

CASE STUDY – ELEVATE APARTMENTS - A ROCKING 15 STOREY APARTMENT BUILDING

J.D. Tait¹, G. K. Sidwell² and J. F. Finnegan³

ABSTRACT

The recent earthquakes in Christchurch have highlighted the need for building structures which not only protect the life-safety of the building occupants, but offer protection and resilience against loss of function following a large earthquake.

This paper introduces one of Aurecon's recent projects which implements low damage design technologies in a 15 Storey Rocking Steel Framed Building, located in Wellington, New Zealand. The design complexities, philosophies and detailing are described, as well as the challenges encountered with implementing a rocking primary frame compared to traditional design approaches, such as the allowance for higher mode effects.

The primary bracing includes rocking concentrically braced frames (CBFs) across the building, and moment resisting frames (MRFs) with sliding hinge joints along the building. Ringfeder friction springs are provided at the base of the CBF columns to allow the building to uplift and rock during design level earthquakes. Allowing the building to uplift limits the earthquake forces in the structure and provides protection to the seismic resisting elements.

The use of low damage technologies for the design has led to efficiency in structural steel weights and has allowed the designer to achieve the stringent architectural constraints on a compact site. This project has highlighted that the implementation of low damage seismic systems has become commercially viable, even within the competitive residential building market.

Introduction

Having identified an increasing demand for residential housing in central Wellington, our client, Stratum Management Limited, approached Aurecon NZ Ltd to help develop a solution for a 15 storey residential apartment building (Elevate Apartments). Stratum Management Limited have built a number of multi-storey residential buildings in Wellington over the past decade, and are very experienced in developing and building seismically dominated structures of this size.

The procured site, near the corner of Wakefield and Taranaki Street in Wellington, is bounded on three sides by existing buildings. In order to fully utilise the site and maximize the size of the building floor plate, a 600mm distance to boundary limit was imposed on the building envelope. This limitation was a key driver in the design of member sizes and resulted in the seismic design and detailing being governed by drift and deflection rather than by strength.

The 35 metre by 25 metre L-shaped site is situated on reclaimed land and is underlain by end tipped fill placed in the early 1900's. Historical maps also indicated that a number of streams from the Te Aro area entered the harbour near the site. Borehole logging classified the subsurface conditions as moderately dense to very dense marginal marine deposits of gravels, sandy and silty gravels, gravelly silts and gravelly sands.

¹Structural Engineer, Aurecon, Level 1, 102 Custom House Quay, Wellington

²Engineering Consultant, Aurecon, Level 1, 102 Custom House Quay, Wellington

³Technical Director, Aurecon, Level 1, 102 Custom House Quay, Wellington

The geotechnical investigation concluded that no significant hazard from liquefaction was expected at the site. For seismic loading, the site subsoil category was classified as Class D (Deep or Soft Soil Site).

In the past, Aurecon's design of residential apartment buildings for Stratum Management Limited have been based around reinforced concrete coupled lift cores as the primary seismic lateral load resisting system. In order to maximise floor plate area and to accommodate architectural demands, the size of the stair and lift cores in Elevate were minimized, and during the early conceptual stages of the seismic design, it was identified that a concrete core solution would not provide sufficient strength and stiffness to support seismic demands. Traditional seismic systems were not appropriate for this complex building form, which ultimately saw the design progress towards a steel framed solution, in order to satisfy stringent architectural requirements on a very compact site.

The development and implementation of low damage seismic structural systems in recent years have proven to be superior to traditional systems in terms of building performance and recovery after a large earthquake. Stratum Management Limited embraced the concept of using low damage systems in Elevate. With their support, rocking concentrically braced frames (CBF's) with Ringfeder springs, and Moment Resisting Frames (MRF's) with Sliding Hinge Joints (SHJ's) were adopted as the seismic resisting structural systems for Elevate. The following sections of this paper describe the structural engineering philosophies, complexities and efficiencies which were encountered during the design of this complex building.

Structure Overview

The building is 15 storeys high with a height of 52m from the ground floor. The building has an L-shaped footprint with dimensions of 35 metres by 25 metres.

The structure consists of tension limited CBFs in the E-W (transverse) direction and Moment Resisting Frames with Sliding Hinge Joints in the N-S (longitudinal) direction. There structure consists of a total of 7 CBFs; three are located on the southern end of the building, three through the centre of the building and one on the northern end. There are a total 7 MRFs in the longitudinal direction spaced approximately evenly across the building

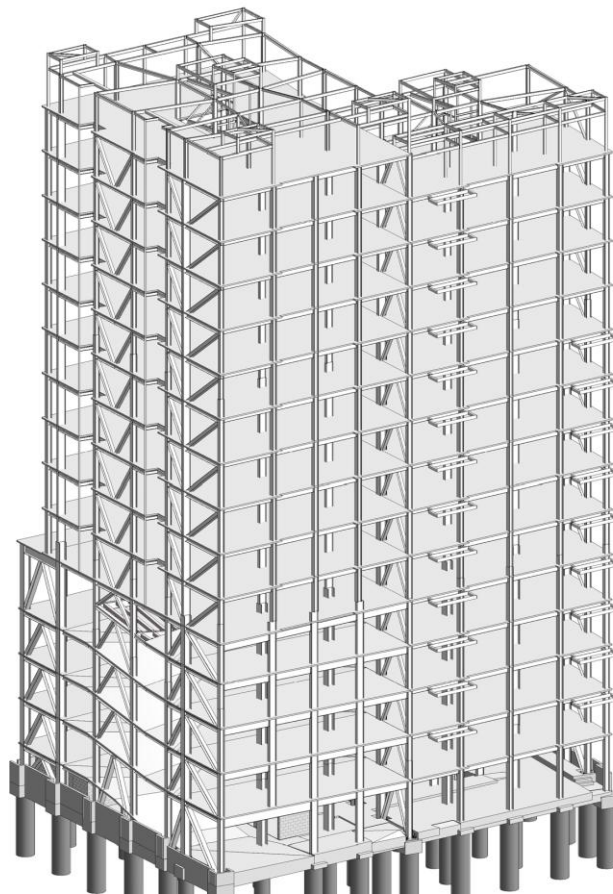


Figure 1. 3D View of Building Structure.

