion, recognising the mixed-ductility nature of the system.

Table 1. Load distribution from the ETABS model and Displacement–Based Design.

Analysis type	Direction	BRB	MRF	North Walls
Initial stiffness proportion-based analysis (ETABS)	EW	81.6	9.1	9.3
	NS	75.3	8.7	16
Displacement-based non-linear strength proportion	Yield Δy	72.3	20.4	7.3
	ULS Δuls	51.4	35.8	12.8
	O/S Δo/s	51.3	35.9	12.8

3.4.3 *Capacity design and over-strength calculation*

The combination of DDBD and FBD method provides a rational means to undertake a capacity design for the system and calculate the over-strength factor for the combined dual system. The method also allows an explicit consideration of the over-strength contribution from the elastic MRF system, which increases as the imposed lateral displacement increases.

3.4.4 *P-delta effects*

Due to the vertical irregularity and the potential to concentrate ductility demand at Level 3 to 4M, it is important to consider P-delta effects directly in the design process. DDBD allows the p-delta demand to be assessed explicitly at the design phase.

3.5 Capacity Design Principles

Appropriate capacity design principles were used in conjunction with the global analysis outputs (DDBD or force-based design) to ensure the inelastic action will be limited in local mechanisms that can sustain the ductility demand (i.e.) the BRB yielding in tension/compression, beams in moment-resisting frames, diagonal bracing in tension yielding (etc.). Capacity-design principles are also applied to ensure the sub-structure (below Level 3) respond within the nominally ductile / elastic range.

4 BUCKLING RESTRAINED BRACES (BRB) DESIGN

4.1 Design approach

Buckling Restrained Braced Frames (BRBFs) are a relatively new type of concentrically braced frame system that uses steel braces capable of inelastic yielding in both tension and compression, and a high degree of post-elastic reliability (Beazley and Built, 2013).

Currently BRBs are not explicitly included in New Zealand design standards. In generally, the provisions in NZS3404 are adopted where applicable (e.g. Concentric Brace Frame, bolted and pinned connection design). When these are not applicable, the use of current US practice for designing BRB has been employed (AISC-341; NIST, 2011; Lopez and Sabelli, 2004).

The following paragraphs only provided a brief summary of the key issues with the BRB design. Interested readers are referred to a separate publication that will be presented at the 2017 SESOC conference (Kam et al, 2017)

4.2 Over strength Actions

The over-strength factors used to compute the adjusted BRB tensile and compressive strengths are based on the maximum ductility demand on the BRBs at the design displacement.

The ULS design displacement, Δ_{bm}, is estimated from both DBD and ETABS analysis output (factored up by μ and k_{dm} factor as per NZS1170.5). The average over-strength brace strain, $\varepsilon_{BRB,\ o/s}$ is taken to be 2x Δ_{bm} / L_{ysc} where L_{ysc} is the un-bonded length of the yielding steel core of the BRB. A factor of 2 is applied to account for higher drifts than expected due to potential higher mode effects and higher member ductility demands.

Once the brace strains are calculated as above, compute strength adjustment factors, ω and $\omega\beta$, from the backbone curve derived from the test results from proprietary supplier (e.g. Saxey, 2014). More recent test of long length (>10m) BRBs have indicated the β factor for the compression over-strength is generally higher than previous literature (Saxey, 2016).

By specifying a force-based performance criteria for the proprietary BRB design, the over-strength factor from material yield strength variability was reduced significantly. The yield strength of the steel core was established by coupon tests and the area of the yield core are adjusted appropriately after the coupon testing to ensure the target ULS axial load capacity of the BRB is achieved thus minimising the overstrength loads to be taken by the rest of the non-yielding structure.

4.3 Connection Design

Bolted and pinned connections are used in the NZICC project. There are a large number of different configuration of BRB connections due to the BRB layout and complex structural configuration of the building.

The BRB connection is designed to NZS3404 requirements. The connections are capacity-designed to the adjusted BRB tension and compression capacities. For some connections, the design actions are capped to elastic loading (μ=1.0, Sp=1.0) in selected areas where the BRBs are very unlikely to yield. We have supplemented the connection design with additional checks based on the US BRB design practice (AISC Design Guide 29, 2015; AISC-341).

