Seismic retrofit of a two-storey reinforced concrete building using moment-resisting frames (MRFs) incorporating the optimised sliding hinge joint (OSHJ)

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ABSTRACT

A considerable portion of existing buildings do not meet the requirements of the current seismic standards and from an earthquake assessment are categorized as earthquake-prone buildings. These require either demolition and reconstruction or seismic retrofit. Seismic retrofitting is the modification of existing structures to make them more resistant to seismic activities to satisfy fully or partially the requirements of the new building standard (NBS). The decision whether to demolish and reconstruct or to implement a seismic retrofit involves many factors and is very project specific. One of the possible seismic retrofit solutions is adding structural steel moment-resisting or braced frame(s) to the existing structure, however these must be stiff under normal conditions and flexible in a severe earthquake. Incorporating seismic friction connections in such steel frames may be a superior and more resilient solution as they may be designed to limit forces and dissipate seismic-induced energy imposed on the structure. One such system is the Optimised Sliding Hinge Joint (OSHJ).

The OSHJ is a resilient and cost-effective seismic resisting system developed primarily for Moment Resisting Steel Framed (MRSF) buildings. The OSHJ incorporates Asymmetric Friction Connections (AFCs) with Belleville Springs. The MRSF may be designed as elastically stiffer than

the existing structure for normal conditions, with the OSHJ designed and constructed to rotate through controlled friction sliding at a pre-specified level of moment demand, dissipating large amounts of severe earthquake-induced energy and limiting the internal actions on the retrofitted building.

This paper describes using the OSHJ frame(s) as a seismic retrofit solution in a two-storey reinforced concrete building in Hastings, New Zealand. The practical considerations and project requirements, as well as the concept design, and lessons learned are covered.

1 INTRODUCTION

1.1 Building Description

The original building was constructed in 1969 for New Zealand Breweries Limited as a club in Hastings. The physical location of the building is 131 Heretaunga Street East, Hastings, at the corner of Karamu Road and Heretaunga Street East, as shown in Figure 1 below.

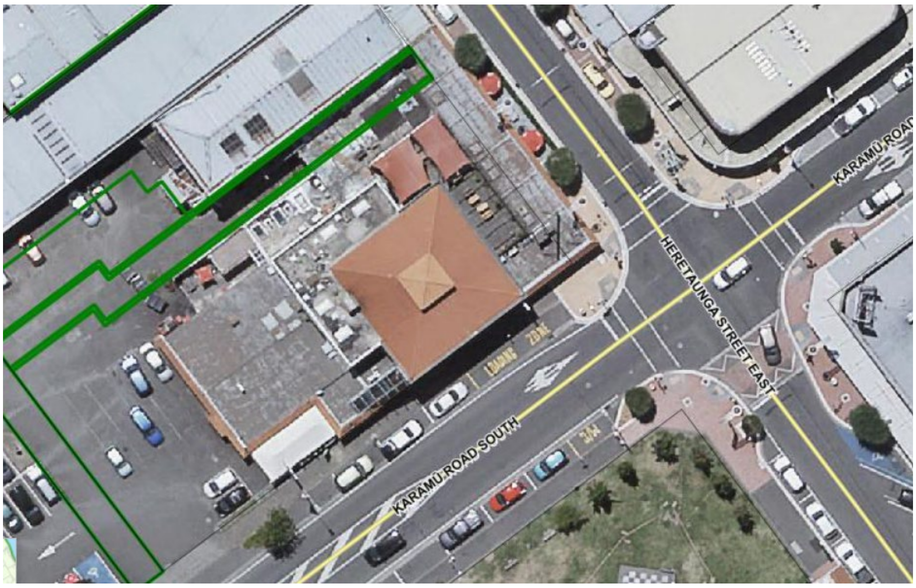

Figure 01: Location Plan of the Building

The building underwent some internal alterations in 1996. Prior to the current alterations, it was known as the Breakers, Hastings.

The building is founded on a grid of ground beams and some reinforced concrete piles at most of the grid intersections. The piles are octagonal shaped and around 8m deep.