Transverse Bracing

The building's transverse bracing system consists of CBFs with tension limiting base connections. The base hinge consists of pre-stressed Ringfeder friction springs and a vertically orientated sliding friction connection. This system allows controlled hold down of the CBFs which limit the lateral loads to be resisted by the structure. This makes the base connection of the CBF the strength limiting element, preventing damage from occurring in the primary structural elements. This has advantages over a traditional CBF which relies on yielding of the braces to impart ductility into the structure, resulting in inelastic deformation of the structural system, requiring significant repairs post-earthquake.

The Ringfeder springs enable the connection to be preloaded so as to set the performance criteria at which "lift off" occurs. The friction sliding connection has two functions, firstly it provides additional resistance to uplift which reduces the size of the Ringfeder, and secondly it provides resistance during the downward motion of the column, reducing impact loads from the column.

Under a design level seismic event, the tension columns of the CBFs are designed to uplift. Uplift will occur once the spring pre-stress, sliding friction connection and gravity loads are overcome. As the column begins to uplift the Ringfeder spring is compressed between the baseplate and a cover plate.

This system limits seismic forces in the primary structure as well as the foundation and prevents damage occurring in primary structural elements and connections. This results in a building with immediate occupancy post-earthquake.

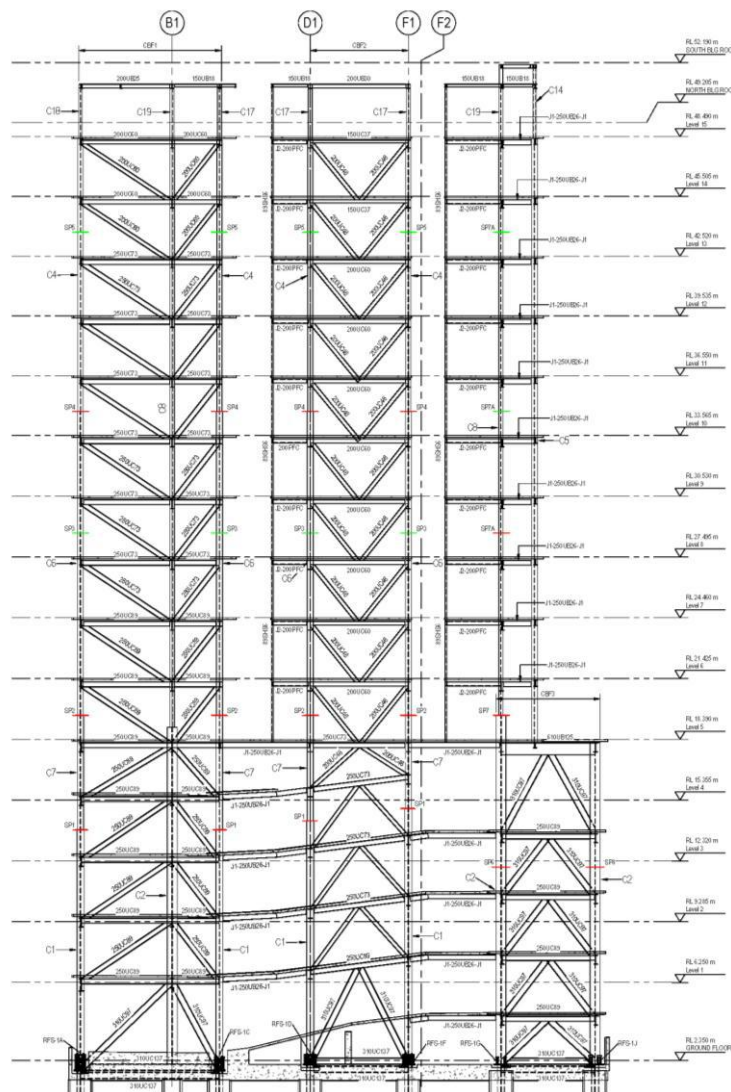


Figure 2. Elevation of Gridline 1 showing CBFs

Longitudinal Bracing

The longitudinal bracing system consists of Moment Resisting Frames utilising Sliding Hinge Joints. One of the main advantages of this form of construction is that stiffness of the beam can be decoupled from its strength. This means that larger beam sections can be chosen to limit seismic drifts without having adverse effects on column sizes. This has led to efficiencies in column and foundation sizes.

The Sliding Hinge Joint is essentially a semi-rigid beam column connection that provides a rotational pin on the top flange and a sliding detail on the bottom flange. By positioning the pin at the top flange any undesirable floor slab participation can be minimized.

The sliding hinge joint works when the moment demand from seismic actions induces beam flange forces that exceed the sliding resistance of the bottom flange and web plate bolts, at which point the joint will slide allowing rotation to occur about the top flange. Once the imposed moment reduces the sliding stops and the joint becomes rigid. The design ensures that at the design based earthquake, inelastic rotation occurs within the slotted holes equating to only minimum joint degradation and minor slab cracking. At the MCE the SHJ's will retain their integrity but will suffer joint damage.

The philosophy of the joint is to ensure performance characteristics are achieved for both design level earthquake (ULS) and Maximum Credible Event (MCE). This joint is suitable for moderate ductility, high rotation applications.

The sliding hinge joints have been designed in general accordance with HERA bulletin no. 68. More recent research of SHJ bolt and shim performance has been adopted for the design.

