# The Damage Avoidance Design of steel frame buildings - Fairlie Terrace Student Accommodation Project, Victoria University of Wellington.

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**ABSTRACT:**

The design philosophy implicit in Seismic Design Standards is that structural damage is acceptable during design level earthquakes, provided this damage is confined to suitable locations. It is expected that these locations will sustain damage without a significant reduction in strength.

Designing a structure with conventional ductility has the advantage of reducing initial construction cost whilst ensuring acceptable structural performance for 'life safety', ULS event. However, this approach means acceptance of the disadvantages of remediation cost and disruption caused by damage which in seismic events that exceed the serviceability design, SLS event.

Recent advances in Seismic Engineering have focused on a "Damage Avoidance Design" philosophy, whereby a structure is designed to withstand a design seismic event with minimal and easily repairable damage. This typically involves incorporating mechanisms in the structure that can control loads and sustain large deformation without causing damage. In these systems ductility is provided by added components that remain undamaged, or can easily be replaced.

Connell Wagner has recently undertaken several multi-storey building projects which have incorporated innovative damage avoidance systems. These systems include 'sliding hinge joint' moment frames and 'braced frames anchored with pre-stressed friction dampers'. The systems were developed by Connell Wagner utilising research conducted by HERA and the Universities of Auckland and Canterbury. The systems were implemented in these projects for minimal differential costs.

This paper provides a review of a range of suitable and available damage avoidance options solutions for this project and their application. The paper outlines the concepts and application of the damage avoidance systems implemented by Connell Wagner. Details of the methods of analysis used in their verification are not given.

# 1 INTRODUCTION

## 1.1 Planning for Earthquakes – Damage Avoidance Design Philosophy

Building developers consider structural engineers to always present the latest research developments in their designs, whilst complying with all current design standards. In New Zealand, a structural solution is always expected to be economic. This expectation unnecessarily limits consideration of damage avoidance design due to the cost perception of the available technology.

Modern structures are more rigorously designed, and in the more seismically active areas of New Zealand, to higher levels of lateral acceleration than were required by previous loading standards. However compliance with strength and displacement requirements of our standards may not necessarily address or limit control of ‘damage’.

Modern designers often use appropriately high levels of ductility. Well detailed high ductility structures will undoubtedly perform well in terms of their ability to survive a big event and dissipate seismic energy via predetermined hinge zones. Although this damage may not lead to instability or failure, structural damage resulting from significant earthquakes in buildings of high ductility will likely require repair and in severe cases may leave properties untenable.

Client perception and expectation may be that the ‘modern’ building will be useable after a big seismic event. Is this really the case? Is control and limitation of structural damage adequately considered in the traditional design process? Seismic damage after the Northridge earthquake in California created significant damage to some modern structures, which suggests damage is often not adequately addressed.

It has been a well documented recent trend from research institutions to consider incorporating a ‘damage avoidance design’ philosophy in the structural design process. Numerous leading academics have been utilising both concrete and steel structures. While numerous presentations of concepts and test buildings have been presented to consulting engineers, a common market perception is still that some of these solutions are neither elegant nor cost effective to implement. To date few tall steel structures in New Zealand have been designed and detailed with a damage avoidance philosophy.

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.

Structural damage needs to be planned for in design, controlled and limited, to ensure a building can remain useable after a large earthquake. A solution for controlling seismic damage in taller steel framed buildings has been developed. It is believed this technology means it is now economically viable to achieve this objective.

## 1.2 **Fairlie Terrace Student Accommodation Project for the Victoria University of Wellington**

Recent advances in Seismic Engineering have focused on a “Damage Avoidance Design” philosophy, whereby a structure is designed to withstand a major seismic event with minimal and repairable damage. This typically involves incorporating mechanisms in the structure that can control loads and sustain large deformations without causing damage.

At the request of Victoria University of Wellington, Connell Wagner was tasked to undertake the design of the new student accommodation buildings at 74-87 Fairlie Terrace, to incorporate a “Damage Avoidance” philosophy. The purpose of the damage avoidance design of this project was to ensure that after a large earthquake, the student accommodation buildings could be utilised for administration or as a mixed use facility, while other university buildings were under repair, ensuring the University can continue its operations.

The buildings are situated on the Universities Kelburn Campus on an elevated site overlooking Wellington City’s CBD. The development is located approximately one kilometre from the active Wellington Faultline. Under NZS1170.5 the site subsoil was assessed as “Class B –rock”. The site geology consisted of highly weathered greywacke rock overlaid with a softer soil lens. It’s expected that the site will generate lower levels of seismic acceleration than other ‘lower’ CBD sites that are founded on reclaimed harbour which vary from shallow to deep soil sites.

The format of the building is compromised of three accommodation buildings of five, ten and eleven storeys. Due to the very steep nature of the site, project wide floor levels were assigned. At the top of the site ‘The Terrace’ building provided street level access. The common or circulation level consisted of a single storey Administration Building and concourse link with a 14m span Bridge structure linking the ‘Tower’ and ‘Edge’ Buildings.

The proposed buildings extend up to 11 storeys or 37m to roof level. To suit student accommodation, the buildings were narrow at 12m and relatively long at up to 55m. It was determined that the most economic structure would consist of a structural steel frame and lightweight façade, coupled with short span prestressed concrete floors. This would assist in minimising the seismic mass.

To compliment the architecture a seismic resisting scheme was developed that included perimeter longitudinal moment resisting frames with transverse bracing frames reducing diaphragm spans and controlling torsion at the perimeter. The buildings had estimated natural periods in the range of 1-1.6sec depending on direction considered.

The challenge to create damage avoidance design features was complicated by the proposed building form. Typically short, stiff heavy buildings with low periods are suited to base isolation, but few options are available for tall, relatively light, flexible steel buildings. Damage avoidance features available on the market are often viewed as expensive and complicated, and have not been widely utilised.

## 1.3 **Review of damage avoidance options**

We undertook in-depth analysis of suitable technologies for the project; considering base isolation, viscous damper systems and parallel in plane friction control systems.

The Fairlie Terrace Student Accommodation project had numerous key drivers, but speed of construction and economy of the system chosen were paramount to the selection of the bracing system and its damage avoidance features.

# 2 BACKGROUND RESEARCH

## 2.1 Damage Avoidance Systems and Options Considered

This section outlines a summary of some of the damage avoidance design options considered for this project with comparative strengths and weaknesses in this projects application.

### 2.1.1 *Base Isolation*

The most widely known form of seismic mitigation system is base isolation. The most common forms are lead rubber bearings and sliding bearings which are supplemented by an elastic or plastic mechanism to control displacement and provide a restoring force.