Typical structural plans of the building are shown in Figures 2 and 3 below.

First floor slab is 200mm thick and has two layers of reinforcement. Part roof of the upper floor is concrete and it is L shaped, making this part of the building irregular. The other part of the roof structure consists of steel rafters and lightweight roof cladding.

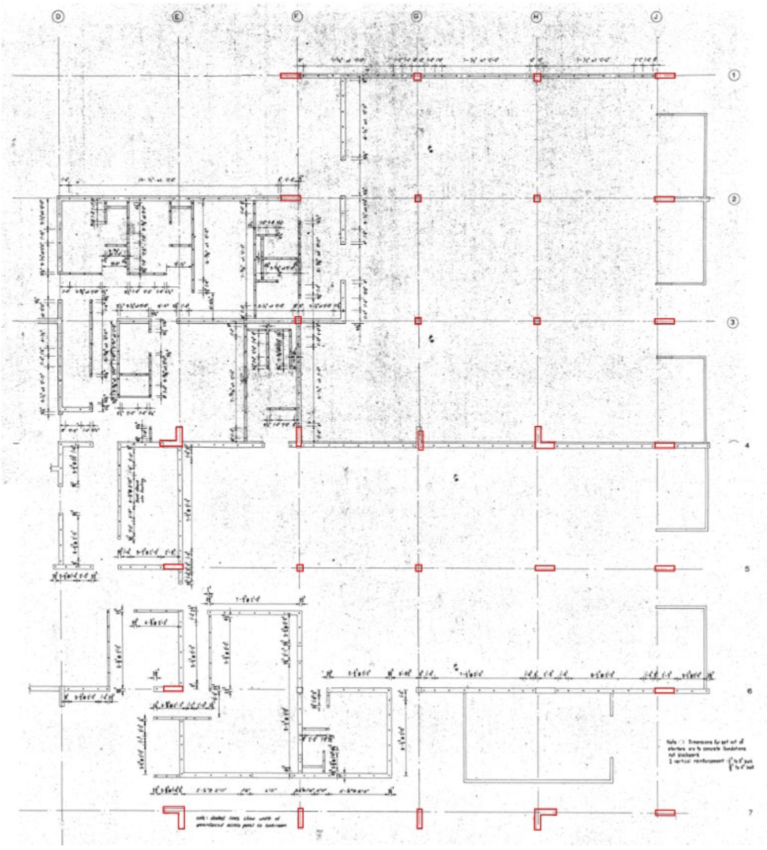

Figure 02: Ground Floor Structural Plan of the Building

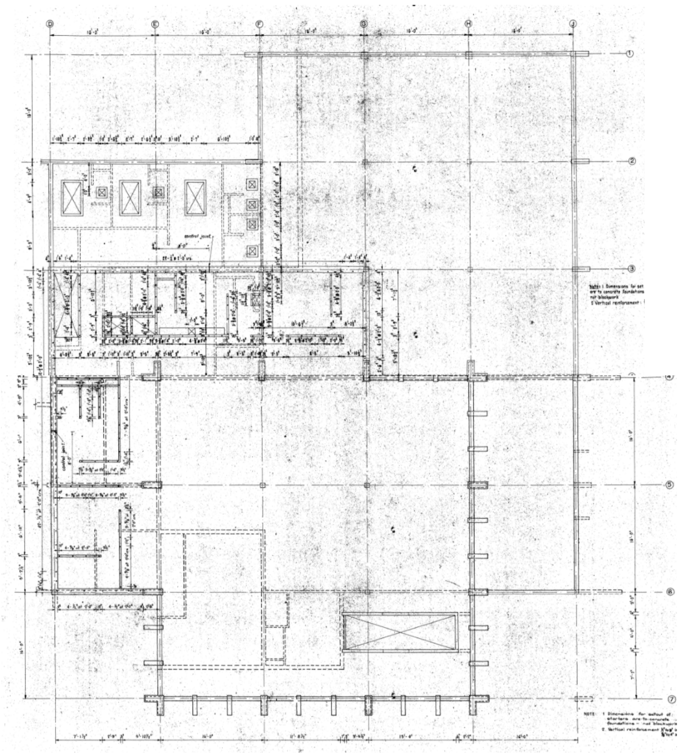

Figure 03: Upper Floor Structural Plan of the Building

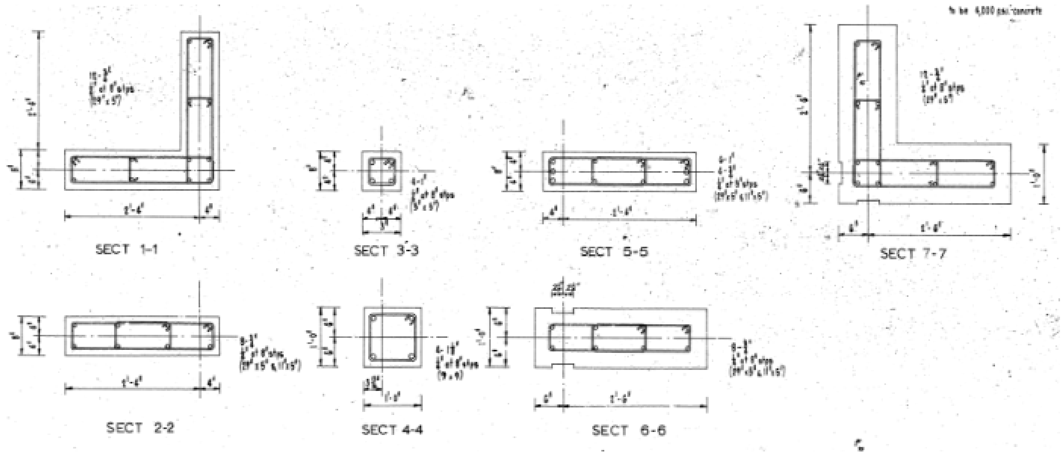

Figure 04: Typical Column Cross Sections and Reinforcement Details

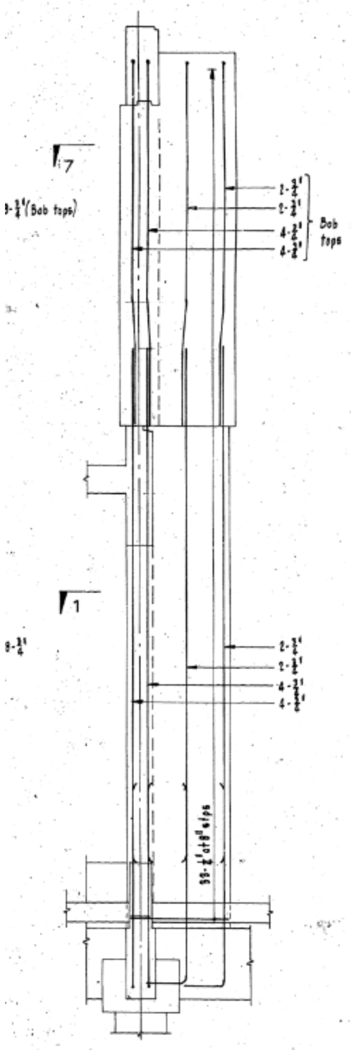

Figure 05: Typical Column Reinforcement Details in Elevation

Based on the available reinforcement details and the structural configurations, we could not establish any ductile mechanisms for the building to resist earthquakes above the elastic limit for the building.

1.2 Work Completed So Far

An Initial Seismic Assessment (ISA) was completed for the building in 2021 and the building recorded a score of 45% NBS(IL2). The aim was to seismically strengthen the building to 70% NBS(IL2). The new owners decided to redevelop the building with various changes to the internal layout. During the internal cladding removals, most of the structural elements were exposed and it was apparent that most of the walls were unreinforced block masonry partition walls.

