Steel Innovations Conference 2013
Christchurch, New Zealand
21-22 February 2013

STEEL MOMENT FRAMES WITH SLIDING HINGE JOINTS – LESSONS LEARNT DURING IMPLEMENTATION

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ABSTRACT

A moment resisting steel frame is a key structural system which enables engineers to frame openings and embellish architectural design solutions providing view and amenity to buildings. A traditional ductile MRF is typically a lateral resisting system that when well-designed will develop a hinge in the beam section and the column base. Moment frames can be gravity or seismic governed and often it is the beam stiffness that designers will optimise to reduce building drift.

In 2005, to reduce seismic hinge generated damage in the traditional ductile beam part of MRF's, Associate Professor Dr Charles Clifton (UOA), invented the Sliding hinge joint (SHJ). The connection is an Asymmetrical Friction Connection (AFC) that pivots about the top flange, with friction generated sliding resistance between bolts and shim plates located on the web and bottom flange providing energy dissipation, with numerous technical papers have been written on the research conducted to date.

In summary the frame is suitable for high ductility in the range of 3-4 with the joint designed to protect the beam and column from seismic damage. It offers the added advantage of decoupling beam stiffness and strength, meaning the column is sized for the overstrength generated from the connection rather than the beam. The joint also reduces and almost eliminates slab damage due the top flange acting as pivot point with the slab topping being isolated. Recent evolutions of the design have reviewed the joint's self-centring capability and looked to enhance its already good performance with additional devices, aimed at promoting dynamic self-centering and retention of joint stiffness after significant sliding has occurred.

Since 2005, over $1 Billion of Commercial and Residential multi storey buildings have been designed and delivered with MRSFs with ductile Sliding hinge joints. The semi rigid joints have proven simple to design and construct good robustness, resilience, provide good self-centring and are simple to recover post event.

This paper provides designers summary of key lessons learnt based on observations form implementing several building projects with SHJ's. It provides simple design guidance and considerations for implementation of the SHJ.

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Introduction

The art of seismic design enables the built environment to respond in a desired fashion to imposed ground shaking induced by seismic forces and movements. Developing a low damage solution requires complete engagement of the entire design team to ensure the structural and geotechnical engineering solutions for buildings and bridges encompass resilient vertical and lateral support systems, yet achieve necessary economic, architectural and societal demands.

Being located in one of the most seismically active zones of the world, has driven New Zealand based academia and practicing engineers to pioneer the development and application of our modern seismic design philosophy better known as “capacity design”. This recognizes the expected behavior of building systems and allows damage to occur in predetermined locations, whilst protecting others against the expected overstrength actions that may develop in connections, columns and hold down bolts.

A fundamental underlying principle of our structural codes and standards is that buildings are designed with life safety in mind. That is, engineers design buildings to allow safe egress, yet be allowed to have ductility, and hence damage which has seen the wholesale demolition of buildings following the Christchurch Earthquakes.

In response to the devastating events, our industry has learnt key lessons and in response our engineering skill, knowledge and analytical capabilities have increased which has also seen a growing societal expectation that “engineers can solve the problem” and will easily provide resilient and recoverable buildings.

This is not an unreasonable assumption which continually challenges our industry best practice and standards to evolve to enable improved resilience to nature’s forces in the design of buildings and infrastructure

More following the Christchurch earthquakes, clients now more than ever are engaged and supportive of our industries drive towards low damage or recoverable structures which has seen researchers and designers working harder than ever to evolve systems towards improved performance objectives.

Our Building Performance Levels

In New Zealand the seismic design codes and standards requires engineers consider three seismic performance levels for “normal” buildings. A serviceability limit state (SLS) earthquake relates to a return period of a 1 in 25 year earthquake. After such an event, it is intended that there should be no permanent damage to the structure or its contents.

A design level or ultimate limit state (ULS) event for a normal building is a 1 in 500 year return earthquake as experienced in Christchurch in September 2010. It is intended that the building shall not collapse, allow egress and preserve occupant life safety. However, significant damage to the structure and contents (due to shaking) is likely and accepted under current design philosophy.

The third paradigm is called a maximum considered event (MCE), but may be similar to a 1 in 2,500 year earthquake event. During this event, it is expected that a modern, well designed structure would not collapse, but it would be extremely unlikely to be repairable. It is anticipated that although modern structures may survive the initial earthquake event of this size, collapse may occur during aftershock sequences.

To ensure that modern buildings achieve these seismic performance levels, they are designed with capacity design principles to have strategic “plastic hinge” or regions where localised damage occurs, much like that of a crumple zone on a car or a fuse in an electrical system.

However, damage sustained to these plastic hinge regions’ during a severe earthquake often results in some loss of function of the building and usually results in costly or irreparable damage to the structure requiring demolition.

Surely as we learn to better plan for damage, a considered approach will be to engineer systems resulting in less damage to buildings and their contents with the primary structure being recoverable and only requiring replaceable fuses or parts, resulting in significantly less potential loss following future events.

Using Steel Moment Resisting Frames as a seismic system

As the Christchurch rebuild gets underway in anger, a key structural seismic bracing system is the perennial Moment Resisting Fame (MRF). Its appeal is not necessarily evident in its stunning seismic performance, but more in geometrical beauty. Consisting of only beams and column it easily achieves vistas over the growing garden city, enables open plan offices, retail future flexibility, compliant retail use on ground floor levels.

However as the new city grows, engineers should take care with their frivolous use of the MRF without considering how it deforms and how during a large earthquake it will behave.

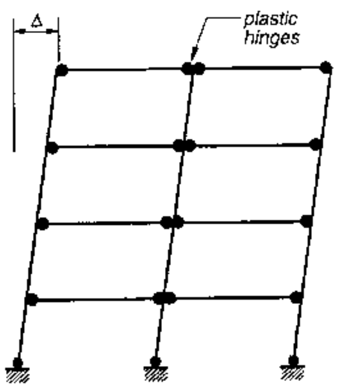

Fig. 4.1
Beam sidesway or weak-beam
strong-column mechanism

Figure 1 – Deformation/damage locations in a traditional MRF Frame

Certainly for steel framed construction, the solution structural engineers are seeking is a simple semi-rigid beam column jointing system that is cost effective to fabricate and install, ensures stability, controls hinging, building drifts and most importantly limits structural damage. These systems would also control and limit forces on bracing elements and supporting foundations.

Damage needs to be planned for in design, controlled and limited to ensure a building can remain useable after a large earthquake. Aurecon has developed an elegant solution for controlling seismic damage in steel framed buildings. We believe this technology means it is now economically viable to achieve this objective. Major new buildings constructed in earthquake prone areas should have an inherent low damage design philosophy. In the future, low shaking, low displacement solutions may reduce damage to primary structures, services, fit out and contents. To design and build superior, resilient buildings, the issue of ground shaking must be addressed i.e. minimized and a recoverable building structure should be proposed.

Recent research has focused on minimising the level of seismic damage to structures so that buildings may be rapidly reinstated for post-disaster purposes or quickly returned to their usual function following a major earthquake.

Examples of these technologies are buildings which incorporate special connection detailing, supplemental energy dissipating devices, rocking systems and base isolation techniques. For short heavy, relatively low rise buildings base isolating devices are currently a good method for combating ground shaking by providing isolation between the structure and the ground while at the same time dissipating seismic energy. The perceived expense associated with these devices often limits their application to high importance, high budget buildings. However in most cases, base isolation of structures encompasses only a small percentage of the total build cost (3% to 5% depending upon the scale and nature of the project) [2]

[2] RLB Cost of Base Isolation Cost Comparisons submitted to the Royal Commission - dated 31st March 2012.