The column base of the MRFs must allow for large rotations required for compatibility with the deformation profile. Normally this is achieved as a flexural hinge within the steel section. For these buildings the base has been designed much as a sliding hinge joint, so that any extreme rotation takes place in a sliding action between two plates bolted together. The bolts are selected so that in conjunction with the gravity axial load the desired moment can be resisted. This connection prevents a hinge forming within the column section, therefore limiting damage to the super structure and foundations.

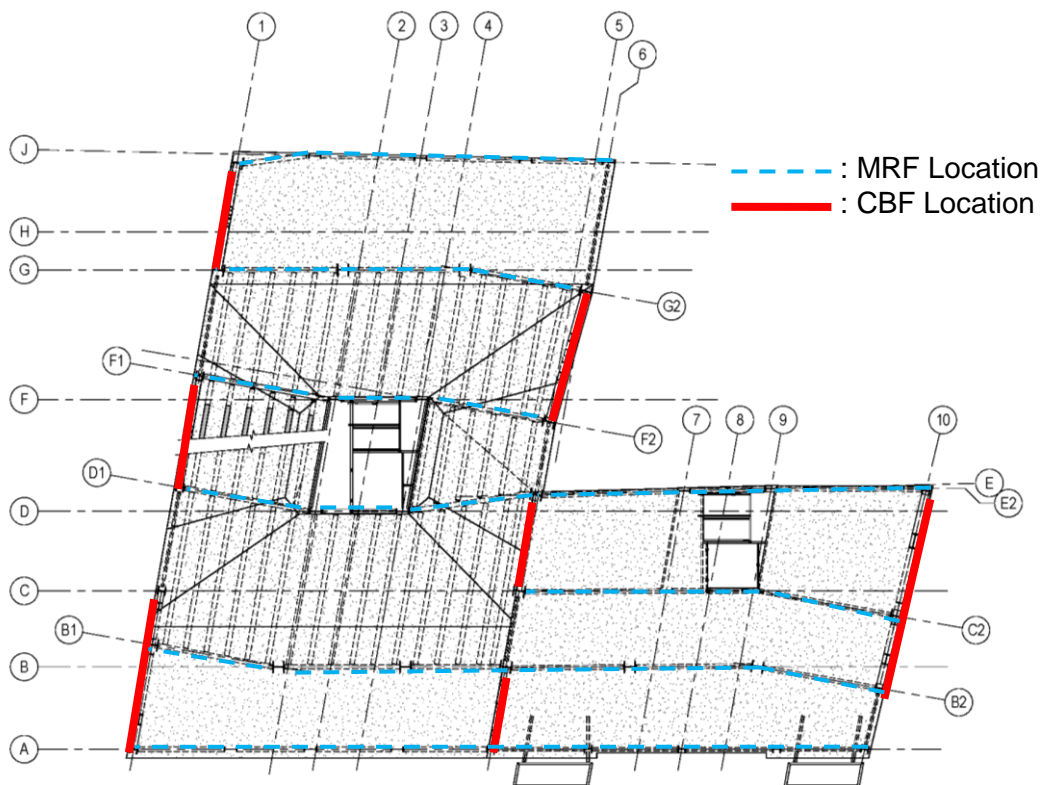


Figure 3. Typical floor plan showing location of structural systems.

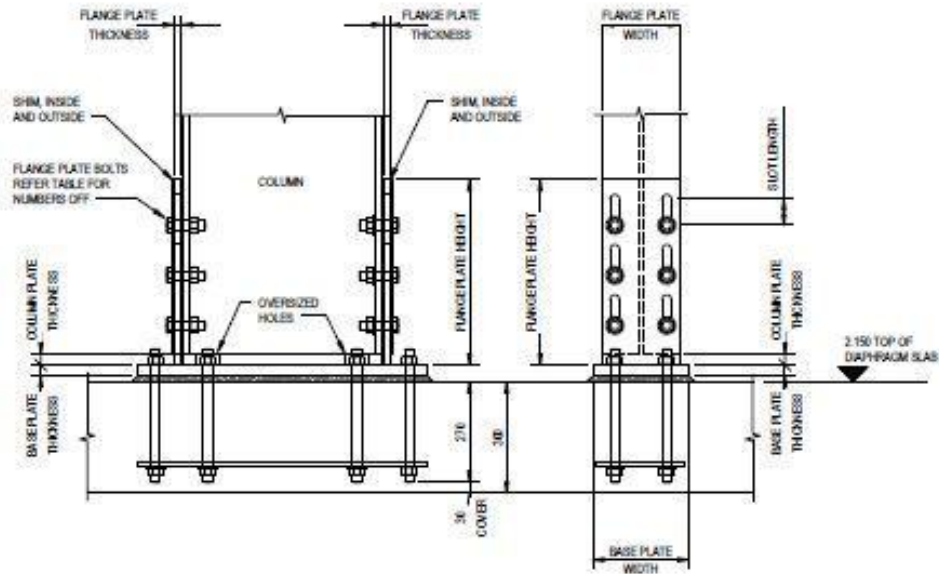


Figure 4. Typical Column Base Sliding Hinge Joint.