2 STRUCTURAL STRENGTHENING TO THE BUILDING

During the detailed structural analysis of the building for structural strengthening, the building was modelled in ETABS. Foundation piles were modelled as springs. The existing foundation pile capacities were a big limitation, as any structural strengthening scheme had to be worked around the existing pile capacities

Based on the available details, elastic seismic response had to be considered. Preliminary analysis resulted in 610UB125 portal frames. However, some foundation pile strengthening was required. After several iterations, pile strengthening could be minimised for ductile seismic demands. Portal frames, in this case, did not behave in a ductile manner and so other options were investigated for using ductile demands. Due to the small lateral drift capacity of the existing building, some proprietary systems could not be used here. Optimised sliding hinge joints (OSHJ) for the moment resisting portal frames became an ideal application for our case. The bolts can be tensioned to achieve the desired moment capacities (provided that other conditions are met). With the use of OSHJ, the steel frame size was reduced to 460UB82, which provided the required elastic stiffness up to the joints starting to rotate, then allowing further controlled deformation within the joints with very little increase in strength, thus limiting the forces transferred into the existing structure and the foundations. The aim with using this system is raise the damage threshold of the retrofitted building to a level of earthquake higher than the Serviceability Limit State and closer to the Ultimate Limit State, then to allow controlled damage to the existing structure, with the lateral load capacity and the vertical load capacity retained through the new MRSFs with OSHJ connections.

3 SLIDING HINGE JOINT (SHJ)

The sliding hinge joint (SHJ) (Clifton 2005) with asymmetric friction connections (AFCs) is a low damage beam-column connection developed for seismic MRSFs. Figure 6 shows typical layout of the SHJ. AFC is a type of slotted bolted connection (SBC) acting as fuse for the SHJ. AFCs are the friction energy dissipating components and are designed to slide at a pre-determined sliding force, under severe earthquake, to dissipate input energy by friction and to limit the maximum beam end bending moment to a pre-determined level, hence effectively preventing any over-loading induced damage to the SHJ beam and column (note the slotted holes in the bottom flange plate as well as web bottom bolts level of web plate allowing such sliding to occur). Being subjected to an event larger than the ULS event, the SHJ may undergo large beam-column relative rotation through sliding in two AFCs which are located at the beam web bottom bolt and bottom flange levels. No lateral movement occurs between the beam and column at top flange level through an axially stiff, rotationally flexible detail. For the events smaller than the ULS, the SHJ is intended to remain rigid. At the end of an earthquake which is larger than the ULS earthquake, the SHJ is ideally intended to come back to its as-built condition. This necessitates the SHJ to dynamically self-centre and to retain its as built AFCs' sliding strength through retaining the AFCs' clamping force. This ideal performance has been achieved after developing the Optimised Sliding Hinge Joint (OSHJ) as is explained in section 4 of this paper.