Traditional Moment Resisting Steel Frames - Behaviour

To design and detail Steel moment resisting frames (SMRF's) designers are required to following sections 4 through to 8 of HERA's R4-76 "Seismic Design Procedures for steel structures" (Clifton 2005), and the provisions of our NZS3404:1997 (SNZ 1997) steel code. The outcome is a traditional ductile MRF, which will good levels of ductility via a weak ductile beam and strong sway column. The connections are sized to ensure that the ductility or hinge formation occurs in the beam ends and lower section of the column only, not the connections.

A significant number of laboratory tests have indicated the hinge formation will likely see the beam ends adjacent to the bolted moment end plate buckle. In the lower section of the column from ground (assumed seismic base) to first floor, a hinge will also form causing flange buckling and potentially permanent building deformation.

The hierarchy approach developed by Clifton and HERA design considers cumulative over strength actions, and aims to suppress the formation of soft Storey mechanisms in the frame via ensuring one column within a bent to remain elastic.

The reality with this form of design is that post disaster, the steel moment frame may experience out of plumb issues due to plastic hinge formation, which may mean the building becomes unserviceable. Recovering the building post this level of shaking is both disruptive and expensive.

Evolution of Steel Moment Resisting Frames (SMRFs) - Sliding Hinge Joints SHJ's and other types

In 2005 Dr Charles Clifton completed his PhD thesis (Clifton 2005a) on low damage, semi rigid connections suitable for steel semi-rigid joints for moment-resisting steel framed (MRSF) seismic-resisting systems.

Intended as the ductile link in the seismic-resisting system, in accordance with a strong column, weak joint philosophy, Dr Clifton's work designed and detailed the joints and systems to withstand a design level or ultimate limit state (500 year return period) earthquake with minimum damage. Four beam/column joint types for systems MRSF were considered. These were the:

- Ring Spring Joint (RSJ), where the beams are clamped to the columns with flush endplates and Compressible ringfeder springs
- Post-tensioned Tendon Joint (PTJ), where the beams are post-tensioned onto the columns
- Flange Bolted Joint (FBJ), where the beams are bolted to the columns through flange and web plates that are designed and detailed to undergo dependable cyclic extension and compression under inelastic rotation demand.
- Sliding Hinge Joint (SHJ), where the beam is pinned to the column at the top flange level and is connected at the bottom flange and the bottom of the web by a unique asymmetrical sliding shear detail

The RSJ, FBJ and SHJ were developed through to the experimental stage, with large-scale tests on representative joints undertaken. Following this the FBJ and SHJ were further developed, through small-scale static and dynamic testing, finite element and numerical integration time-history analyses into fully designed and detailed systems. The FBJ was reported as suitable for low ductility or within drift control frames, whilst the SHJ was found suitable for good resilience, higher ductility applications, with both systems found more economical to fabricate than compared with conventional MRSFs.

Design procedures and detailing requirements for the two fully developed systems were presented in Dr Clifton's thesis along with results from physical tests compared with outcomes from his analytical modeling. Dr Clifton then explored and provided design and detailing requirements for the joints and the frames are covered and fully worked design examples for the Flange Bolted and Sliding Hinge Joints are presented. The results reported in his thesis and later during application during various construction projects demon that the semi-rigid, strength-limited joints developed, when used in properly designed moment-resisting steel frames, have considerable advantages over conventional rigidly jointed frames for meeting strength, stiffness, ductility, damage-resistance and economic criteria. This document was then later used to create Steel Design & Construction Bulletin (DCB) 68 (Clifton 2002) and HERA's R4-134 (Clifton 2005b) which provides a design guidance note on the application of the Sliding Hinge Joint.

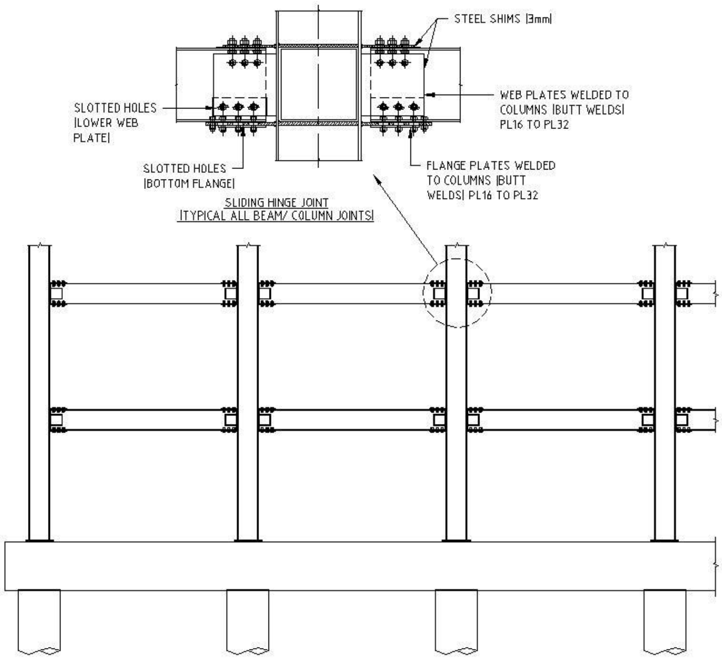

Figure 2: Sliding hinge joint frame assembly.

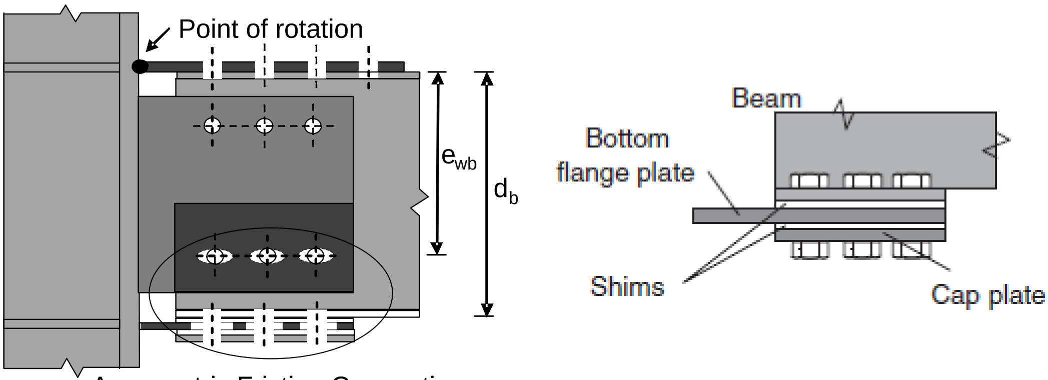

Figure 3: Sliding hinge joint and beam column joint assembly.

Sliding Hinge Joints (SHJ's).

The performance of steel moment resisting frames may be improved through the use of Sliding Hinge Joint connections between beam and columns sections. Sliding hinges joints provide ductility without the local damage that occurs in the conventional plastic hinging moment frame system. Additionally any damage in the hinge system is restricted to items which may readily be replaced.

SHJ's Response to Earthquakes – Joint and Frame Behaviour

The Sliding Hinge Joint (SHJ) is a low damage connection which functions via rotation generated by seismic moments. The top corner of the beam is pinned to the column through the top flange plate which acts as the point of rotation, with asymmetric friction connections (AFCs) in the bottom web and bolt groups that slide to allow inelastic joint rotation as shown in Figure 3. The sliding hinge joint acts to open and close, each time pivoting about the top flange connection as indicated below.