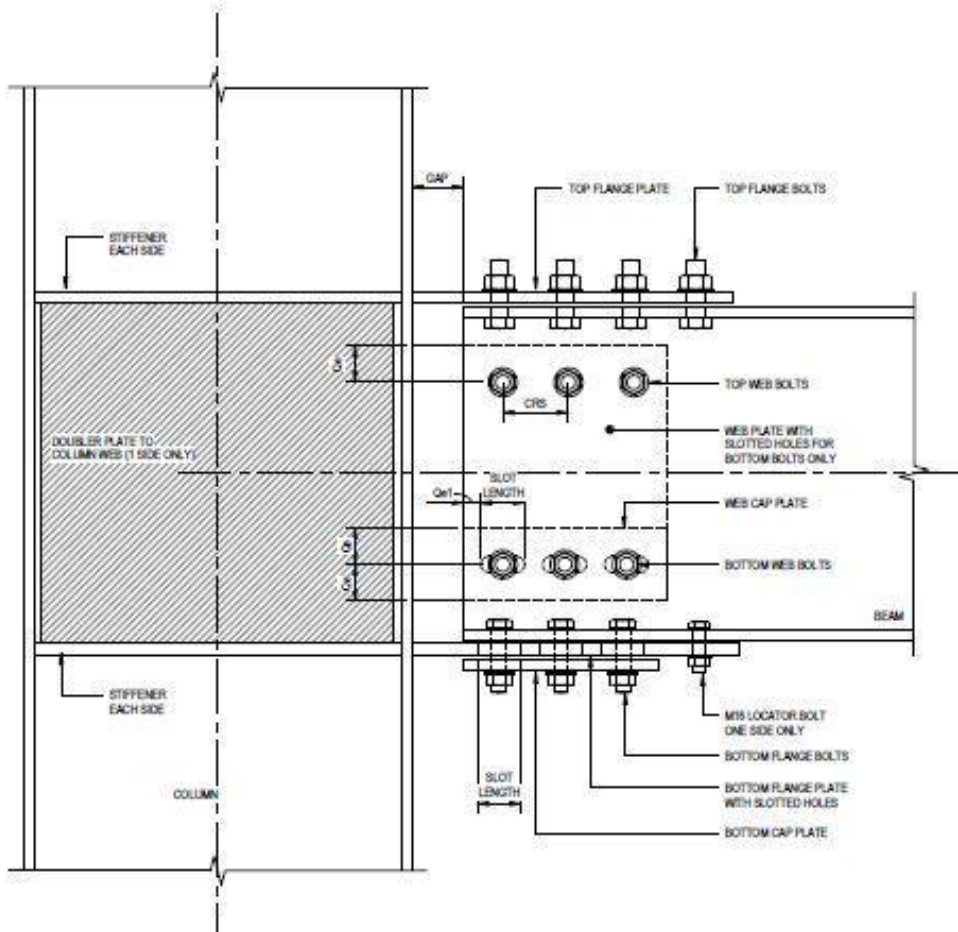


Figure 5. Typical Sliding Hinge Joint

Geometry

One of the more unusual impacts of the site, and the preplanning of the residential and car park configuration has been the complex geometry of the building. This is not a regular right angled building by any means of the imagination. Frames and elements have been carefully co-ordinated to provide maximum and ideal residential layouts. This has impacted on the geometry resulting in complex angles for beam lines and column locations. More complex still is the car park helix which twists and rotates through the first five levels. We are fortunate to be able to draw in 3D Revit and Tekla structures to model this accurately.

Gravity System

Composite steel tray floor systems span in the transverse direction between MRFs. Due to the closely spaced MRFs very few gravity only beams are required to resist gravity loads. The gravity load is then transferred to the MRF and CBF columns and into the foundation. An advantage of using the SHJ used in the MRFs is that the concrete floor can be poured prior to SHJs being tightened. This results in a beam which is effectively simply supported. Once the floor is poured the SHJs can be tightened, this simplifies design of the SHJs and reduces connection size. As the SHJ can now be designed for only seismically induced moments.

Floor System

The Floor System for the apartment floor is Conflor 80 with a 90mm topping. This will be placed continuously over the steel beams with Nelson studs applied through the decking. On the front façade, the sheeting butts to the spandrel beams with only the topping over the beam. This is to maximise the façade glazing for the external views.

The car park helix has adopted precast prestressed ribs with timber framework between. This gives a reasonable allowance for construction tolerance in this challenging 3D geometry.

Foundations

The foundation system consists of 1.2m diameter reinforced concrete bored bell piles. Piles are situated below CBF columns and MRF columns. There are a total of 40 piles with an approximate depth of 20 metres. In order to utilize all the piles, a 300mm reinforced concrete foundation diaphragm was used. This diaphragm allows distribution of lateral loads amongst the piles thus preventing shear forces from being concentrated at CBF column piles.

Ringfeder and Sliding Joint CBF Base Connection

The Ringfeder spring is a compression only ring spring. It is constructed from a stack of two different sized rings. Upon compression the outer ring and inner ring are forced outwards and inwards respectively. At all instances the rings remain in the elastic range. "Lock-up" occurs when displacement capacity of the rings is exhausted effectively resulting in a solid stack of steel rings. Figure 6 shows an example of a Ringfeder spring.