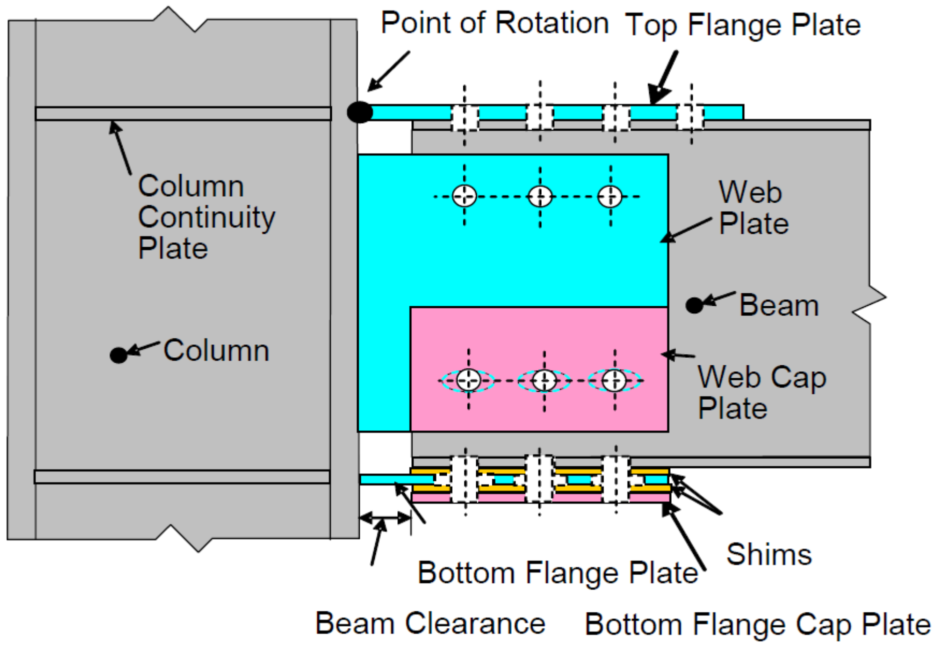

Figure 6: Sliding Hinge Joint Side Elevation (Not to Scale) (MacRae, Clifton et al. 2010)

The AFC consists of five plies, including the beam bottom flange or web, bottom flange plate (cleat) or web plate, cap plate, and two shims at both sides of the bottom flange or web plates, all clamped by the pre tensioned high strength friction grip (HSFG) structural bolts. The AFC has two main sliding surfaces at both sides of the cleat. When the applied force overcomes the frictional resistance of the first sliding surface, i.e. the cleat and the upper shim interface, this surface starts to slide (i.e. the cleat moves relative to the beam bottom flange), while the cap plate remains fixed relative to the cleat and moves relative to the beam bottom flange. After a short time or distance of movement along the first sliding surface, the cap plate starts to become fixed in position relative to the beam bottom flange. From this point onwards for that direction of displacement, the cleat slides between two sliding surfaces (i.e. upper and lower shims), which almost doubles the sliding shear resistance (called the stable sliding shear resistance) developed by the AFC. At this stage, the bolts are deformed into the double curvature state. Following load removal and then load reversal, the sliding occurs on the first interface followed by the second interface pushing the bolts again into double curvature in the opposite direction. This provides the AFC with a "pinched" hysteretic behaviour, requiring less work to be done (or energy to be spent) to recover all of the travelled displacement compared with a rectangular typical hysteretic behaviour. This provides the SHJ with an inherent dynamic self-centring capability, which is desirable.

4 IMPROVEMENTS TO DEVELOP OPTIMISED SHJ (OSHJ) AND AVAILABLE DESIGN GUIDE

Due to a number of conditions, the SHJ could not satisfy all of its ideal performance definitions. These are briefly outlined below and are the main drivers behind developing the OSHJ.

- The AFCs, as the SHJ's key energy dissipating components, were subject to a considerable post-sliding clamping force reduction because of the following reasons:
  - The bending moment, shear force , and axial load (MVP) interaction in the bolt body during stable sliding state (Ramhormozian, Clifton et al. 2017, Ramhormozian, Clifton et al. 2019) may further yield the bolt material causing the post-sliding bolt tension to drop. It is worth

noting that the bolts are traditionally tensioned into the post yield condition at installation in practice, using part-turn method of tightening according to NZS3404 (1997/2001/2007), making this clamping force reduction inevitable. Hence tightening the AFC bolts on installation only into the elastic range seemed a desirable solution.

  - The prying effect at the SHJ beam bottom flange level under large beam-column relative rotation may impose an additional axial plastic strain on the AFC bolts causing the post-sliding bolt tension to reduce.
  - Wearing of the sliding surfaces may reduce the total thickness of the AFC plies per bolt, resulting in the bolt tension reduction.
  - AFC bolts may rub against sides of the cleat elongated holes. This may reduce the AFC bolts' integrity, causing a bolt tension drop.
  - Vibration induced self-loosening, joint creep, and bolt relaxation are amongst the common bolt tension loss reasons that may be considered for any bolted connection amongst which the AFC may not be an exception, although this is very unlikely to occur in these joints in buildings