The AFCs are designed to be rigid under working load and serviceability level events (SLE), slide under design level earthquake (DLE) loadings, and return to an effectively rigid connection at the end of the earthquake shaking.

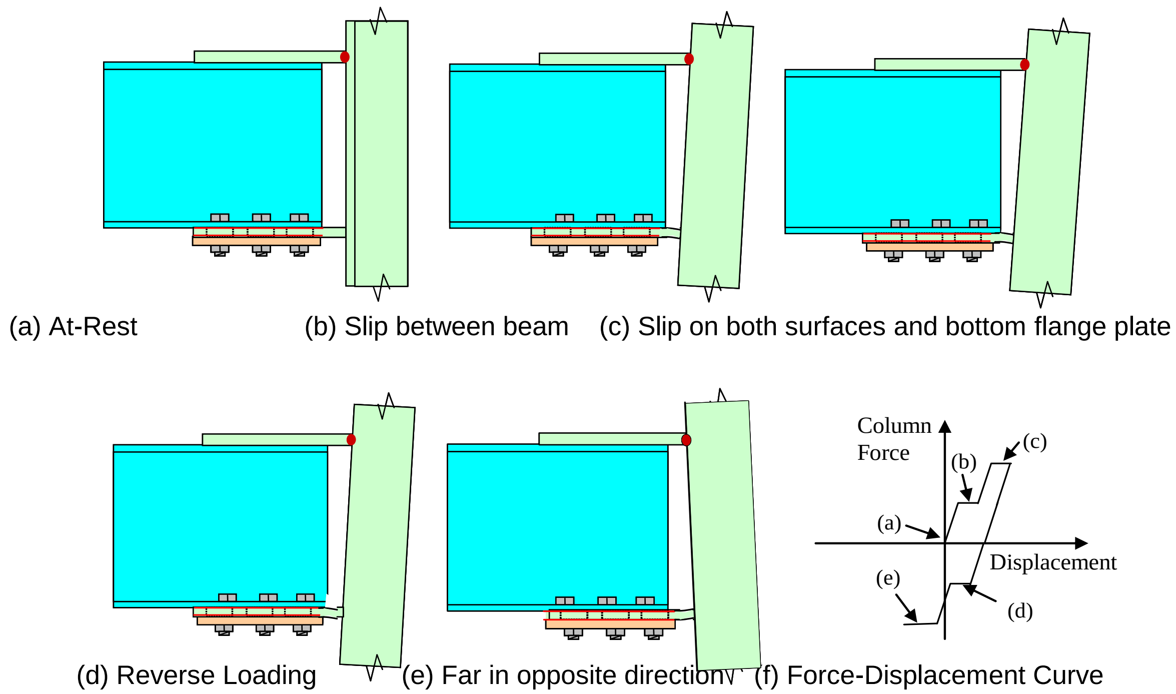

Figure 4: Sliding hinge joint and beam column joint behavior – Courtesy of MacRae et al. (2010)

Advantages of Sliding Hinge Joints

The SHJ has several advantages, over traditional welded joints namely;

- Lower cost. The use of sliding hinge joints has been proven as a cost effective solution for high rise buildings.

- In some cases sliding hinge joints can reduce structural costs through a reduction in column and foundation elements, through the sum of joint overstrength actions being weaker than the sum of beam overstrengths.

- Decoupled joint strength and stiffness which allow the use of smaller joint overstrength factors allowing the designer to control seismic drifts without introducing larger forces in columns and foundations.

- The asymmetric rotation about the top flange plate also isolates the slab and minimises floor slab participation and damage.

- The sliding hinge joint is formed by adding abrasion resistant high hardness steel shims (such as Bisalloy 400) and slotted holes to conventional flange plate connections. This enables the web and bottom flange to slide placing the bolts in double curvature, helping to provide joint opening and closing momentum.

- Sliding hinge joints in moment resisting frames and can be employed in the beam column joints and at the column base, protecting the column section from hinging.

- Joint performance may be enhanced though the addition of Belleville springs which limit bolt deformation, maintain bolt tension and reduce the need for post event repair.

- Limited Joint Stiffness and strength degradation ensuring stable hysteresis and energy dissipation.

- Improved dynamic re-centering ability,

- Ease of repair following an earthquake as any significant inelastic demand is confined to the bolts

Disadvantages of Sliding Hinge Joints

During the application of SHJ's on various projects we have only discovered minor weaknesses worth discussing or considering in responses to concerns raised by other consultants such as;

- The SHJ does undergo some loss of elastic strength and stiffness once forced into the sliding state. Finite element analysis and experimental testing of the AFC by Clifton et al indicated that the fully tensioned (i.e. yielded) high strength friction grip bolts are subject to the interaction of moment, shear and axial forces generated during sliding. This leads to further yielding and hence a slight reduction in tension. Moreover, non-linear time-history analyses on five and ten Storey frames by Clifton indicated the frames may be subject to small residual drifts following a major earthquake, unless columns are design as elastic or re-centering devices are added.

- Durability. The SHJ should only be used in dry sealed environments away from salts or moisture, (based on current knowledge). Further research on this matter will be presented at this conference.

- Spatial limitations of bottom flange bolts. Due to the downward projection of the bottom flange bolts care and consideration are required to integrate the SHJ solution into the architectural building envelope. This has often required additional bulk heads, window framing and architraves to create space needed to reticulation and future re-access to the joint.

- Only high hardness, low alloy, high abrasion resistant plate should be used as shims, such as Bisalloy 400 with specified Brinell hardness of 370-430 HB. Although this plate is an imported material, Aurecon found this to only be slightly dearer than standard plate. The use of mild steel shims would result in greater variation in sliding resistance, which requires larger strength reduction and overstrength factors in design. The low alloy Bisalloy shims very found to be very resilient and did not display degradation during severe testing as reported by Khoo et al. (2012a).

- Design, verification and quality assurance efforts. Due to the significant number of small and detailed parts, there is 5-10% increase in the installation and verification effort required for each joint. This is primarily due to the number of plates, bolts and the bolt tightening requirements. However once design tools are created, the design and specification of each joint type becomes routine. Similarly after a fabricator and structural engineer have designed and installed a small number, efficiencies and a procedure is quickly developed and the joint becomes routine.

- Concerns have been raised around the performance of the top flange plate behavior (curvature) under cyclic rates of dynamic loading, weld resilience and fatigue issues and top plate elongation. Khoo et al. (2013b) have completed an in-depth study on this matter and will address all concerns in their upcoming paper in the NZSEE Bulletin.

- Concerns were also raised on interaction with the frames and diaphragm floors. Our view is that due to the resilience and duplicity of web and flange to floor connections and evidently low damage at high rotations observed in testing, we are unconcerned by the diaphragm floor performance.

4 cycles > 2% drift

32 cycles > 2% drift

Fig 5 C.Clifton – Report to the Royal Commission March 2012

SHJ's - Previous Project Applications

Bellagio Apartments
In a New Zealand first application of this technology, Aurecon (as Connell Wagner), successfully used sliding hinge joints in the eleven storey Bellagio Apartment Building, in Taranaki Street, Wellington.

Te Puni Student Village – Low damage solution for Moment Resisting Frames
For the Te Puni Student Village (Gledhill et al. 2008) project at Victoria University of Wellington, three buildings ranging from five to 11 stories in height had moment resisting frames in the longitudinal direction with Sliding hinge joints connecting the beams to columns. The terminal columns were shared with the rocking CBF coupling beam between frames have been utilised, to confine any plastic distortion to the bolts associated with the bottom flange connection, making for a readily reparable connection should a major earthquake occur.