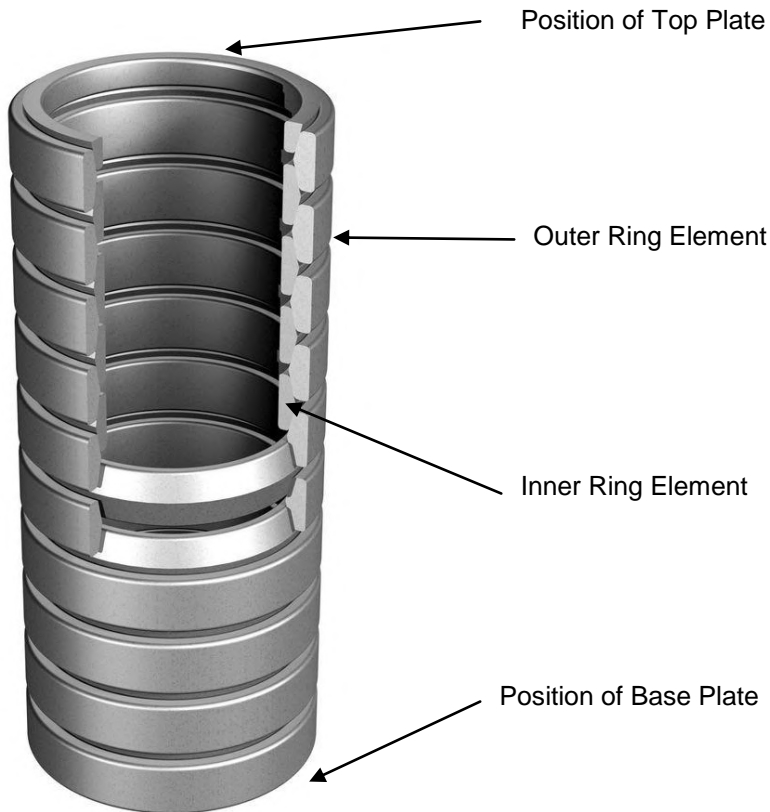


Figure 6. A cross section of a typical Ringfeder spring.

The spring is compressed between two plates using a high strength large diameter turned down bolt. The bottom plate is connected to the CBF column while the top plate is free. The operation of the joint is formed by a combination of elements acting in series. As seismic axial tension forces develop in the CBF column, gravity loads are overcome, once the gravity loads are overcome further hold down is provided by the prestress of the ring springs. Once the level of prestress is reached the sliding joint will provide the next tier of resistance until the sliding force is reached at which point the column will begin to uplift. As the column uplifts, further compression of the spring provides additional resistance to uplift. The Ringfeder spring is typically prestressed to about 50% of its ultimate capacity, so once the spring begins to lift there is approximately another 50% of reserve capacity. The purpose of the turned down bolt is to act as a fuse. The diameter of the turned down length is chosen so that the bolt yields prior to "Lock-up" of the spring, this is to prevent damage occurring to the spring and building structure. Figure 7 shows the loading curve for a typical CBF column base connection.

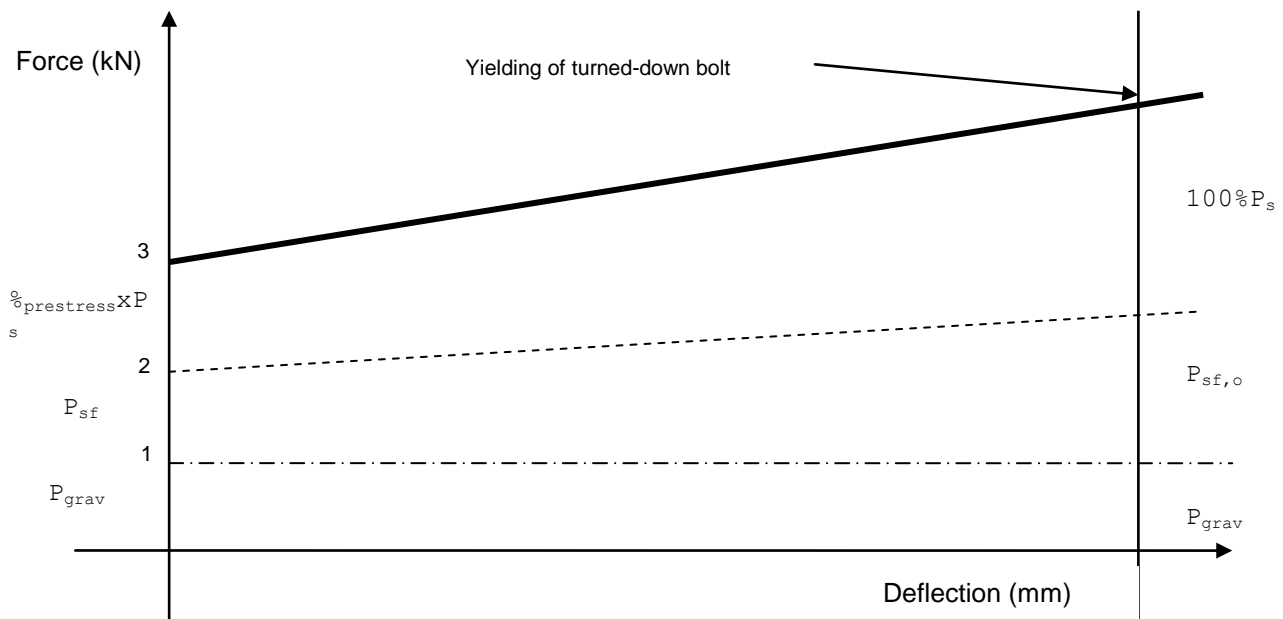


Figure 7. Combined Loading Curve for Ringfeder and Sliding Joint CBF Base Connection.

Essentially as outlined above, as tension loads increase on the column base the following occurs:

- 1) Gravity load is overcome
- 2) Friction sliding connection is overcome
- 3) Ringfeder prestress is overcome

Once the Ringfeder prestress is overcome the column begins to lift leading to a significant decrease in overall stiffness. The resultant stiffness is now that of the Ringfeder spring. This stiffness is chosen so the spring can reach the target displacement shown as the blue line before the spring capacity is reached. By adding or removing rings from the spring stack the stiffness of the spring can be tuned to the required stiffness. As indicated above by the vertical line, the turned-down hold down bolt will yield before the ultimate capacity of the spring is reached.

At the maximum Ringfeder travel, the total tension resistance is the gravity load, the max Ringfeder strength and the ultimate capacity of the sliding joint. This represents an upper bound of the connection strength and is used to determine overstrength actions for design.

Column to Pile Connection

The connection of the CBF column to the RC pile needs to be robust enough to resist the ultimate strength of the base connection. A cast in UC column member with shear studs is embedded approximately 2m into the pile. The cast in member is used to transfer tension and compression forces into the pile. The length of the cast in member needs to be long enough to develop full strut and tie action. The upper portion of the pile is tightly bound with steel spiral to prevent bursting of the column under tension loads.