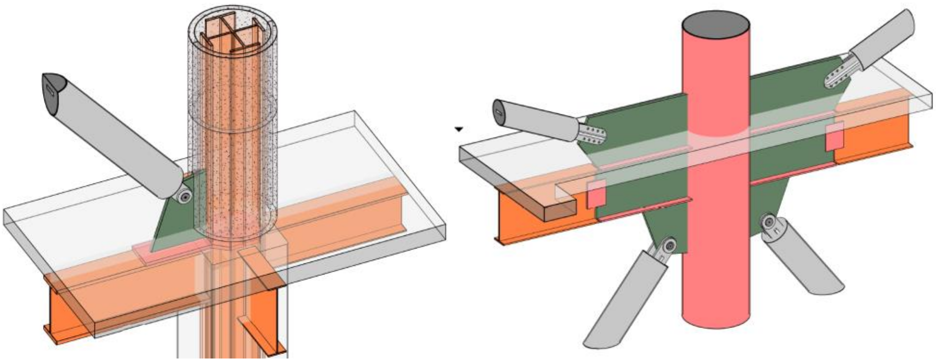


Figure 11: BRB Connections: Type A and Type J1.

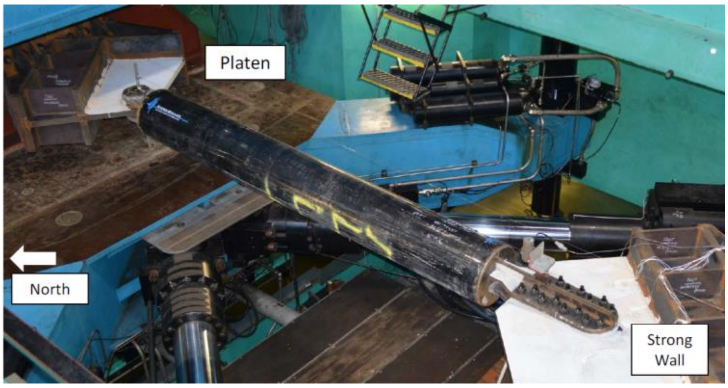
In addition to the standard gusset connection design checks, the following aspects are critical for BRB connection:

- **Load path continuity** – the design sought to minimise the connections (load transfers) along the BRB-to-gusset-to-BRB load path. This is generally achieved by having a large continuous gusset plate, slotted through and welded to CHS columns.
- **Framing action** – In-plane frame action imposes compatibility demands on the gusset and the gusset-to-beam/column weld connection e.g (Palmer et al, 2013). For NZICC, the beam moment is released with the axial/drag forces transferred via top flange continuity plate. Research has shown beam-moment release can improve the overall performance of the connection (Wigle and Fahnestock, 2010).
- **Out of plane yielding and buckling of gusset plates –** research has shown the current AISC-341 provisions can be un-conservative and premature buckling of the gusset plate can occur (Macrae and Clifton, 2015; Sitler et al, 2017). For the NZICC connections, the following was undertaken to mitigate this design aspect:
    - Whitmore section local buckling check using effective length factors of ke =2.0. both AISC-341 and NZS3404 equations are used (Westenberg et al, 2016)
    - Out-of-plane bending check of the gusset plate using a nominal 2.5% of the maximum compression axial load as per NZS3404 Clause 6.7.2.
    - Ensuring the connecting columns and beams are stiff and stiffener plates added where appropriate. Top flanges of beams are typically restrained by the floor slab at Level 3, 5, and 5M and 6.
    - Independent design check using Uniform Force Method approach by BRB supplier's engineer (CoreBrace)
    - Full-scale testing of the most critical connection (for the largest BRB at Level 3 and 4M)

4.4 Full-scale Testing

Two full-scale tests of BRB's with an eccentric pinned and bolted gusset connection (representation of the critical connection at Level 3 to 4M) were undertaken to provide further confidence to the design and to demonstrate compliance to AISC-341 provisions. The test was completed at the Seismic Response Modification Device (SRMD) Test Facility at the University of California, San Diego (Figure 12a).

The cyclic loading protocol was composed of three stages. The first stage loading was the same as that specified in the AISC Seismic Provisions. The second stage loading was developed to impose a greater deformation demand to the BRB specimens to demonstrate that the specimens could achieve a cumulative inelastic axial deformation of at least 200 times the yield deformation. The third stage loading was intended to evaluate the ultimate cumulative deformation and energy dissipation capacities. For Test Specimen 2, an out-of-plane displacement demand was imposed to one end of the connection, to create the most adverse bi-directional loading condition for the brace.