Figure 6: Sliding hinge joint and beam column – Te Puni Student Village

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, the joint will slide, allowing rotation to occur. The capping plates are locked into position by the bolts allowing it to slide relative to the flange and web surfaces. Once the imposed moment reduces the sliding stops and the joint becomes rigid.

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 at bottom flange and bottom bolts of the web plate. The top plate pin keeps undesirable floor slab participation and damage to a minimum.

The philosophy of the joint ensures performance characteristics are achieved for both the DBE and the MCE. The joint is suitable for high ductility, high rotation applications. The design ensures at the DBE, inelastic rotation occurs within the slotted holes equating to only minimum joint degradation and minor slab cracking may occur. At the MCE the SHJ's will retain its integrity but may suffer minor joint/slab damage.

One of the main advantages of this form of construction is the stiffness of the beam can be divorced from its strength. This has the major advantage that the overall stiffness of the building can be based on beam sections that would potentially be too strong for the columns.

For the MRF frames the ductility level adopted was been based on μ=2, at this ductility no additional P-delta effects needed to be incorporated into the analysis. The sliding hinge joints have been designed in general accordance with the recommendations of the HERA DCB 68 and subsequent recommendations from HERA.

Figure 7: Te Puni Student Village

MRSF with Column Base Hinge Protection

Also at Te Puni Student Village, the column bases of the MRSF frames were designed to allow for large rotations in the extreme MCE event. Normally this is achieved as a flexural hinge, allowed to develop within the steel section. At Te Puni, the column bases were designed in a similar fashion as a sliding hinge joint, so that any extreme rotation takes place in a sliding mode between two plates bolted together. The bolts are selected so that in conjunction with the gravity axial load the desired moment can be resisted.

The bases of the columns have a sliding hinge type connection so as to obviate the possibility of inducing a hinge within the section. These joints employed vertical friction plates, a slotted plate arrangement for sliding and base shear key plate.

Elevate Apartments (under construction)

This fifteen storey steel framed apartment building utilises MRF frames as longitudinal direction bracing. The ductility is provided by virtue of the nonlinear force deformation characteristics of the beam to column connection. This is achieved by the moment capacity of the connection being controlled by the friction across a bolted interface and slotted bolt holes to allow for movement. The moment capacity of the connection will be less than that of the bare steel section so as to restrict any initial inelastic deformation to the SHJ.

As a moment frame the basic demands that result in the nonlinear performance are these beam moments and as such are fundamentally controlled by the shears in the storey above and below the level under consideration. For this reason the base shear derived from a modal analysis is to be scaled to that corresponding to the equivalent static requirement in accordance with NZS 1170.5 (SNZ 2004).

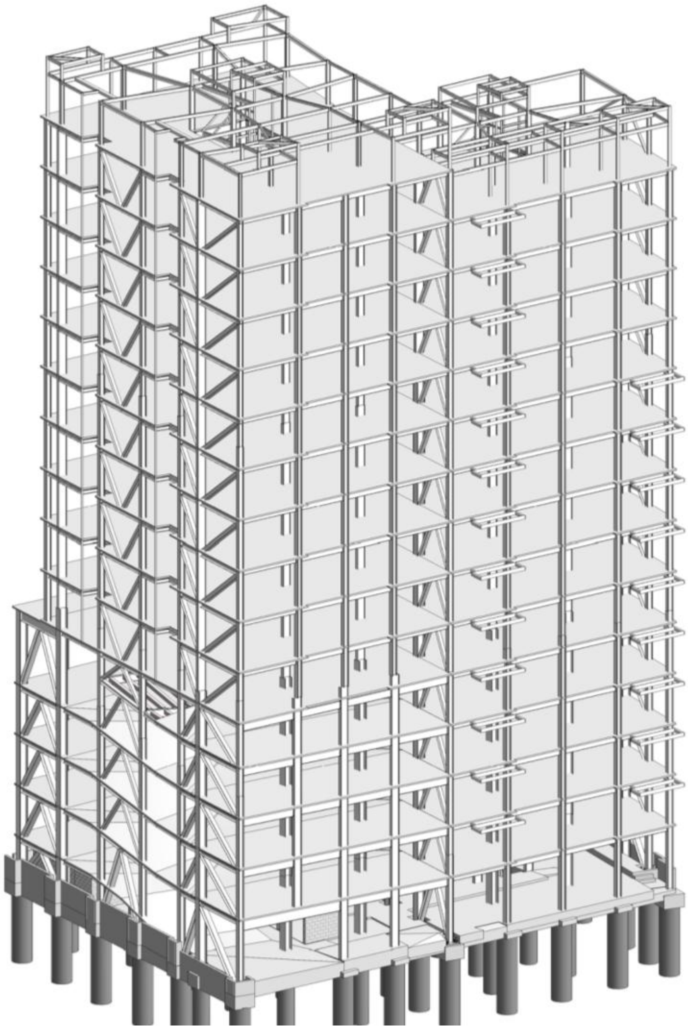

Figure 8: Elevate Apartments, Wellington

The stiffness of a SHJ moment resisting frame is not directly dependant on the stiffness of the beam section as the connection strength is not controlled by the section properties for strength or stiffness. However, like most steel moment frames the control of building drift is a controlling influence on the beam section selection. For this apartment block the floor to floor height is such that the possible beam depth was restricted to 460 mm.

The stiffness of these beams will be evaluated by considering the partial composite stiffness for all beams remote from the perimeter of the structure. There will be sufficient shear studs to achieve 25% composite action and the stiffening effect evaluated in accordance with NZS3404:C13.1.2.6 this inertia i.e. will be averaged with that of the bare steel section to obtain an overall member flexural stiffness. Inter storey drifts will be evaluated incorporating the kdm factor of NZS 1170.5

The columns of these frames will designed using capacity design procedures to fulfil the normal provisions relating to the columns of a steel moment resisting frame.

Structural Modeling and Analysis

Site Specific Data

A detailed geotechnical investigation is required, providing expected site subsoil velocities and subsoil flexibility for scaling of an appropriate suite of earthquakes if NLTHA is required.

Building Modeling

Most building structural arrangements can be adequately analysed using current versions of ETABS NL or SAP 2000 Advanced by Computers and Structures Inc of Berkeley California. Both these packages are capable of suitable analysis forms including simple equivalent static, modal, pushover and nonlinear time histories.

Often a single or planar frame can be simpler to validate than a 3D model. Careful and experienced use of these packages enables the addition of linear elastic and nonlinear hinge properties where these are required and understood. Typically the locations of nonlinear behavior are beam ends and at the base of the frames a non-linear spring element should be considered.

A base model of the structure should be created and subsequent models for the various possible arrangements of accidentally generated from the base model. In generating this base model all principal structural members should be included so as to capture the mass of the building. The floor slabs should be modeled as accurately as possible, with suitable FEA concrete elements, meshed to ensure diaphragm interaction between the floor and MRF beams.

The models should take into consideration all of the requirements of our codes and standards, particularly NZS1170.5, including P-Delta, for horizontal and vertical regularity. The form of analysis undertaken was a response spectrum model analysis and with the results (for regular buildings), scaled to be such that the base shear was 80% of the shear determined by an equivalent static analysis.

Additional area mass was ascribed to the main floor area to account for super imposed dead load, seismic live load, partitions and the mass of any precast stair flights. Models for the various eccentricities should also be generated, suitable consideration for flexible or rigid diaphragms being modeled.