The methodology adopted to provide a base shear transfer of loads to the ground floor diaphragm is a relatively simple shear key arrangement. There is an upper beam, just below floor level, connected to the base of the CBF frame. There is also a steel beam at lower level connected to the embedded pile stubs and to the concrete diaphragm. A traditional three plate shear key allows the superstructure frame to rock and lift up, while still providing a shear takeout mechanism. Refer to Figure 8.

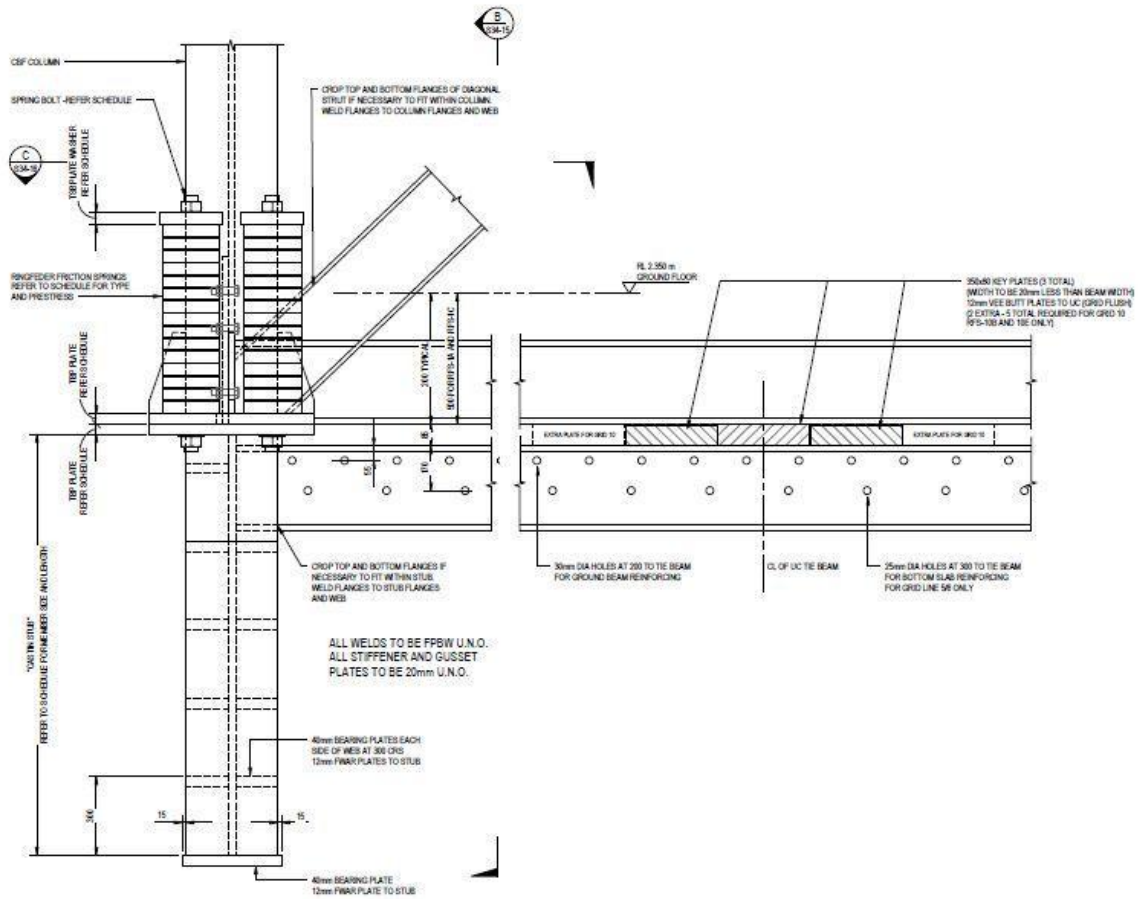


Figure 8. CBF column-base connection.

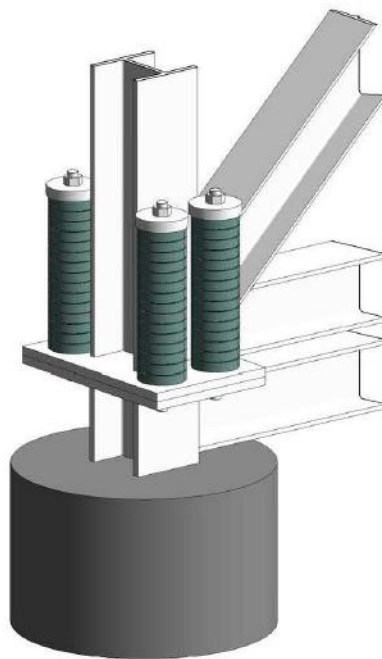


Figure 9. 3D view of CBF column base connection.

Structural Analysis

The structure comprises two dissimilar lateral load resisting systems, Steel moment frames, with sliding hinge joints, in one direction and concentrically braced steel frames, with Ringfeder springs to provide controlled tension hold down, in the other.

For both systems the underlying design philosophy is that in the event of a major earthquake the damage is to be confined to elements that are easy to repair.

The maximum acceptable level of ductility is that which will not result in any damage in the serviceability limit state (SLS) earthquake of wind event. The level of structural ductility chosen for both the longitudinal and transverse bracing systems is $\mu = 4$, this coincides with a SLS earthquake.

Modeling

The structure was analysed using the 3D analysis software ETABS. A Modal Response Spectrum Method was used in accordance with NZS1170.5. A modal response spectrum method was required due to the size and complexity of the building. As a result of the stringent drift constraints on the building, p-delta effects were not significant.