Base isolation techniques are well established and have been applied in New Zealand and around the world. Base isolation is generally adopted to limit forces and accelerations in super structures of buildings. Technically, base isolation is suited to stiff, heavy, low-rise structures. Its principal application is therefore for historic buildings or for important buildings, such as hospitals, where a large level of protection is provided to ensure operation even in the event of large earthquakes.

Recent innovations in base isolation technology, such as post tensioned lead rubber bearings, or the Robinson Roglider, have extended the application to a wider range of building types. However base isolation devices generally are not the most effective solution for lighter high rise structures.

Procurement, design and testing of bearings within the short design and documentation period were also perceived as issues.

### 2.1.2 *Ductile Steel Bracing Frames*

Braced steel frames are often considered the most cost-effective solution for the lateral restraint of a steel building. The conventional means for providing seismic protection is through the inclusion of a ductile link member; eccentrically braced frames (EBF). The link is typically the beam section between diagonal braces.

However in a design earthquake event, the link member is likely to undergo large plastic deformations, resulting in significant local damage in the adjacent structure, and there is likely to be some permanent building and/or element deformation. Post event repair of the damaged link element would generally be difficult.

The performance of braced frames can be improved through the addition of more sophisticated link mechanisms as outlined on fig 2.1 below which shows two separate systems on the same sketch. These features may increase damping, decrease plastic deformation, or be more readily replaceable if damaged.

Available ductile link options considered included;

- Lead Rubber bearing damper; produced by Robinson Seismic, NZ

- Sliding Hinge Joint; proprietary items available overseas, or project specific fabricated items may be used (utilising technology developed by HERA and University of Auckland)
- Hysteretic Axial Damper (Yielding link between the brace ends); proprietary items available overseas, or project specific fabricated items may be used
- Hysteretic Flexural or Shear Damper (Yielding link between brace and floor beams); proprietary items available overseas, or project specific fabricated items may be used.

This solution would provide significant enhancement of the buildings performance in the transverse direction of the middle ten storey "Tower" building and the Eastern eleven storey "Edge" building. In the longitudinal direction, this solution would need to be coupled with another system.

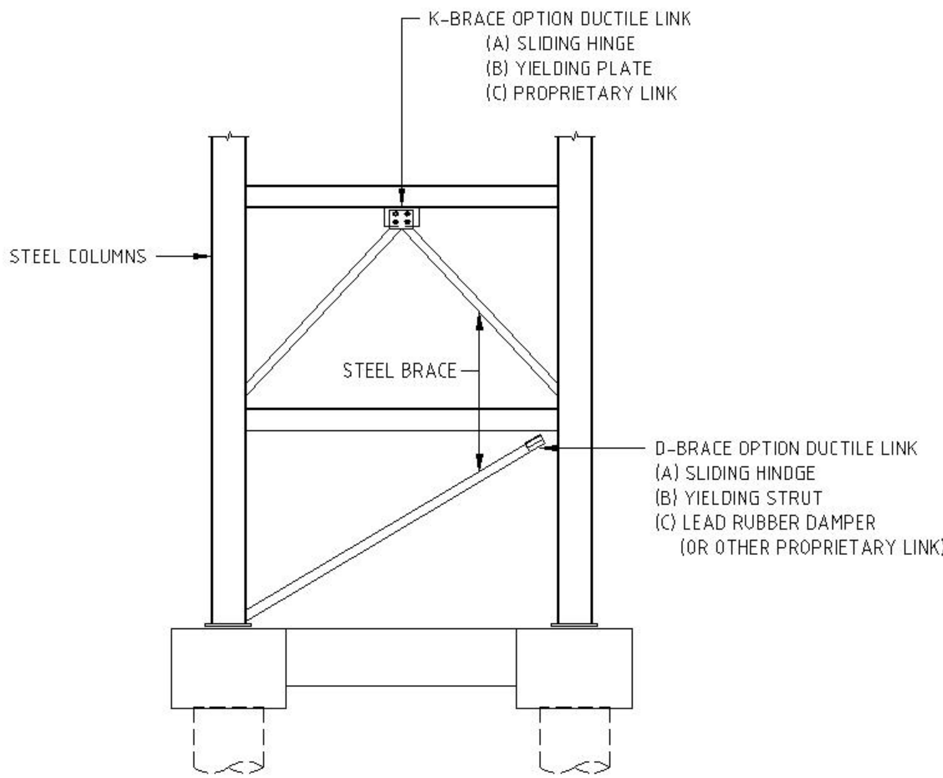



Fig 2.1 Different Options for a Ductile Steel Braced Frames

### 2.1.3 *Viscous Damped Steel Frames (VDSF's)*

Displacements and hence damage of flexible frame structures may be reduced through the addition of viscous dampers. These typically take the form of viscous dampers in diagonal brace elements.

Viscous damper braces differ from conventional braces in that they do not significantly add to the elastic stiffness of the system, and damping forces are out of phase with the elastic frame forces. Viscous dampers are therefore an effective means of controlling damping in a frame systems without increasing base force. Viscous dampers would typically be proprietary items produced overseas.

Viscous dampers are therefore an effective but possibly costly seismic mitigation option for high rise buildings.

In this project, viscous dampers are **only** suitable in the long direction between or enhancing moment resisting steel frames. Viscous dampers, although offering good performance would require architectural consideration due to the impact on window zones, as per Fig 2.2.

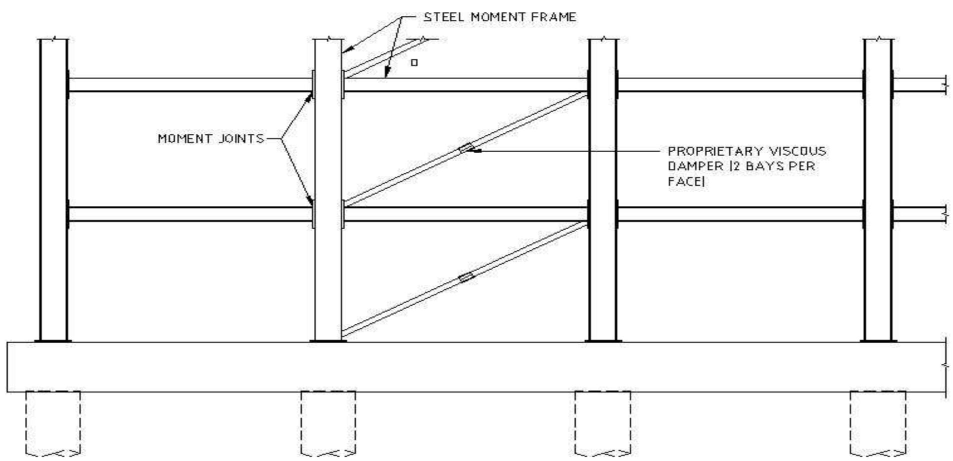



Fig 2.2 Viscous Damped Steel Frames (long Direction only).