Where corner columns or compound members are shared by orthogonal seismic systems, an equivalent section should be modeled, orientated in the appropriate direction, with consideration of all appropriate flexural, axial and appropriate stiffness property modifiers.

Based on the output from these models the critical eccentricity case for the design of the frames can be identified.
Further models should be created from the base model to conduct push over or NLTHA studies. These can be used to further assess the performance of various NL items such as the SHJ selected (i.e. post design, or items such as a flexural pre-tensioned spring base connection). These models considered various assessments of the nonlinear properties of these connections.

In analysis of the MRF frames, Aurecon typically employs two push over models, with a target displacement of the COM equal to 1.5% drift and maximum allowable joint rotation equivalent to 0.03 at radians at maximum drift demand.

These push over studies all used the equivalent static load distribution associated with a linear acceleration profile as a loading pattern that was incremented until the deflection at the uppermost concrete floor reaches its expected displacement. For ductile systems the deflection the response spectrum actions are scaled by an appropriate Sp factor, with consideration of K_{dm}.

To check the influence of higher modes on the design of the elements of the MRF system the response in these modes was determined by using a specially truncated response spectrum. The response at all periods above those associated with first modes for translation in each direction and first mode torsion was set to zero. This response was than combined with the pushover actions as root sum squares combination to obtain the actions to be used for the design of the members.

Steel Moment Resisting Frames with SHJ’s - Design Procedure

To design the sliding hinge joints is a simple process but multiple step process, and we would recommend designers following the procedures outlined in HERA DCB 68, R4-134, with minor updates as per HERA Bulletin in 2007 and consideration of the latest bolt model, recently released in 2012 from UOA. We would also recommend engaging the services of Dr Charles Clifton (as inventor and expert) to validate the outcomes via peer review.

In general the SHJ is suitable for high ductility applications and we would recommend consideration of μ=3 - 4 as a guide. The joint should remain elastic in response to SLS earthquakes, ULS wind, and respond in a nonlinear manner to a design level or ULS seismic event.

SHJ - Performance Objectives

Section 2.1 of HERA's DCB Page 6 No. 68, June/July 2002, outlines the following objectives, which we have added below herein with commentary where applicable. The MRSFs with SHJs have been developed to deliver the following performance characteristics for the three levels of earthquake described below;

(1) For the serviceability limit state earthquake (i.e. as represented by 1170.4 Section 2.1.1, involving a return period of 25 years for normal structures (as defined by Table 3.1 of AS/NZS 1170.0 [SNZ 2002]):

- The joint and system shall remain effectively rigid, with negligible inelastic action from any component
- This condition shall apply even when the system has been subjected to a prior ultimate limit state design level event.

(2) For the design level ultimate limit state earthquake (i.e. as represented by NZS1170, involving a return period of 500 years for normal structures):

- Negligible inelastic demand in the beams
- Minimal inelastic demand in the columns at base level (such that fixed column bases will be readily repairable) and none at higher levels. The author notes that Aurecon have evolved this provision, with the addition of a column base hinge protection device, precluding hinge formation in the lower column section.
- The rotation demand on the joints is not to cause the bottom flange bolts to contact the ends of the slotted holes
- Column panel zone rotation demand to be 1%
- P - D effects to be accounted for either through provision of suitable frame stiffness through increased strength (i.e. Satisfying Clause 6.5.4 of 1170.5).
- Lateral drift not to exceed 2%. We would recommend consideration of tuning the structure to 1.5% interstorey drift to reduce the extent of shaking related damage on facades, fitout and services.
- The positioner bolt may need replacement. We would consider if a positioner bolt is required.
- Minor cracking only to the concrete floor slab surrounding the frame. We would note that subsequent tests by Khoo et al. (2012b) indicate very limited slab damage around the column, even at very high drift/rotations.

(3) For the maximum considered earthquake (i.e. based on a 2500 year return period event or higher):

- Negligible inelastic demand in the beams, except in the vicinity of bolts to the flange and web plates
- Inelastic demand in the columns to be able to be dependably resisted(this applies especially at the base, which is the only location likely to be subjected to appreciable inelastic demand)
- In the extreme case, joint rotation demand may cause the bolts to impact the ends of the slotted holes, requiring replacement of the sliding bolts and possibly bottom flange plate replacement
- Panel zones may rotate in excess of 1% strain demand
- Lateral drift to be within sustainable limits, including the influence of P – D effects
  The positioner bolt will need replacement
- Minor cracking only to the concrete floor slab surrounding the frame.

Application of the design procedures for the force based method of design involves:

- Analysing the frame for the design level earthquake using the Equivalent Static
- Method or the Modal Response Spectrum
- sizing the members and connection components to meet the required strength and stiffness criteria for this event
- Following the joint design and detailing provisions given herein (section 3) such that the joint can sustain the MCE rotational demands while delivering the performance characteristics of (3) above.

Design Process - Selection of member sizes

In general the design process is as outlined in HERA DCB 68 section 3.1.

1. The beams are designed to resist the maximum applied gravity loads (dead, live loads) in a simply supported condition
2. The joint is sized to resist only the moment generated by the earthquake action, i.e. M_{code}, μ_{design}. This moment is calculated and applied independently of the beam's section moment capacity.
3. The columns are designed to resist the overstrength action developed **by the joint**, not that from the beam. Thus the beam depth can be chosen for gravity strength and lateral stiffness control without impacting on the column design.

Design Process – Sliding Hinge Joint Frame and joints

Based upon our experience and the recommendations outlined in DCB 68, the following is a summary of the procedure recommends engineers to tackle the following actions;

- Complete Gravity and Seismic analysis as above
- Determine Gravity Bending and Shear actions in beams and columns (all required load cases and combinations as defined in NZS1170).
- Determine the envelope elastic seismic actions for each beam 'bent', at each level and determine required ductility for joint design. Undertake any additional scaling of the envelope seismic actions.
- Undertake frame design process, utilizing steps outlined in section 3 and 4 or by following the worked example as outlined in section 5 of 5] HERA DCB68
- Undertake SHJ Joint design tackling steps 1-14 as below.
- Detail SHJ connection
- Schedule SHJ on structural drawings, including number of top flange, web and bottom flange bolts, plates, shims welds, doubler plates, gussets and stiffeners and welds.
- Specify bolt tolerances and tightening procedures

The author notes there is complete design example as provided in HERA DCB 68, section 5, but consideration should be made towards sourcing up to date information on bolt capacities, materials and the bolt model.

Design Process – SHJ Joint Design Steps

There are fourteen steps required to design a sliding hinge joint as outlined in DCB 68 are as follows.

1. Determine design moments and shears
2. Determine sliding bolt group layouts
3. Determine initial bottom flange plate width and thickness and initial web plate thickness
4. Determine bolt size and numbers for moment adequacy, then finalise bottom flange plate width and thickness
5. Design web top bolts for vertical shear resistance
6. Design web plate
7. Design top flange bolts and plate
8. Check on reduced tension capacity of the beam at the bolted connection
9. Design welds between plates and column
10. Dimension flange and web plates
11. Design, detail positioner bolt and shims
12. Design tension/compression stiffeners
13. Calculate joint overstrength capacity
14. Design joint panel zone. The full SHJ design procedure, starting with determination of joint design moment and design shear, is given in sections 3.4 to 3.21 of DCB 68.