- The optimum sliding components material and surface preparations of the sliding surfaces had to be researched and identified. This was to identify an optimum yet common material type and surface preparation level to be recommended in practice associated with a consistent and accurate coefficient of friction as well as minimum degradation of the sliding surfaces and clamping force.
- An accurate, precise, reliable, and versatile bolt tightening method in the elastic, and not plastic, range of the bolt tension was needed to be developed so that at the time of installation the delivered clamping force could be accurately what the designer has identified.
- The optimum level of bolt tension in the elastic range needed to be researched and identified to exhibit an optimum seismic performance with minimum degradation and clamping force loss.
- Retaining the design bolt tension as well as minimizing its variability both over the life time of the building and during as well as following a severe seismic induced sliding was needed.
- Using Belleville springs (BeSs), a conical washer type spring (see figure 2), was the best initial option considered as a remedy to the bolt tension loss. However this had to be researched and the way of designing and installing them to fit the purpose, had to be developed. Moreover, a potential disadvantage of retaining the post sliding AFC clamping force could have been reducing the self-centring capability of the system through exhibiting resistance to slide while the system tends to come back to its original position. This issue had to be researched and addressed too.

All of the abovementioned challenges were researched, and overcome through detailed analytical, numerical, and extensive experimental research to develop the Optimised Sliding Hinge Joint (OSHJ). Figure 7 shows the OSHJ layout from different angles. Note that the BeSs and shims are shown in red colour.

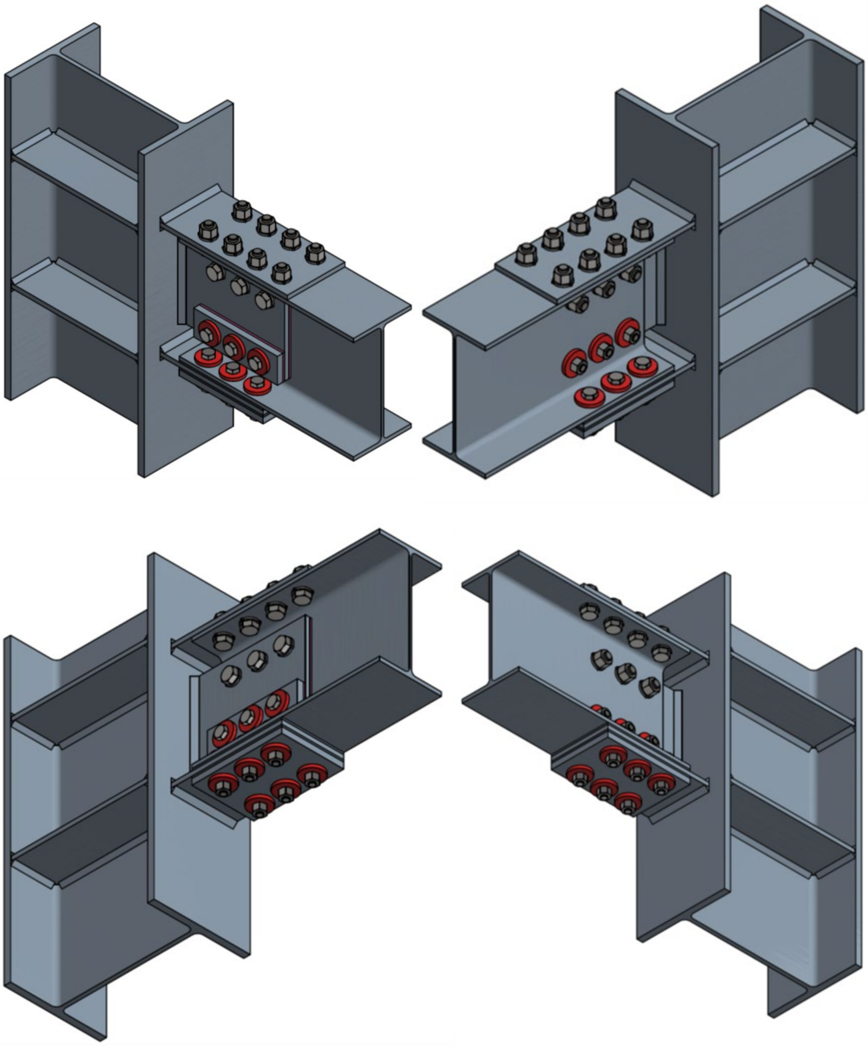

Figure 7: 3D Elevations of the Optimised Sliding Hinge Joint (OSHJ) (Not to Scale) (Ramhormozian and Clifton 2020)