### 2.1.4 *Prestressed Steel Braced Frames*

The prestressed braced frame is an innovative solution to converting what is normally a non-ductile connection into a location at which ductility can occur. This is in the hold down connection between the column and their foundation.

Using an unbonded prestressing cables this connection can be made to act in a similar way to the cables utilised in concrete wall design utilised in PRESS technology applications.

A major benefit of this approach is that the tensions in the cables are restricted to ensure that they remain elastic, the long unbonded length achieves this. Since they remain elastic the cables return the structure to vertical after the earthquake has occurred. Other conventional ductile solutions result in permanent deformations after an earthquake.

An extension of this is to also utilise damping devices at the column bases so as to absorb some of the energy that would be otherwise stored in the uplifted column and stretched prestressing cables. A localised prestressed Ringfeder spring is a very suitable extension of this concept.

These solutions may be located in the transverse short direction of the Tower and the Edge Building.

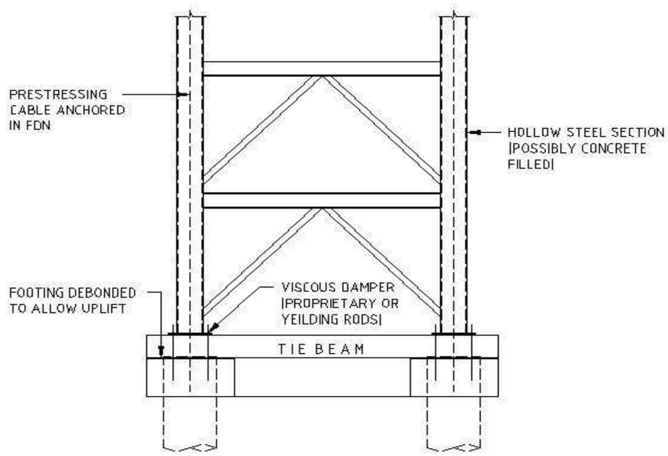



**Fig 2.3 Prestressed Steel Braced Frame**

### 2.1.5 *Sliding Hinge Joints (SHJ's).*

This solution was elected as a suitable damage avoidance design feature for moment resisting steel frames (MRSF's).

The performance of steel moment resisting frames may be improved through the use of flange bolted sliding hinge joint connections between beam and columns sections. Sliding hinge 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. Sliding hinge joints also have the advantage of de-coupling stiffness and strength, allowing the designer to control seismic drifts without introducing larger forces in columns and foundations.

A sliding hinge joint has been developed in New Zealand by HERA and the University of Auckland. The sliding hinge joint is formed by adding shims and slotted holes to conventional flange plate connections. Performance may be enhanced though the addition of Belleville springs, which limit bolt deformation and the need for post event repair.

The use of sliding hinge joints can be 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 overstrengths being weaker than the sum of beam overstrengths.

In the short or transverse direction, braced steel frames as outlined above in Figures 2.2 and 2.3 would be more applicable.

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

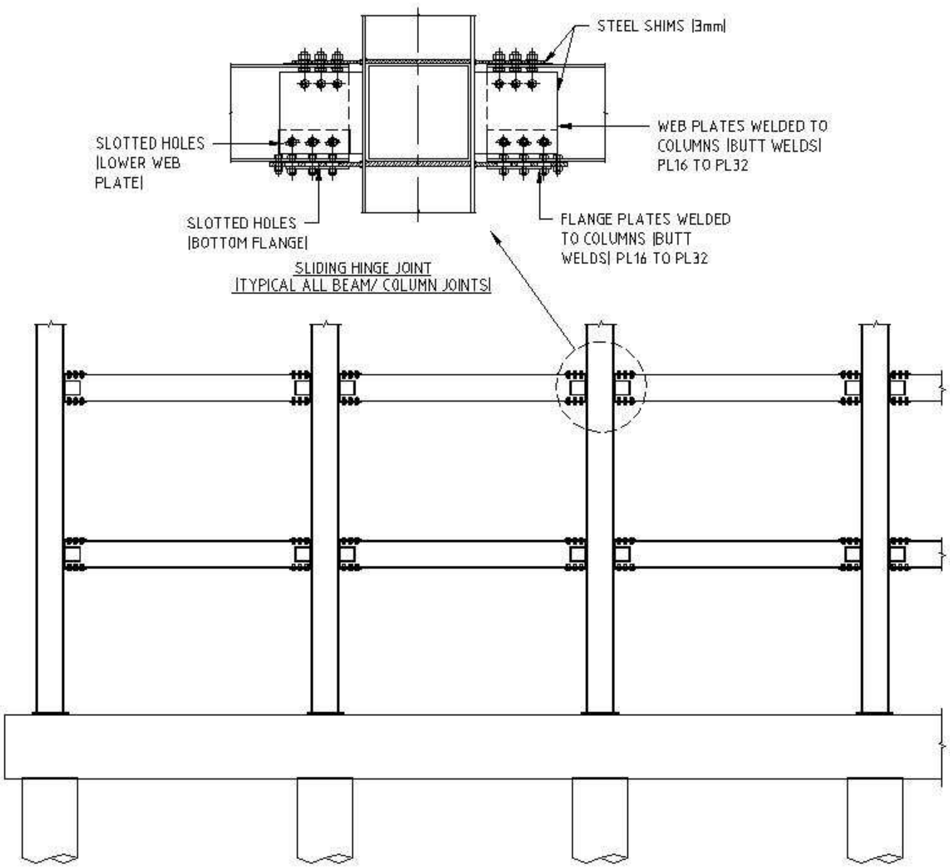



**Fig 2.4 Sliding Hinge Joint at a Beam Column Joint.**

# 3 CONCEPT CHOSEN & DEVELOPMENT OF DAMAGE AVOIDANCE DESIGN

## 3.1 Chosen Concept

To meet the challenge Connell Wagner developed and has refined a new system for damage system applicable to high-rise steel buildings. The system utilised research conducted by both HERA and the University of Auckland.

The Damage Avoidance system adopted for this project featured coupled concentrically braced frames with prestressed Ringfeeder Springs and sliding hinge joints between columns and foundation. The system also incorporates steel beams with Sliding Hinge Joints which were refined in conjunction with HERA.

The system provides a “designed hinge” at the base of the building to enable reduction of foundation and column forces, displacement control and limit of damage to easily repairable small local items and hence addressees a host of questions about sustainable seismic design.