SHJ Design and Detailing Tools

To assist in the design process to complete these calculation steps, Aurecon have developed in house structural software, which follows the design procedure recommended by Professor Clifton in his thesis and as outlined in DCB68 and supporting data. The software enhances the design process by designing each joint type and then enabling the user to extract the data directly to XLS or even a data file for direct addition to the structural drawings.

Automated Design Process

The following images provide a simple design example with extracts indicating key entry parameters.

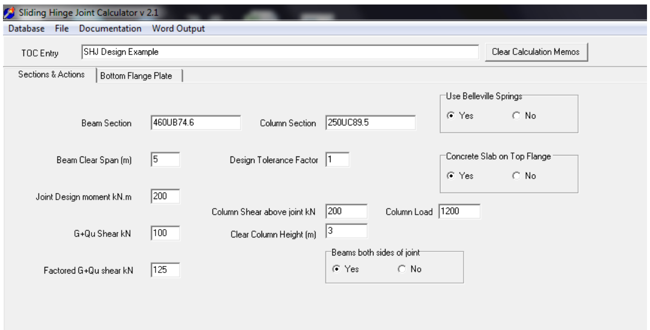

Figure 9 - Step 1 - Geometry, Beam Sizes (as preselected for drift control), seismic demands, joint type

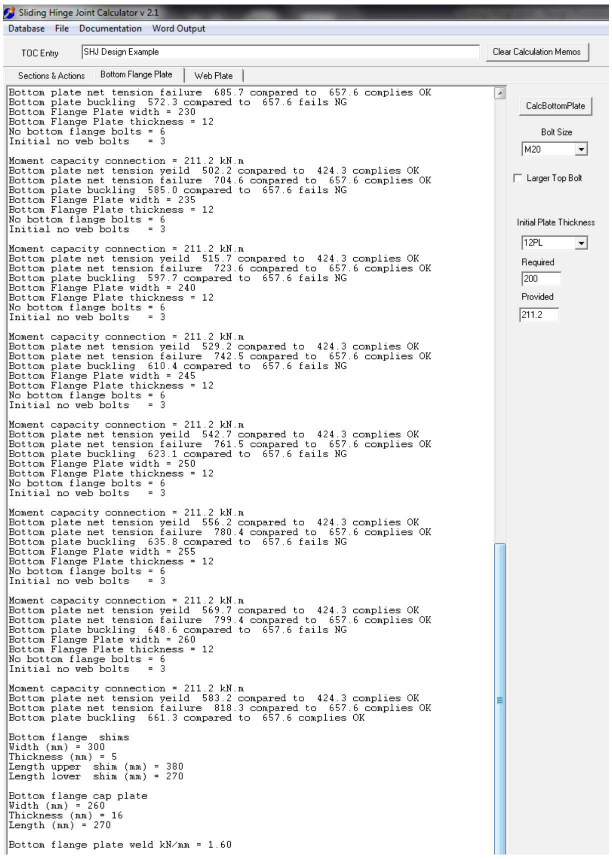

Figure 10 - Steps 2 – Joint Arrangements and Bottom Flange

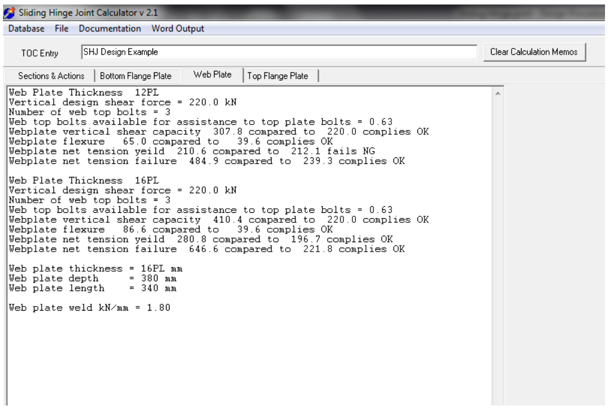

Figure 11 - Step 3 – Web side plate

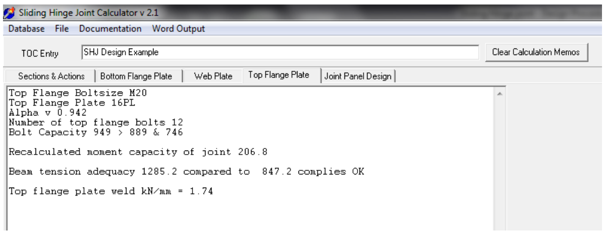

Figure 12 - Step 4 – Top Flange

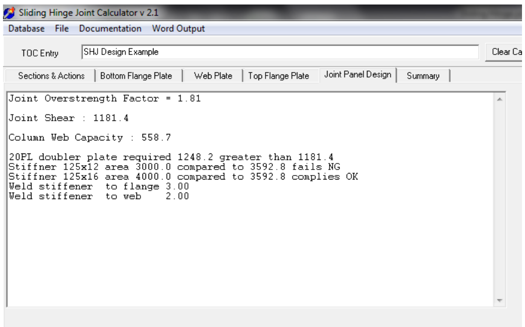

Figure 13 - Step 5 – Column Panel Shear Checks

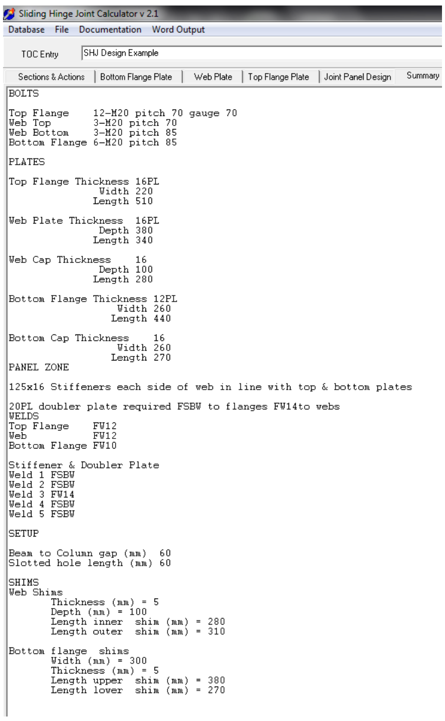

Figure 14 - Step 6 – Joint Design Summary

Outcome of Automated SHJ Design Process

This table, is 35 columns wide and typically fills the base of an A3 drawing sheet, but provides the fabrication team all relevant data on each Sliding Hinge Joint Type.

1:10

SLIDING HINGE JOINT SCHEDULE																														
			SET UP (mm)					BOLTS - FLANGE BOLTS ARE IN PAIRS				PLATES							SHIMS				WELDS TO COLUMN							
																										JOINT WELDS				
Type	BEAM	COLUMN	GAP	SLOT LENGTH	Qe	Qe1	GAUGE	TOP FLANGE BOLTS	TOP WEB BOLTS	BOTTOM WEB BOLTS	BOTTOM FLANGE BOLTS	TOP FLANGE	WEB	BOTTOM FLANGE	WEB CAP	BOTTOM CAP	STIFFENERS	DOUBLER PLATE	INNER WEB SHIMS	OUTER WEB SHIMS	UPPER BOTTOM FLANGE SHIMS	LOWER BOTTOM FLANGE SHIMS	TOP FLANGE PLATE WELDS	WEB PLATE WELDS	BOTTOM FLANGE PLATE WELDS	JOINT WELD 1	JOINT WELD 2	JOINT WELD 3	JOINT WELD 4	JOINT WELD 5

Figure 15 – Sliding Hinge Joint Design Process – Extracted to Structural Drawing

By detailing of the sliding hinge joints using a verified automated design tool improves the speed and repeatability and enables the designer to produce all joint detailing data in tabulated form referenced directly into the structural drawings, ensuring human error in transposing detail information is significantly reduced. This tool also aids in the development of fabrication drawings, installation and quality assurance checking on site.