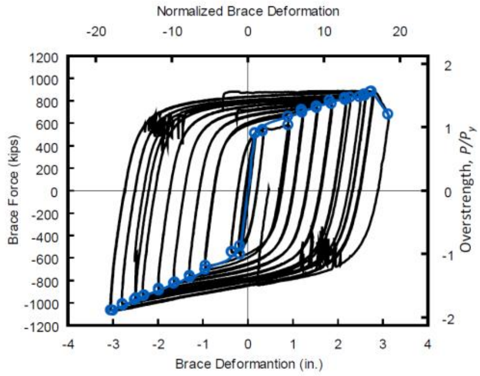


Figure 12: BRB connection testing (photograph provided by Corebrace / UCSD): a) Test setup; b) Force-displacement of Test Specimen 1 (without out of plane displacement demand).

Both BRBs test performed very well. Test Specimen 1 (without out-of-plane demand) sustained deformation demand up to 20Δy with cumulative inelastic axial deformation 5x code requirements of 200Δy (see Figure 12b). The Test Specimen 2 had similar performance, sustaining up to 25 cycles at 8Δy in the third stage loading and a cumulative inelastic axial deformation 8.5x code requirement. No damage of the connection was observed in either tests.

The presence of out-of-plane displacements, which Specimen NZ2 was subjected to, tended to reduce both the ω and β values, but the difference in the dissipated energy was not of great significance.

5 BASEMENT AND SUPERSTRUCTURE WALL DESIGN

5.1 Basement walls seismic design

An integrated sub-structure and superstructure ETABS 3D model incorporating soil-structure interaction along the perimeter basement walls was used to analyse the lateral load distribution between the basement reinforced concrete walls and substructure.

The dynamic behaviour of superstructure was modelled in the combined basement-superstructure analysis ETABS model. The integrated soil-structure model included the flexibility of the retained ground, retaining structure and floor diaphragm. This provides a better estimate of the load distribution and soil-foundation-superstructure interaction.

The design earthquake load combinations incorporate the following components

- Seismic inertia forces from BRB frame superstructure above Level 3
- Seismic inertia forces of the structure between Level 1 and Level 3
- Seismic soil pressure of the out-of-balanced retained ground

Two design scenarios were considered, in which the larger design actions are used:

- The over-strength lateral load demand from the ductile superstructure above Level 3 are superimposed with the nominally-ductile lateral load demands from both seismic inertia of substructure and retained soil. This requires a static elastic analysis to combine the various load cases. This two-stage equivalent-static approach is based on ASCE-7's recommendation for deep basement design.
- Modal response spectrum analysis of the combined sub-structure and superstructure with the base shear scaled to the equivalent static base shear as derived from FBD for nominally ductile loading (μ=1.25). Seismic mass at Level 1 and below (in the ground level) are ignored.

Limited sensitivity analysis was undertaken to confirm that the superstructure dynamic behaviour is not significant to the substructure dynamic response in earthquake.

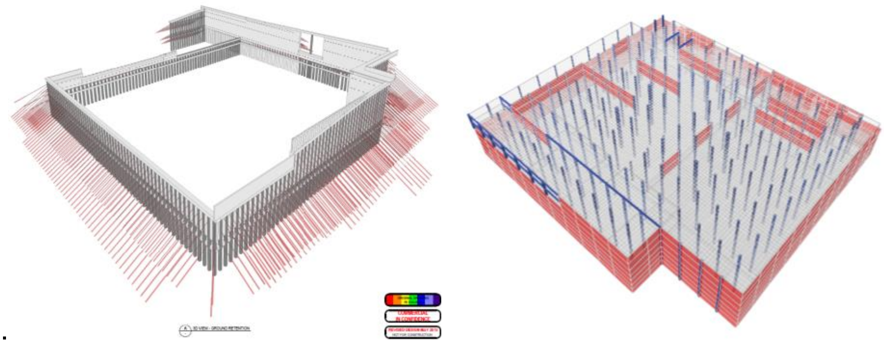


Figure 13: a) Ground retention and perimeter wall system b) foundation structure and walls between Level 1 and 3

5.2 Superstructure Reinforced concrete walls

The lateral load stiffness and contribution from the northern core walls were assessed using an imposed-displacement pushover analysis in the ETABS model. The resulting base shear at Level 3 is checked for the different level of lateral deformation imposed at Level 6 (where it is tied to the main NZICC building).