The short or transverse frames consisted of CBF steel frames, coupled at the internal column with sliding hinge jointed beams. The longitudinal bracing was provided with steel moment resisting frames with sliding hinge joints at beam and column joints and also with vertically orientated sliding hinge joints to provide column base protection.

### 3.2 **Transverse Bracing - Tension Limited Rocking Steel Shear Walls - Basic Concepts**

At the Victoria University of Wellington Student accommodation project, the building's transverse bracing frames consisted of coupled concentrically braced steel frames (CBF's), with a tension limiting base level hinge. The base hinge consists of prestressed Ringfeder friction springs and vertically orientated hinge joint friction plates. This system allows formation of a tension limited, protected hinge at the base of the CBF that controls deformation and limits damage. The coupling beams have sliding hinge joints at each end.

The Ringfeder springs enable the connection to be preloaded so as to set the performance criteria at which uplift can commence. The sliding bolts provide an initial resistance to uplift and also provide a means of reducing the impacts that may occur when the reverse cycle of loading closes the gap. The coupling beams stiffen the system to control drifts and provide additional energy dissipation.

Lateral seismic and wind forces are distributed via rigid concrete diaphragms to the transverse bracing frames.

The coupled CBF's behave in a similar fashion to a coupled shear wall, but with a tension limited foundation connection. As in all walls, lateral forces are resisted via overturning consisting of tension and compression edge zones with a diagonal strut and horizontal shear forces acting throughout the section. In a concrete shear wall plastic hinge formation often relates to concrete spalling and potential buckling of vertical reinforcing at the wall edge zones.

The CBF is effectively a 'steel shear wall' utilising external columns, a horizontal shear 'collector' beam at underside of the floor slab and a strut/tie diagonal brace. The lateral loads are then braced via the diagonal strut/tie and collector beam down the building to a piled foundation. The base shear is resisted via contact bearing whilst the vertical loads are controlled by the hinge mechanism. The lateral loads are resisted by a number of fixed-head bored concrete piles.

Under seismic lateral loads the CBF's are designed to uplift at tension column. At the internal column, uplift is limited by coupling beams, hence less spring compression force (to resist column tension) is required resulting in less friction spring elements. Uplift occurs after the coupling beam overstrengths, gravity and the sliding hinge resistance is overcome.

At the external columns, for the column to rock upward, net actions of the diagonal brace and column must overcome the initial Ringfeder prestress, the friction from a vertically orientated column flange mounted sliding joint and the gravity load in the column. Once friction is overcome the upward rocking motion of the frame compresses the Ringfeder springs between a thick cover plate and an upper baseplate.

The elegance of the solution is controlling net tension at foundation level; ensuring hinge formation occurs under a stable rocking mechanism that dissipates seismic energy.

### 3.3 **Combined Ringfeder and SHJ - Hinge System**

The Ringfeder springs are prestressed (compressed) with a central turned down bolt through a baseplate connected to the foundation and pile assembly. The hinge works via a hierarchy of devices. Initially once the seismic column axial tension forces overcome gravity loads, the Ringfeder prestress provides a further uplift threshold. Once prestress is exceeded the sliding hinge joint friction plates are engaged and once this friction is overcome, the column uplifts and additional Ringfeder spring compression begins.

The prime design consideration for the Ringfeder system was the force and spring travel relationship. A significant advantage is that Ringfeder friction springs provide a much higher weight of spring to work ratio than other forms of non viscous steel dampers, meaning excellent damping performance for their size, travel and cost.

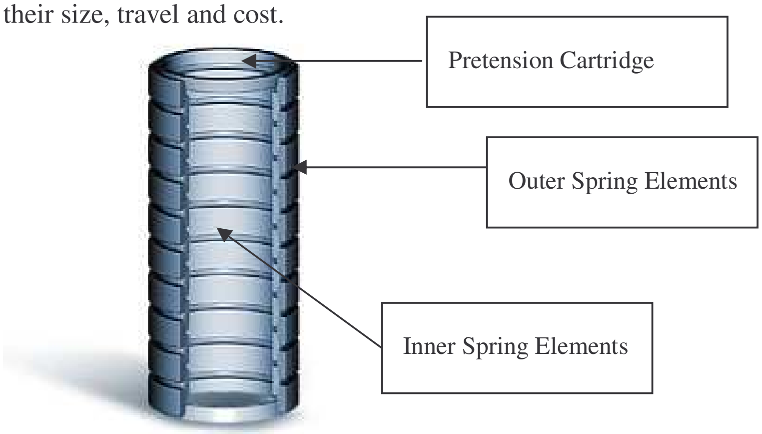



Fig 3.1 Ringfeder Friction Spring Elements

The Ringfeder friction springs consist of an inner and outer concentric spring steel elements, fully utilising the spring in a force balancing application. As the spring is compressed the outer spring element has tensions induced in it and the inner compressions. The assembled spring cannot resist tensions. Each pair of elements is supported by one another, with a gap formed between inner and outer rings. The contact surface is a steep, inwardly inclined surface, which during compression forces allows the rings to move inward closing the gap and decreasing the height of the stack. Essentially the spring loads up in compression travelling linearly with load and displacement until either unloading or later; full lockup.  Upon seismic load reversal the spring is unloaded, but returns with a non linear curve. Ringfeder provide a force spring travel curve for each element type.

If spring travel is reached or exceeded further protection tension limiting devices are provided. Upon 'travel exceedence' the Ringfeder spring effectively reaches 'lockup'. When this occurs, the spring stack becomes almost a solid steel stack. The turned down prestressed bolt is designed to begin yielding at approximately 90% of spring lockup.

Horizontal shear takeout is provided via a steel collector beam, and another steel frame connecting the pile heads and concrete foundation frames. Refer to Fig 4.6 below. Two PFC side plates are provided to prevent the brace and column displacing sideways. The load/displacement behaviour of the Ringfeder springs follows an initial prestress, linear load up (compression). This stage is followed by vertical unloading to approximately a third of the peak force. The unloading curve does not return to zero, but essentially changes slope to reflect the loadup curve, resulting in a residual prestress or system slack.

The sketch indicates a shaded area reflecting the energy dissipated or damping achieved by the Ringfeder friction spring during this load/unload cycle. The prestressed Ringfeder hysteresis loop for a single column joint (not the entire frame), is as outlined on Fig 3.2.