The first comprehensive article on the design and detailing provisions and design example for the traditional SHJ was published as a HERA report, i.e. HERA Steel Design & Construction Bulletin (DCB) No. 68, in 2002 (Clifton, Butterworth et al. 2002). This explains a background to the SHJ developments, performance of the SHJ and its design philosophy, design and detailing of the SHJ, design of the MRSFs incorporating the SHJ, and SHJ design example. A more detailed and comprehensive version of this article, as a PhD thesis on the SHJ and other seismic resisting systems developments, was published in 2005 (Clifton 2005). The outcome of the research and practical applications has led to modifications of the original design procedure and detailing requirements published in 2007 (Clifton 2007). In 2009 Steel Construction New Zealand (SCNZ) published a design example of the SHJ (Cowie 2009) which is adopted from (Clifton 2005). Finally in 2020 a design and installation guide was published by Heavy Engineering Research Association (HERA) (Ramhormozian and Clifton 2020) which supersedes the abovementioned guides and needs to be followed in any use of the OSHJ given the optimised performance. The key modifications implemented to develop the OSHJ and included in (Ramhormozian and Clifton 2020) are as follows:

- ✓ Material selection of the shims;
- ✓ Surface preparation of the sliding surfaces;
- ✓ Coefficient of friction to be used;

- ✓ Allowable range of the AFCs installed bolt tension in the elastic range;
- ✓ Sliding shear capacity calculation incorporating modified bolt model, modified strength reduction factor, and modified overstrength factor;
- ✓ Design and use of the Belleville Springs (BeSs) as necessary components;
- ✓ Method of checking and tightening the bolts in the elastic range with BeSs;
- ✓ Occasional changes in detailing and dimensions;
- ✓ Philosophy and approach of considering ULS earthquake loads.

5 MODELLING, LOADING, ANALYSIS, AND DESIGN OF THE MRF WITH OSHJ

The modelling of the structure has been undertaken using the same approach as the traditional MRF, i.e., the MRF beam-column connections are defined as moment (rigid) connections and the analysis is undertaken under any desired load combination. The column base was modelled as a pinned support as there were some practical difficulties in creating a base fixity with the existing pile details. The column bases have been analysed and designed as pinned, to reduce any foundation alterations. Minor foundation alterations were required to attach the portal frame bases to the existing piles to transfer the vertical and lateral loads.

Due to some structural irregularities, the seismic actions were calculated using Modal Response Spectrum (MSA) Method. As for the ductility factor, the OSHJ is reliably able to accommodate very high ductility demand up to 4, which is the value given in NZS1170.5:2004 (2004) at which the strength requirements for the ULS limit state are the same as those for the SLS limit state. Hence a ductility factor of 3 was used for the analysis.

As for the design lateral load, it is required to check for the ULS design moment from earthquake (which includes the reduction from the elastic load due to ductility) and the ULS design moment from wind and only use the greater of the two as the design moment for the OSHJ. Note that softening of the traditional SHJ after sliding which made the structure more vulnerable to wind induced deflection is no longer the case with the OSHJ. The OSHJ itself is then sized to resist only the maximum of ULS design moment from earthquake and wind with ignoring joint moments induced by gravity only. The design shear force is the maximum of two shear forces namely (design shear force from permanent and imposed load for use in conjunction with earthquake + design shear force derived from out-of-balance design lateral load resulted (earthquake or wind whichever is larger) moments acting on the clear beam length) and (design shear force for full factored permanent and imposed load).

The beam is sized to develop a moment capacity being able to resist the maximum moment from applied vertical loading in a simply supported manner and then checked for providing adequate frame stiffness. If the beam size, and consequently stiffness, is required to be increased, the moment input through the joint into the column does not have to increase accordingly. This is a major simplification in design and versatility in selecting the beam size offered by the OSHJ compared with the traditional MRFs. The columns are then designed to resist the overstrength action developed by the joint, not that from the beam. This means the beam depth can be chosen for gravity strength and lateral stiffness control without impacting on the column design.