Due to the shape of the building in plan, and the layout of the seismic resisting systems, the building response was highly torsional. This torsion was further compounded by the architectural constraints of the MRF member sizes on the East and West face to suit apartment frontages. Because of the restriction of beam and column sizes on the East and West elevation, these frames were less stiff than the internal MRF frames and could not contribute significantly to torsional resistance.

As a result of the torsional component of the building response the CBF frames located furthest from the centre of the building, i.e. the southern and northern frames had significantly greater demands than the internal CBF frames.

The lack of symmetry in the lateral bracing system would mean that in a traditionally designed system each CBF and MRF frame would need to be designed for varying actions, i.e. frames located furthest from the centre of rigidity subject to greater forces. This was overcome by the use of the tension limiting base connection and sliding joint. By altering the Ringfeder spring capacity, pre-stress and the sliding joint strength the overturning capacity of a CBF could be tuned, allowing redistribution of load along bracing lines to adjacent frames. This produced a simpler and more efficient structure in terms of frame member sizes than could be achieved with traditional design methods, resulting in a more rational design.

Drifts

Due to the limitations on building drifts, approximately 600mm at the worst location, Aurecon engineers needed to develop ways of limiting deflections without adding too much cost to the structure. This was achieved in a number of ways. Firstly, a reduced k_{dm} factor was used for scaling inter storey drifts, this reduction in K_{dm} factor was based on research by Uma et al². Secondly, the use of SHJs meant that larger beam sections could be used to control drifts without having detrimental effects on column and foundation sizes. Finally by designing the MRF beams to act compositely with floor slab the beam stiffness could be increased.

Inter storey drifts were determined by extracting drift data from the modal response spectrum analysis and scaling by the appropriate scale factors as described in NZS1170.5. In the MRF direction (N-S) a K_{dm} factor was used in accordance with NZS1170.5. However, a reduced K_{dm} factor was used for the CBF direction (E-W). This is consistent with the approach to the design of the diagonal braces and the findings of Uma et al in their study of the assessment of K_{dm} for different structural forms.

The use of SHJs was an effective tool to increase the stiffness of the building without having a detrimental effect on the column size. Control of building drifts is influenced strongly by MRF beam section size. The SHJ has the advantage that it can decouple stiffness and strength, i.e. the strength of the SHJ is not controlled by

² S.R. Uma, A.B. King and T. Holden, 2012, Inter-storey Drift Limits for Buildings at Ultimate Limit States, 2012 NZSEE Conference

the section properties for strength or stiffness. If a traditional MRF beam column joint were to be used, the column would need to be designed for the full over strength actions of the beam section, leading to larger column and foundation sizes. The strength of the SHJ can be tuned so that it is just strong enough to resist the design forces, leading to savings in column size.

For this apartment block the floor to floor height is such that the possible beam depth is restricted to 460 mm. The stiffness of the beams was evaluated by considering the partial composite stiffness for all beams remote from the perimeter of the structure. Sufficient shear studs were supplied to achieve 25% composite action and the stiffening effect evaluated in accordance with NZS3404:C13.1.2.6.

Maximum Credible Event

The design philosophy of the structure is to ensure performance characteristics are achieved at both the Ultimate Limit State (ULS) and the Maximum Credible Event (MCE). At MCE the performance requirement is that there will be no significant damage to the structure. This is achieved by allowing additional rotation of the CBF frame by yielding of the hold down bolts.

The hold down bolts are designed to yield prior to “lockup” of the Ringfeder springs. This yielding will occur just after the ULS deflections have been reached. The bolt will continue to yield until the MCE deflections are reached. The yielding of the bolt is ensured by necking the bolt, the length of the neck is chosen so that sufficient yield displacement can be accommodated to reach the MCE drift requirements. See Figure 10 and Figure 11.

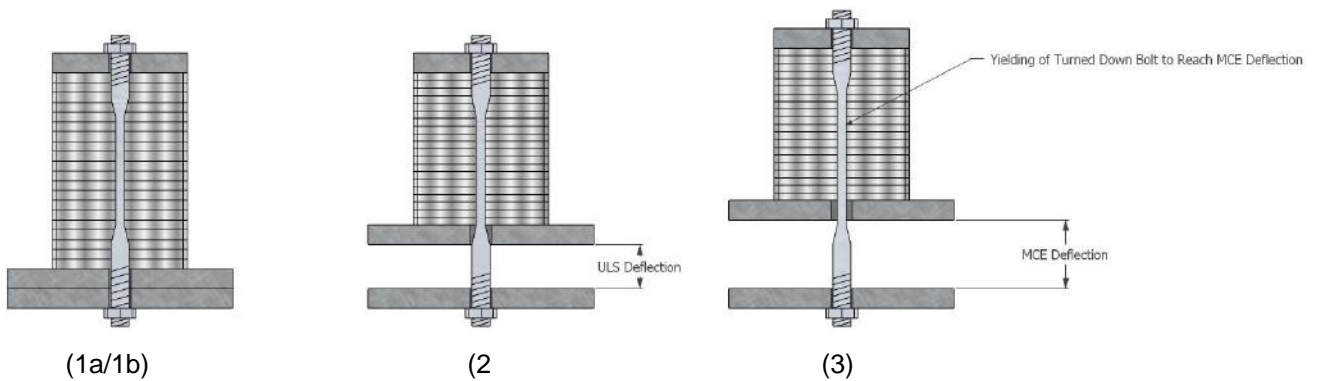


Figure 10. Ringfeder spring behavior.