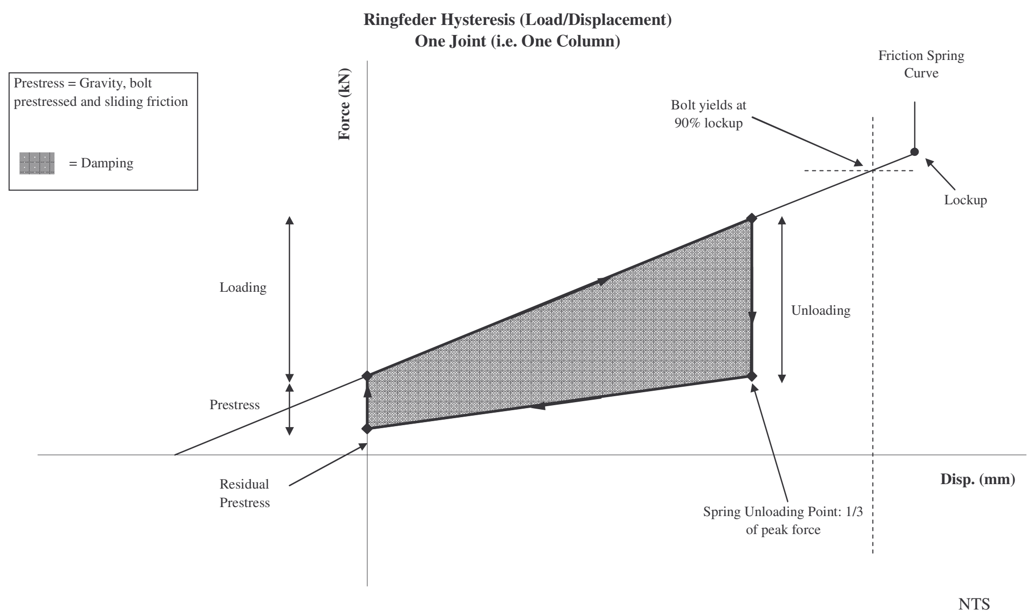



Fig 3.2 Ringfeder Hysteresis Curve

The Friction plate hysteresis curve shows one column or joint only, not the entire frame performance. The ‘pinched’ shaped curve indicates an assumed upper bound load displacement curve. The upper or initial loadup curve reflects the higher initial system strength expected during smaller seismic events. The larger events generate a similar peak force but generate significantly more displacement. The return cycle tends negative indicating the system to some extent tends to ‘hold the column up’. Subsequent load-up curves achieve a lower first yield. This creates the ‘pinched’ hysteresis. Once the bolt rotation ‘turns over’ the strength ramps up again and a similar same peak force/displacement is achieved during the first cycle. Some slack is predicted on subsequent reloading cycles, up to 10% of the first cycle, perhaps due to the bolt and plate elongation. Upon unloading the joint settles vertically as a result of the gravity axial load. The sliding hinge joint hysteresis of the vertically orientated friction plate is indicated in Fig 3.3.

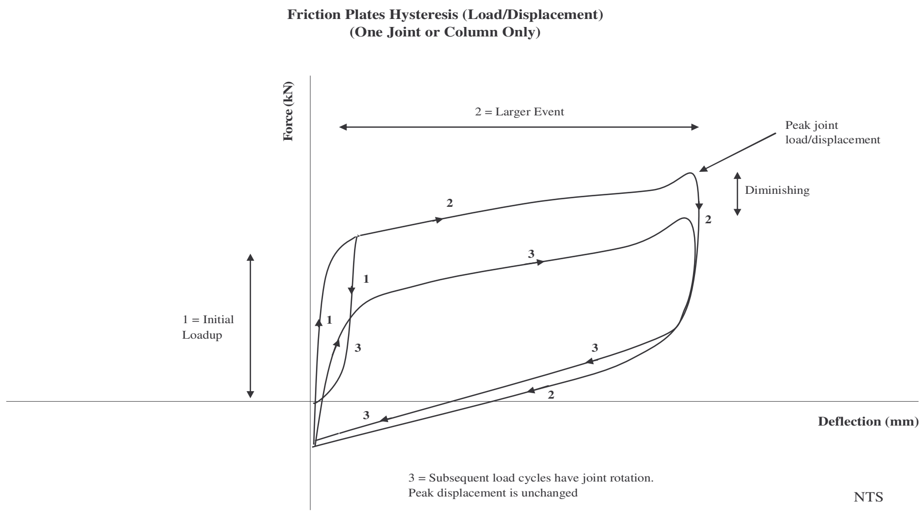



Fig 3.3 Friction Plates Hysteresis

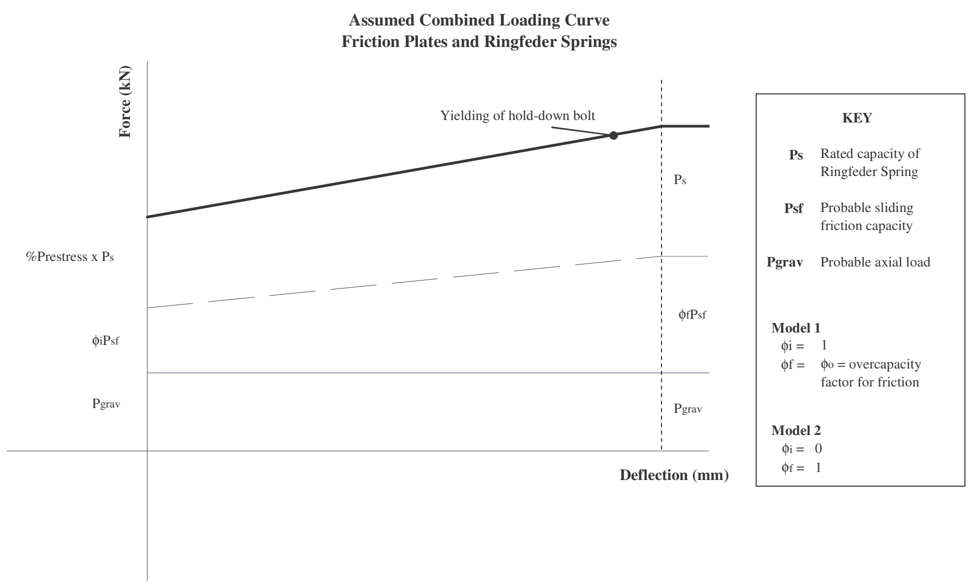



Fig 3.4 Assumed Loading Curves for the Combined Ringfeder/Friction Plate Hinge

Essentially as outlined above, as tension loads increase on the hinge follows this sequence.

a). Gravity Load is overcome

b). Friction plates – bolt/plate friction is mobilised and overcome

c). Ringfeder Prestress is overcome

At this stage the column baseplate can uplift from the foundation, further compressing the Ringfeder spring.

At the limits of the Ringfeder travel the total tension resistance is the gravity load, the Ringfeder Rating (F) plus the final capacity of the friction plates. This concept was used to calculate upper and lower bound hinge yield and ultimate stiffness for structural modelling.