6 CONCLUSION

This paper covers the application of the OSHJ to the retrofit of an existing 1969 building, which was assessed as 45%NBS, with non-ductile modes of failure in a severe earthquake. It includes development details of the OSHJ, highlighting the characteristics which make it so well suited to this application, which requires the retrofitting system to carry the seismic load away from the existing structure up to a defined level of earthquake demand and then to resist additional deformations from a more severe earthquake in a controlled manner, protecting the existing structure from failure under combined vertical and lateral loading.

As of April 2025, the retrofitted MRSFs with OSHJs are being installed into the existing building.

7 REFERENCES

Clifton, C. (2007). Sliding Hinge Joint: Revisions to the Design Procedure. NZ Heavy Research Engineering Assn (Inc), Manukau City.

Clifton, G. C. (2005). Semi-rigid joints for moment-resisting steel framed seismic-resisting systems. PhD, University of Auckland.

Clifton, G. C., J. Butterworth and T. Miller (2002). The Sliding Hinge Joint: Design and Detailing Provisions and Design Example. HERA Steel Design & Construction Bulletin.

Cowie, K. (2009). "Semi-rigid Sliding Hinge Joint." Steel Construction New Zealand.

Danner, M. and G. C. Clifton (1995). Development of moment-resisting steel frames incorporating semi-rigid elastic joints. Manukau City, New Zealand, Heavy Engineering Research Association (HERA).

Engelhardt, M. D. (2001). Ductile Detailing of Steel Moment Frames: Basic Concepts, Recent Developments and Unresolved Issues. XIII Mexican Conference on Earthquake Engineering. Guadalajara, Mexico.

Fugazza, D. (2005). Use of Shape-Memory Alloy Devices in Earthquake Engineering: Mechanical Properties, Advanced Constitutive Modelling and Structural Applications. PhD, Universit`a degli Studi di Pavia.

Khoo, H. H. (2013). Development of the low damage self-centering Sliding Hinge Joint. PhD, University of Auckland.

MacRae, G. and C. Clifton (2013). Low Damage Design of Steel Structures. Steel Innovations 2013 Workshop, Christchurch, New Zealand, Steel Construction New Zealand.

MacRae, G. A., G. C. Clifton, H. Mackinven, N. Mago, J. Butterworth and S. Pampanin (2010). "The Sliding Hinge Joint Moment Connection." Bulletin of the New Zealand Society for Earthquake Engineering **43**(3): 202.

NZS1170.5:2004 (2004). Structural Design Actions - Part 5: Earthquake design actions. New Zealand, Standards New Zealand.

NZS3404 (1997/2001/2007). Steel structures standard, incorporating Amendments 1 and 2, Wellington [N.Z.]: Standards New Zealand.

Ramhormozian, S. (2018). Enhancement of the Sliding Hinge Joint Connection with Belleville Springs. PhD, University of Auckland.

Ramhormozian, S. and G. C. Clifton (2020). Optimised Sliding Hinge Joint (OSHJ): Design and Installation Guide for a Low Damage Seismic Resisting System - R4-155. Heavy Engineering Research Association (HERA), Auckland, New Zealand.

Ramhormozian, S., G. C. Clifton, G. A. MacRae and G. P. Davet (2017). "Stiffness-based approach for Belleville springs use in friction sliding structural connections." Journal of Constructional Steel Research **138**(Supplement C): 340-356.

Ramhormozian, S., G. C. Clifton, G. A. MacRae, G. P. Davet and H.-H. Khoo (2019). "Experimental studies on Belleville springs use in the sliding hinge joint connection." Journal of Constructional Steel Research **159**: 81-94.

Roeder, C. (2002). "Connection Performance for Seismic Design of Steel Moment Frames." Journal of Structural Engineering **128**(4): 517-525.

Yang, T.-S. and E. P. Popov (1995). Experimental and Analytical Studies of Steel Connections and Energy Dissipators. Berkeley, University of California, Earthquake Engineering Research Center.