Lessons Learnt

Key lessons learnt in the application of SHJ technology include:

Using SHJ's for regular buildings

Irregular complex buildings with varying seismic demands will require numerous SHJ's types with varying arrays of plates, bolts welds and shims. To reduce complexity and cost, the designer should look to group joint types and variants, and reduce where possible. Typical the more irregular the structural bracing arrangement, the less likely redistribution and load sharing will enable the consideration of low damage solutions such as SHJ. We recommend where possible, the advantages of SHJ's be exploited on regular structures.

Use of Belleville Springs

The addition of Belleville springs requires consideration of a slightly different bolt model, but the evidence indicates will provide a more dependable, resilient connection, with reduced degradation in bolt tension, which ultimately reduces the need to replacing or re-tensioning bolts.

Use the correct and updated bolt model

With more rigorous testing and improved understanding of Asymmetric Friction Connection behavior researchers have further developed overstrength factors, and improved bolt models. We understand the most recent bolt model for AFC has been published by Dr Clifton. We recommend designers review this and consult experts at UOA and UOC to ensure the validity of the model and its proposed joint/bolt capacities.

Ensure the correct overstrength values are used are correct

We highlight the importance of validation of the design procedure and the use of current and correct overstrength data. These values will vary depending upon AFC arrangement, bolt size, Shim material and whether or not Belleville washers are used. It is critical to calculate the correct bolt capacity, and to understand the application of overstrength actions to ensure correct sizing of beams, columns and base hinge connections.

Shim plate Materials

Consider using low alloy shim materials offering high abrasion resistance. This considerable improves the joints toughness, reduces degradation and improves the likelihood of the joint being self-centering.

Column Doubler Plates

The sliding hinge joint demand at any one level has been based the average beam moment at the column face. These moments along with beam gravity shears, column axial loads and column shears are used to design the joint including the need or otherwise for doubler plates. In general we found joints with beams on both sides of the column require doubler plates and those with only one do not.

Bolt Tightening and Site Procedures

Once the frame is aligned, establish a rigorous bolt tightening procedure with the installers, fabricator and main contractor. Ensure a quality assurance procedure is agreed, monitored and the results from each floor level tightening provided to the engineer.

In general bolts without Belleville Washers, should all be snug tightened, starting with the bottom flange bolts and working up. The tightening pattern should be to NZS 3404, Clause 15.2.4.1. For each group of bolts (e.g. the bottom flange bolts) this means starting with the bolts closest to the column face and working along the row away from the column face. For the flange bolts, this may require two or more rounds of snug tightening to get all bolts snug tight, pulling the flange plate in hard against the flange upper shim.

The bolts are then fully tensioned, starting again with the bottom flange bolts and working up. Tensioning is to the part turn method of NZS 3404, Clause 15.2.5.2. For bolts where Belleville Springs are installed, a rough guide would be to tighten by an extra ½ turn from snug tight over that specified in Table 15.2.5.2. A more precise method would be to calculate the required extra turn based on the deflection of the Belleville Spring stack. Further research is planned for this year at UOA on this tightening method.

Use of Design Aid Tools
For consistence in application, we would recommend that an organization develops a design tool or sources HERA's SHJ XLS design tool. We recommend that HERA and SCNZ engage Dr Clifton to update the tool, add in the latest bolt model and circulate via the SCNZ or HERA websites.

Detailing Issues
The following general arrangements provide details of actual implemented Sliding Hinge joints as extracted from project structural Drawings. A key issue is ensuring the development of verification and quality assurance procedures to ensure the details are correctly constructed, installed and verified.

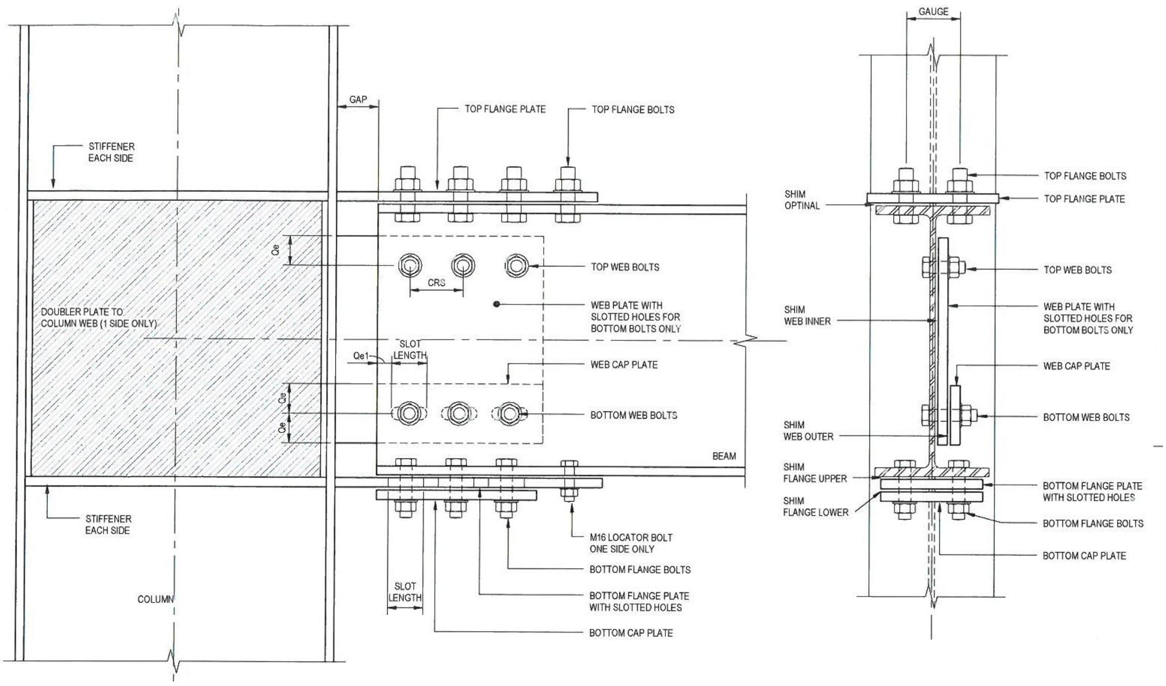

Fig 15 Sliding Hinge Joint – Elevation and Section and plans – Top Flange Section.