6 DIAPHRAGM DESIGN

The diaphragms and the connections between the diaphragms and lateral load resisting system are designed to ensure they remain nominally ductile / elastic under the design level earthquakes. The design actions for the diaphragms and connection were based on the least of:

- Capacity-design derived actions from the selected ductile mechanism, (e.g. over-strength BRB horizontal forces above Level 5 transferred into the Level 5 Plenary Floor slab)
- The envelope design actions from ETABS elastic analyses based on the following:
    - Over-strength force-based design load cases in which the effective ductility and global over-strength factor of the dual BRB-frame/wall system is determined from DDBD (μ=1.62, Sp=0.814 and global over strength = 1.72 at ULS design displacement).
    - Pseudo Equivalent Static Analysis (pESA) as outlined in the draft amendment of the NZS 1170.5 (published by Standards New Zealand for public comment in Sept 2014).
    - Force-based design analysis based on nominally ductile loading (μ=1.25, Sp=0.925). Results from both equivalent static analysis and modal response analysis from the 3D linear analysis were used.

The detailed design of the diaphragm reinforcing is based on strut-and-tie analysis and finite-element modelling of the diaphragm in the ETABS analysis.

The upper and lower roof steel structures have been modelled in Spacegass and are designed based on by parts loading as per Chapter 8 of NZS 1170.5, allowing ductility of 2 for steel members and 1.25 for steel connections. The lateral deflections of the structure, under both serviceability wind and earthquake, have been limited to span/300 as per AS/NZS 1170.1. These deflection limits are dependent on the type of façade finishes and other architectural finishes requirement. The stiffness of these lightweight diaphragm is modelled in the ETABS model using an equivalent concrete diaphragm with the same in-plane stiffness.

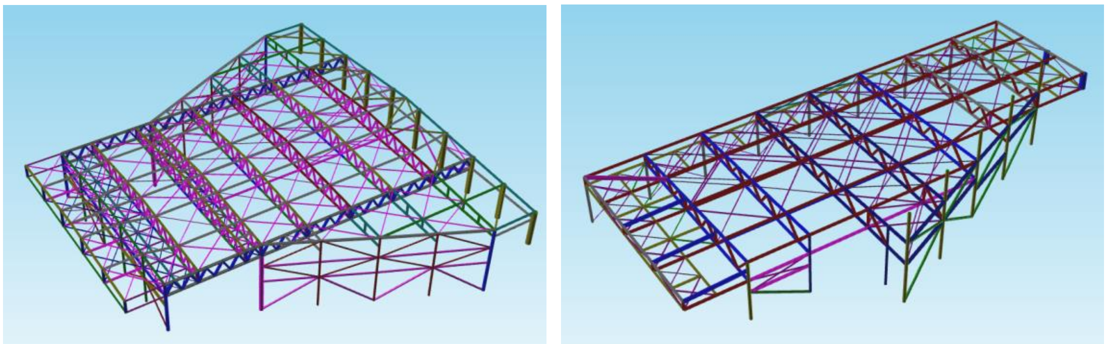


Figure 14: a) Upper Roof Level Trusses; b) Lower Roof Level 6 Trusses

7 CONCLUSIONS

The architectural and functional requirements for a world-class exhibition and conference centre for the NZICC have resulted in a structural system with a number of complex challenges for the seismic design. Some of these issues are beyond the coverage of current New Zealand standards and required the Design Team to look at the design from first principles and available research literature.

- The vertical irregularity and interaction between the buckling restrained braces (BRB) and moment resisting frames (MRF) systems necessitated the use of Direct Displacement-based Design (DDBD). In order to achieve compliance with NZS1170.5 and satisfy the Peer Reviewer, a hybrid DDBD and force-based design was adopted.
- The dual BRB-MRF superstructure provides an added benefit of highly resilient and a limited re-centering seismic system, but resulted in challenges in estimating the over-strength actions from the dual BRB-MRF system. A bounded analysis using DDBD framework and capacity design principles was used to estimate the over-strength actions from the superstructure.
- In the absence of a New Zealand standard and consensus amongst the research/practitioners, the BRB design, in particular the connection, required significant more design effort than a conventional system. The design team has consulted widely with various researchers and international guidelines, as well as completed full-scale testing of the BRB and its connections.
- The seismic design of the substructure walls as part of the deep basement requires soil-structure interaction consideration and also out-of-phase soil seismic loading also present challenges that were addressed .
- The design of the diaphragms and transfer diaphragms required specific considerations, in view of the new NZS1170.5 requirements for diaphragm design. Capacity design principles are applied where possible and appropriate.

We hope some of the identified gap areas in the New Zealand standards e.g. DDBD and BRB will be progressed and standardised in the coming years.

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