### 3.4 **Longitudinal Frame – MRSF with Sliding Hinge Beam Column Joints**

At the Victoria University of Wellington Student accommodation project, the moment resisting frames have Sliding hinge joints connecting the beams to columns. CBF coupling beam between frames have been utilised, to confine any plastic distortion to the bolts associated with the bottom flange connection. This makes for a readily repairable connection should a major earthquake occur.

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.

The sliding hinge joints were designed in general accordance with the recommendations of the HERA bulletin no 68 and subsequent recommendations from HERA, based on the substitution of brass for steel shims and updating bolt capacities.

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 cantilever column for the beam bottom flange and web side plate are slotted, with capping plates over.

The philosophy of the joint is to ensure performance characteristics are achieved for both the Design Based Earthquake (DBE or ULS event) and the Maximum Credible Event (MCE). The joint is suitable for moderate 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 will suffer joint damage. Non linear time history analysis by HERA suggests that little or no joint reinstatement would be needed after the MCE.

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.

A key feature of the application of the SHJ is the utilisation of steel rather than brass shims proposed under the HERA method. This is critical to the economy, practicality and availability of the joint.

### 3.5 **Longitudinal Bracing – MRSF with Column Base Hinge Protection – Concepts**

The column bases of the MRSF frames 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 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 employ vertical friction plates, a slotted plate arrangement for sliding and base shear key plate.

# 4 DEVELOPMENT & DETAILING AND APPLICATION

## 4.1 *System Detailing – CBF Frames with Pretensioned RingFeder Springs*

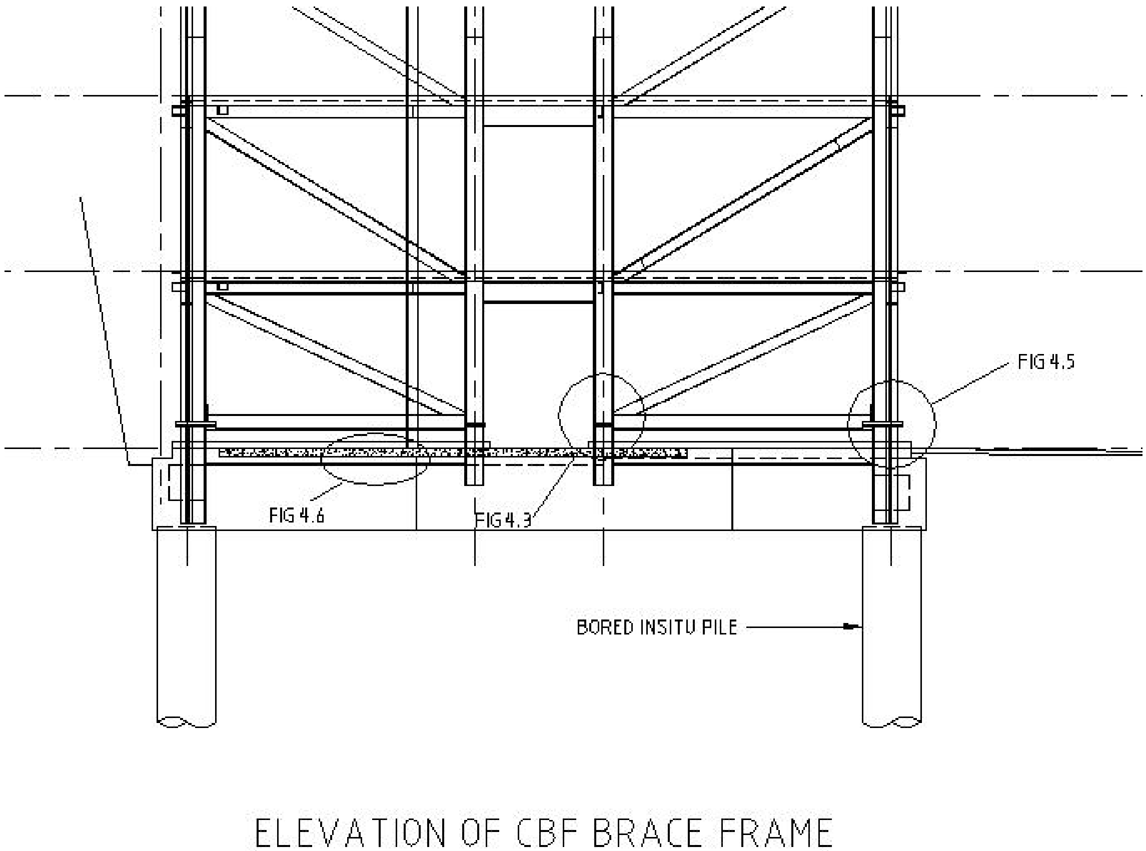
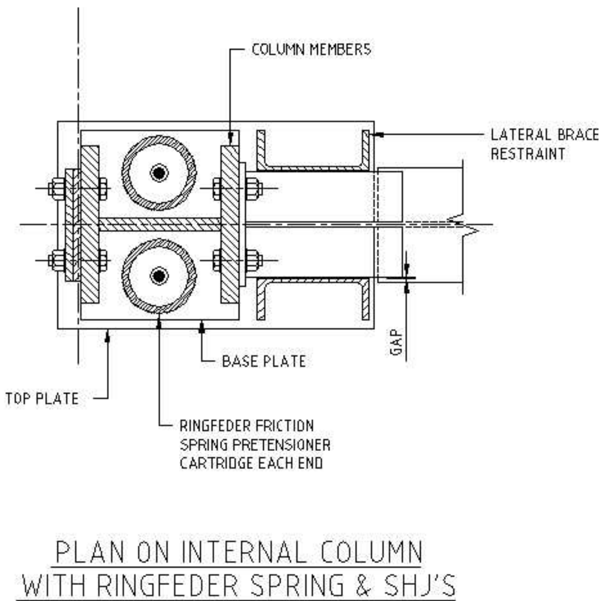



Fig 4.1 External Frame Elevation from the East/West Transverse Bracing Frame

This figure illustrates an elevation of a coupled concentric bracing frame acting as a rocking steel shear wall. The Ringfeder friction springs are located under all four columns.

The two CBF frames are connected with central coupling beams equipped with sliding hinge joints.

Lateral loads are distributed to fixed head piles via large shear key plates under the ground floor collector beam.