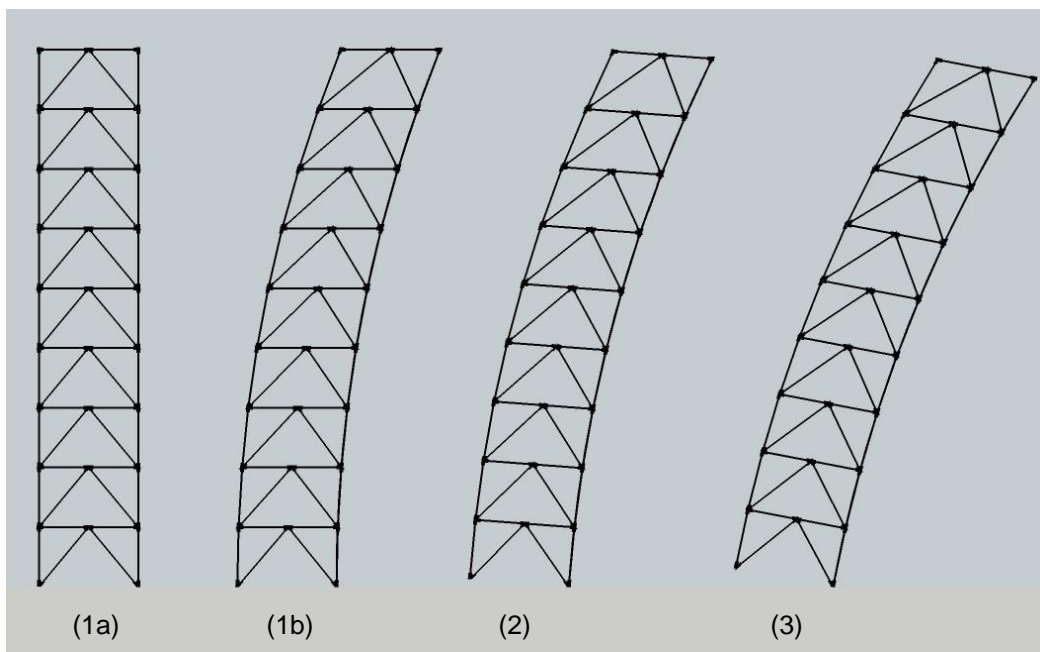


Figure 11. Loading stages of a typical CBF.

As shown above, the performance of the CBF is controlled by the base connection.

- 1a) Starting position of the frame. At this point there is full gravity loads on the CBF base connection.
- 1b) SLS Earthquake. This represents the point at which uplift will begin. At this point the gravity load, sliding joint capacity and prestress has been overcome. Any load greater than this will cause the frame to rock
- 2) ULS Earthquake. The base connection will rock to achieve ULS deflections. Additional force from the spring will mean that there will be some further elastic deformation of the structure. i.e. rotation is not pure rigid body rotation.
- 4) MCE Event. Once the frame has reached the ULS deflections any additional load will lead to yielding of the turned down bolt, this will cause further rotation until the MCE deflection is reached.

Higher Modes

Due to the long periods of the primary translational and torsional modes, the effects of higher modes on the design of the lateral resisting system were expected to be significant. To account for higher modes the modal response spectrum analysis was run twice, firstly with the full response spectra and secondly with a curtailed response spectra. In the second analysis, the response of the periods for the first modes for translation and torsion were set to zero, this enabled the higher mode forces in the structure to be isolated. The forces in the lateral load resisting systems for both analyses were then combined using square root of the sum of the squares (SRSS).

Buildability

The Elevate Apartment Building is a fully steel framed building. It relies on no reinforced concrete vertical element for gravity or seismic resistance. While the building has just been lodged for consent, there has been considerable interaction with the Main Contractor – Stratum Management and the Steel Work Fabricator MJH. MJH is also creating the fabrication drawings for the structural steel.

One of the major drivers for the construction of the steel frame is the location and capacity of the crane. The crane adopted is a Raimondi LR60 which can lift 2.95t at a range of 25m. This has defined the weight and size of the steel elements to be prefabricated and erected internally and at the rear of the development. A mobile crane will be utilised for some of the very heavy elements at the front of the site.

MRF Frames

The MRF frames are typically of two types, one with a 460 UB beam section and 530 UB column section, and one with a 360 UB beam section and 310 UC columns where architectural head room is a prerequisite. Some of the columns at lower level are significant and 500 WC sections are required.

The beam column joints are traditional Sliding Hinge Joints. These are erected in a column and beam separate stick arrangement. Refer Figure 5. A typical lift will be three storeys, however it is proposed to erect 5 storey columns on the front, on the east face.

CBF Frames

The frames generally consist of UC and WC sections for columns and beams, and 200 UC, 250 UC and 310 UC for the braces. The collector beam and braces will be prefabricated in one storey modules and lifted in pre-made to attach to the columns in the braced frames.

Conclusions

This paper has introduced and described the implementation of low damage seismic technology on a 15-storey steel apartment building in Wellington. Tension limited Centrically Braced Frames and Steel Moment Frames with Sliding Hinge Joints were innovative applications of low damage design adopted in the design. The use of low damage steel frame systems meant that stringent site constraints and architectural geometry could be accommodated.

References

- S.M Gledhill, G.K Sidwell and D.K Bell, 2008. The Damage Avoidance Design of tall steel frame buildings – Farlie Terrace Student Accommodation Project, Victoria University, *2008 NZSEE Conference*
- S.R. Uma, A.B. King and T. Holden, 2012. Inter-storey Drift Limits for Buildings at Ultimate Limit States, *2012 NZSEE Conference*
- G.C Clifton, G.A MacRae, H. Mackiven, S Pampanin and J Butterworth, 2007. Sliding Hinge Joints and Subassemblies for Steel Moment Frames, *2007 NZSEE Conference*
- HERA Report R4-134. Clifton, G.C (June 2005). Semi Rigid Joints for Moment Resisting Steel Framed Seismic-Resisting Systems.
- H. Zhou, 2012. Revision of the Bolt Model in the Asymmetric Friction Connection, *Design Report and Research Notes, University of Auckland*.
- Ringfeder – Friction Springs Product Catalogue.