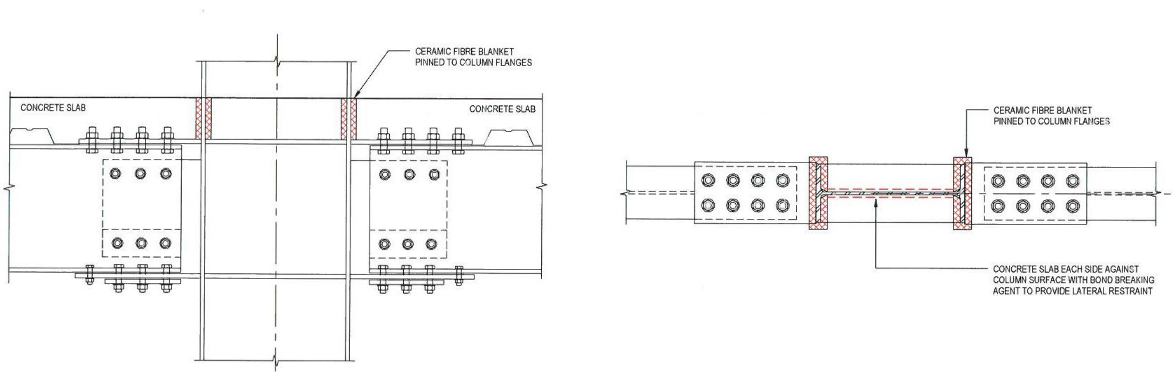

Fig 16 Sliding Hinge – Top Flange/Slab Isolation Details

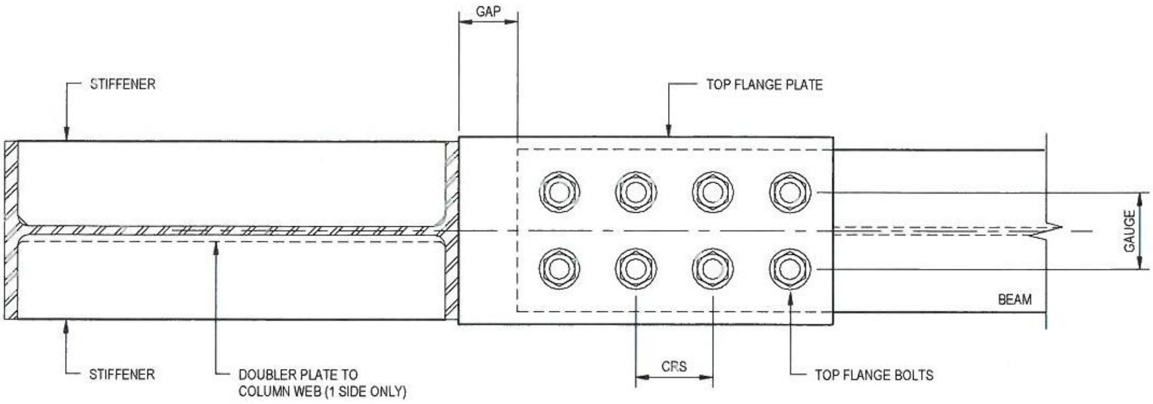

Fig 17 Sliding Hinge – Top Flange Section.

Recovering SHJ connections after ULS earthquakes

According to Clifton (2005a) and Khoo et al (2012b), the performance of the SHJ in major seismic events is exceptional. Providing the correct shims and detailing hierarchy is adopted the only the following recovery actions could be required;

Table 1. Expected SHJ Rotation and Recovery Actions

Seismicity	SHJ Rotation	Recovery Actions
After design level event (ULS)	>exceeds 0.012 radians	Retighten bolts
Beyond ULS Event	up to 0.03 radians,	Replace bolts/ retighten/slab repairs around column

Further Evolution of the SHJ system and further areas of research

Further research has recently been conducted looking at further improvements of the sliding hinge joint. We understand the findings will be reported in upcoming journals and conferences. In particular,

- Khoo (2013a), on self-centering SHJs with Ringfeder friction springs suggests that with limited additional prestress, a fully self-centering SHJ can be achieved
- Khoo et al. (2013b), upcoming report on the behaviour of the top flange plate
- Yeung et al. (2013), upcoming report on the bolt model refinement. Further testing and review of the testing results from UOA will confirm the bolt model which can then be widely adopted by engineers and hopefully adopted by NZS3404.
- Further and ongoing research and reports on the durability performance can be expected and is likely being reported upon during this conference.

Conclusions

This paper presented recent experiences in the application of Sliding Hinge Joint Technology in multi store steel framed buildings with the particular application in Steel moment resisting frames. Aurecon have applied the SHJ technology in both beam column joint and column base applications to provide a semi rigid moment connection.The sliding hinge joint presents a formidable connection and advanced semi rigid connection that enables designers of steel moment frames significant benefits compared to traditional bolted moment end plate connection.

In the application in numerous high rise buildings, the cost to implement the sliding hinge joint in steel moment resisting frames was found comparable, or more cost effective than traditional ductile connection, with the added advantage that SHJ's enable the structure to be recovered post a design level event.

We have summarised the design procedures harnessed, tools used to design and detail the connection. We have also outlined key lessons learnt in the application of the technology by highlighting advantages and disadvantages, how to recover the buildings after design level events and identified current research into further evolutions of the technology.

References

Clifton, G.C., 2001. *Seismic Design Procedures for steel structures*. Heavy Engineering Research Association Report R4-76, Auckland, New Zealand.

Clifton G.C., 2002. *The Sliding Hinge Joint*. Design and Construction Bulletin (DCB) 68 June/July 2002, Heavy Engineering Research Association, Auckland, New Zealand.

Clifton G.C., 2005a. *Semi-rigid joints for moment-resisting steel framed seismic-resisting systems*. PhD thesis, Auckland, New Zealand: University of Auckland.

Clifton, G.C., 2005b. *Semi Rigid Joints for Moment Resisting Steel Framed Seismic-Resisting Systems*. Heavy Engineering Research Association Report R4-134, Auckland, New Zealand.

Clifton G.C. 2007a. *Sliding Hinge Joint: revisions to the design procedure*. Heavy Engineering Research Association, Auckland, New Zealand; 2007.

Clifton, G.C, G.A. MacRae, H. Mackiven, S. Pampanin, J. Butterworth, 2007b. *Sliding Hinge Joints and Subassemblies for Steel Moment Frames*. NZSEE Conference. Paper 19.

Clifton G.C., M. Bruneau, G.A. MacRae, R.T. Leon, A. Fussell, 2011. *Steel structures damage from the Christchurch earthquake series of 2010 and 2011*. NZSEE Bulletin 2011;44(4):297-318.

Gledhill S.M., G.K. Sidwell, D.K. Bell, 2008. *The damage avoidance design of tall steel frame buildings — Fairlie Terrace Student Accommodation Project, Victoria University of Wellington*. NZSEE Conference, Taupo, New Zealand; April 11–13; 2008.

Khoo H.H., G.C. Clifton, J.W. Butterworth, G.A. MacRae, G. Ferguson, 2012a. *Influence of steel shim hardness on the Sliding Hinge Joint performance*. J Constr Steel Res; 72:119-29.

Khoo H.H., G.C. Clifton, J.W. Butterworth, G.A. MacRae, 2012b. *Experimental studies of the self-centering Sliding Hinge Joint*. NZSEE Conference, Christchurch, New Zealand; April 13–15.

Khoo H.H., 2013a. *Development of the low-damage self-centering Sliding Hinge Joint*. PhD thesis, University of Auckland.

Khoo H.H, C. Seal, G.C. Clifton, J. Butterworth, G.A. MacRae, 2013b. *Behaviour of the bottom and top flange plates in the Sliding Hinge Joint*. NZSEE Bulletin (in press).

MacRae G.A., G.C. Clifton, H. MacKinven, N. Mago, J. Butterworth, S. Pampanin. *The Sliding Hinge Joint moment connection*. NZSEE Bulletin 2010;43(3): 202-12.

Standards New Zealand, 1997. *NZS 3404: Steel Structures Standard*. Wellington, New Zealand:

Standards New Zealand, 2002. *NZS 1170.1:2002 — Structural Design Actions*. Wellington, New Zealand.

Standards New Zealand, 2004. *NZS 1170.5:2004 — Structural Design Actions Earthquake Actions*. Wellington, New Zealand.

Yeung S., H. Zhou, H.H. Khoo, G.C. Clifton, G.A. MacRae. 2013. *Sliding shear capacities of the Asymmetric Friction Connection*. NZSEE Conference (under review).