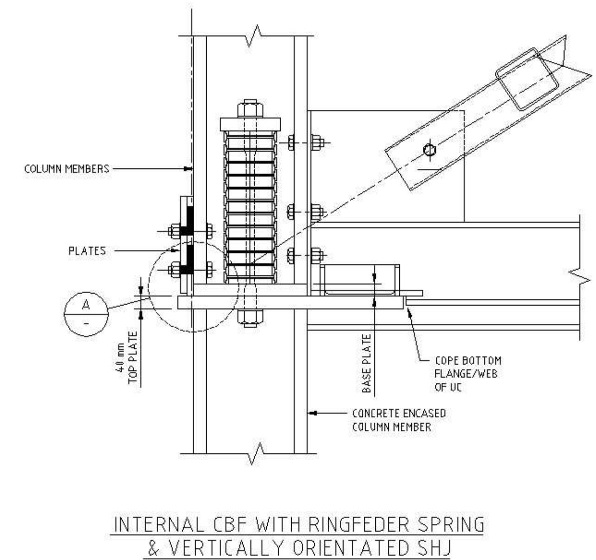




Fig 4.2 and Fig 4.3 Plan and Elevations of an Internal CBF Column Frame Arrangement

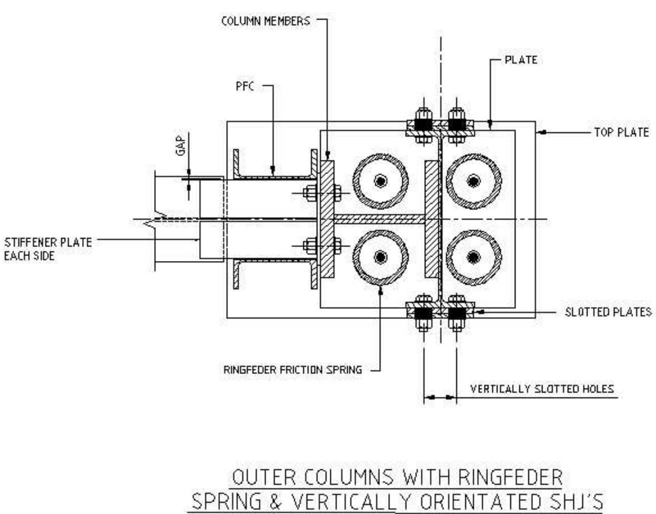



Fig 4.4 Plan on External CBF and MRSF Dual Column

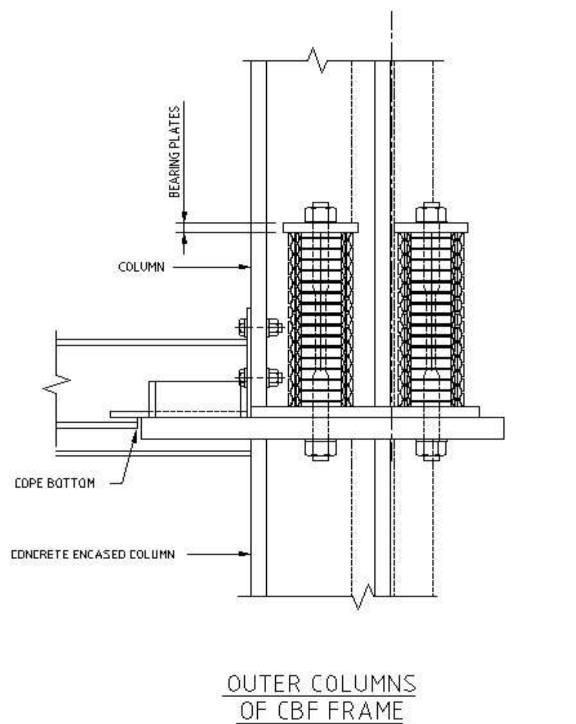



Fig 4.5 Elevation of External CBF and MRF Dual Column

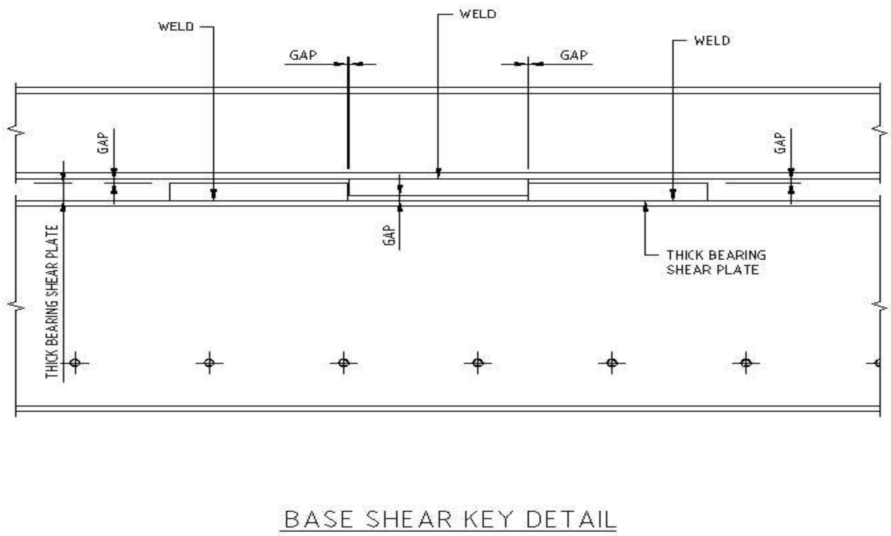



Fig 4.6 CBF Base Shear Transfer Mechanism – Contact Plate Bearing

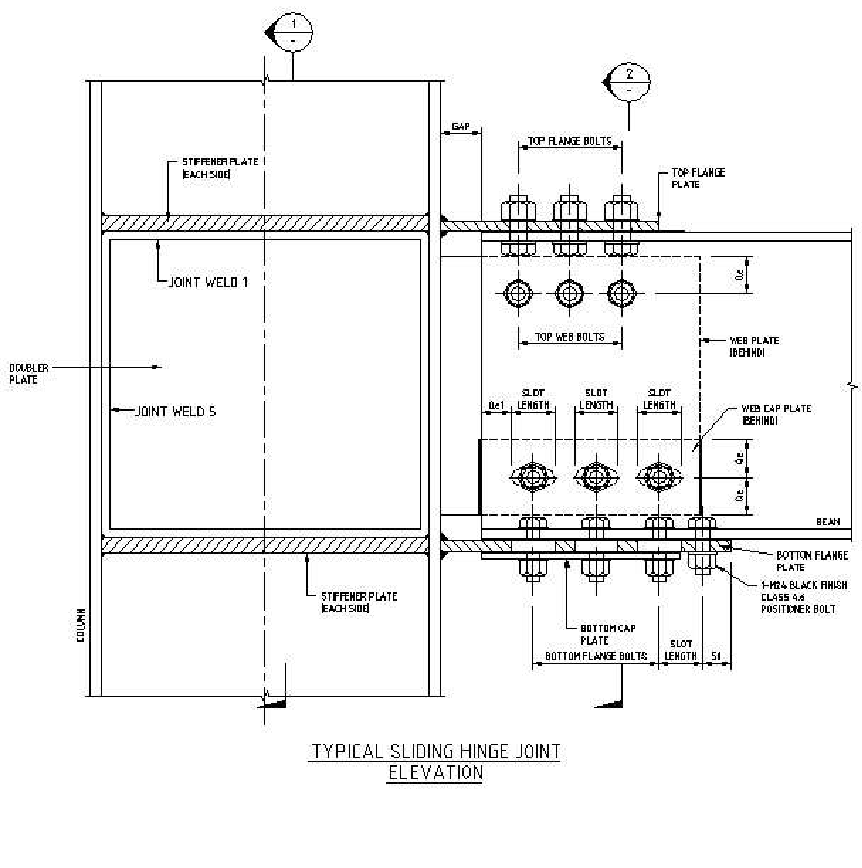



Fig 4.7 MRSF Beam Column Joint - Sliding Hinge Arrangement

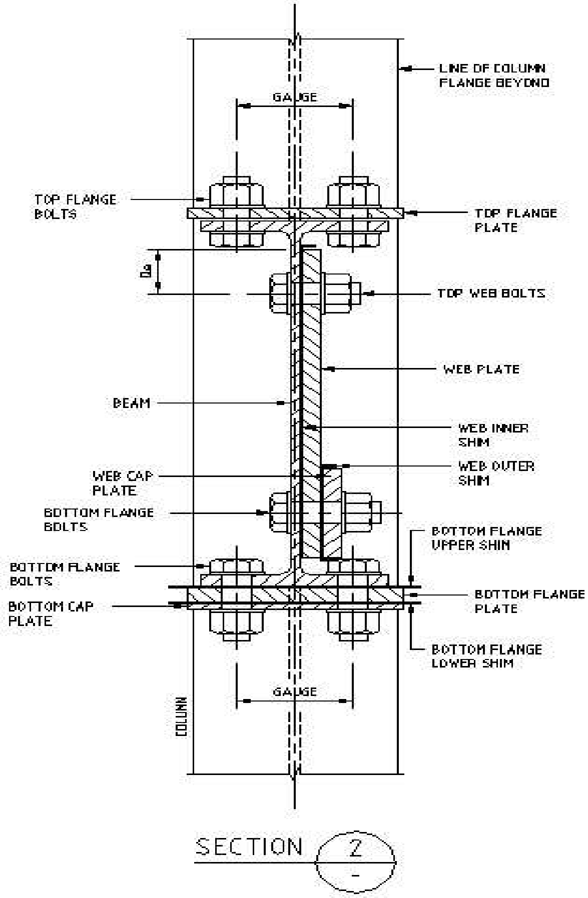



Fig 4.8 Section 1 & 2 – Column and Beam Detailing at SHJ Section

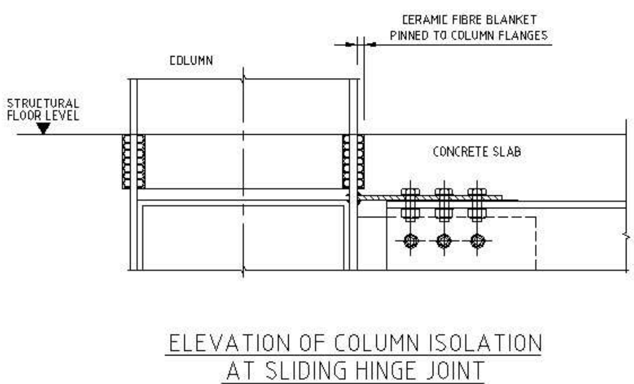



Fig 4.9 Sliding Hinge Joint Column and Slab Isolation Details.

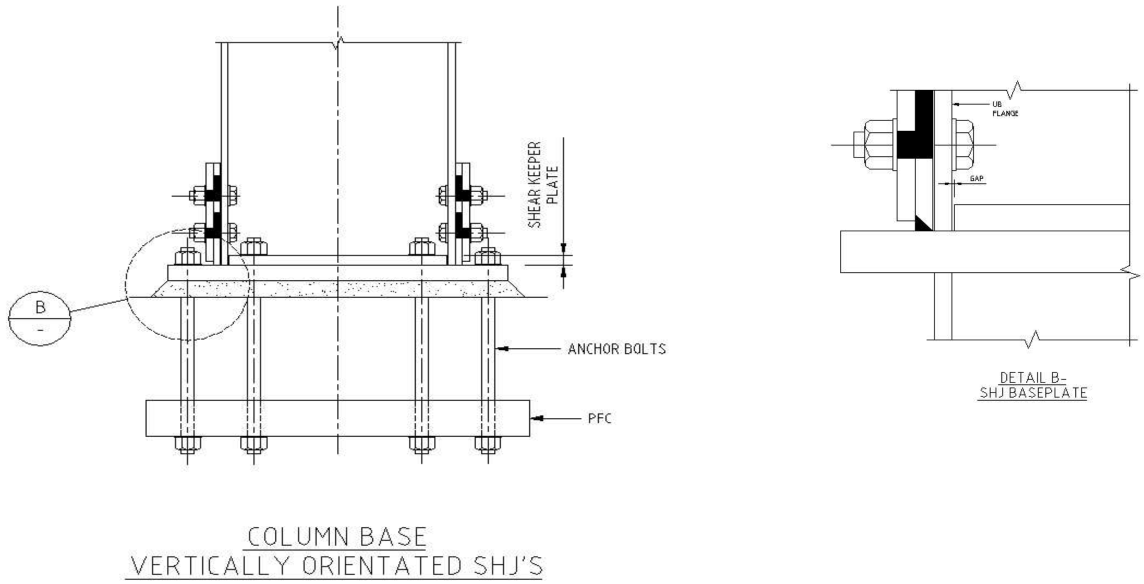



Fig 4.10 Vertical Orientated SHJ for Column Hinge Protection in MRF's and Detail B Cantilever Flange SHJ Friction Plates

## REFERENCES:

HERA Report R4-134. Clifton, G.C (June 2005). *Semi Rigid Joints for Moment Resisting Steel Framed Seismic-Resisting Systems.*

This PhD thesis describes the development of new semi rigid joints for moment resisting steel framed (MRSF) seismic-resisting systems. Four beam column joint systems investigated included: Ring Spring Joint (RSJ), Post tensioned Tendon Joint (PTJ), Flange Bolted Joint (FBJ), Sliding Hinge Joint (SHJ).

NZSEE 2007, Sliding Hinge Joints and Subassemblies for Steel Moment Frames, Paper 19

Clifton, G.C, G.A.MacRae, H.Mackiven, S.Pampanin, J.Butterworth.

RingFeder – Friction Springs Product Catalogue.