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DAMAGE-RESISTANT STEEL FRAMED SEISMIC-RESISTING SYSTEMS

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SUMMARY

For a building to resist a severe earthquake, it must exhibit dependable strength, stiffness and ductility. From the viewpoint of life safety, a building with relatively low strength and high ductility capacity may provide the same life safety protection as a building with higher strength and lower ductility capacity.

When subjected to a severe earthquake, however, the outcome in terms of structural damage sustained will be quite different. While both buildings will remain standing, the low strength/high ductility building is likely to require considerably greater structural repair than the high strength building. Such repair will at best require the building to be out of service for a period of time following the event, with associated occupant and business disruption.

However, with careful consideration, it is possible to develop building systems that achieve a high damage threshold without requiring design for elastic response. This can be achieved through the following:

- Use of semi-rigid joints that are rigid under normal operating conditions, rotate with dependable hysteretic characteristics under severe earthquake attack and become rigid again once the severe ground motion stops
- Use of isolation details to partially isolate each floor from the seismic-resisting systems at that level, thereby reducing the response of the floor to the lateral movement of the ground supporting the building.

To be attractive to owners and investors, such systems must add little cost to that of traditional seismic-resisting system construction.

HERA and the University of Auckland are involved in the development of moment-resisting steel framed (MRSF) seismic-resisting systems with these attributes. One system incorporates semi-rigid joints that are designed and detailed to withstand high rotation demand without structurally significant degradation. The second incorporates simpler semi-rigid joints between the beams and columns and a partial floor isolating detail, that is easy to fabricate and erect and reduces demand on the seismic-resisting system. This paper gives an overview of the systems developed and references sources of further information.

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1. INTRODUCTION

Concept of Project

This project has involved the development of new moment-resisting, steel framed seismic-resisting systems (MRSFs) with an emphasis on the following:

- The ability to withstand a design level severe earthquake with minimal or no damage
- Ease of fabrication and constructability compared with conventional rigid MRSFs
- Ease of design compared with conventional rigid MRSFs, especially in regard to meeting both strength and stiffness criteria for the seismic-resisting system
- Maintenance of a strong column, strength hierarchy in order to provide a high degree of protection against soft storey collapse

Developing the systems has involved the following sequence of operations:

(1) Establishment of a design philosophy and set of target performance requirements to be met for the connections and the overall structural system under severe seismic conditions.

(2) Development of potentially suitable connection details between the beams and columns, at the column base and for the isolation details between the floors and MRSF in the case of the partial isolation system. Once potentially suitable joint and system details have been formulated, the development process has involved:

(2.1) Ascertaining the likely modes of joint behaviour and performance under seismic conditions.
(2.2) Design of representative examples of the joint for experimental testing to determine the actual performance, the moment-rotation characteristics and the ductility capacity.
(2.3) Development of analytical models of the joint's moment-rotation characteristics for use in analytical modelling of MRSF systems incorporating this joint.
(2.4) Undertaking time-history analyses of representative systems to determine rotation demand on the joints and hence to confirm that the expected rotation demand is within the ductility capability of the joint, including meeting damage resistant criteria.
(2.5) Considering constructability and cost issues in order to show that the given joint is a viable option for a MRSF system.

This process is ongoing and iterative for each joint and system considered, involving several cycles of design/experimental testing/analysis/evaluation/design modification, etc.

Background to Project

The first work on this project started in late 1995, with the development of hypothetical moment rotation characteristics for the Ring Spring Joint(RSJ) (Fig 1) and the Post tensioned Tendon Joint (PTJ) (Fig 2), in order that analytical studies could be undertaken to determine the feasibility of the semi-rigid joint concept. This showed the concept of a weak joint/strong column system to be viable and beneficial and led to ongoing development of these joints. However, the experimental testing stage showed that neither of these two joints are viable options for the building superstructure. This lead to consideration of the Flange Bolted Joint (FBJ) (Fig 3) and the Sliding Hinge Joint (SHJ) (Fig 4).

As of mid 1999, the MRSF with FBJs had been shown to be a viable option, details of which are described in the 12WCEE paper by Clifton [1]. Since then, development of the FBJ and SHJ has been completed, details of which are to be given in Clifton [2]. The final stage has been to look at partial isolation of the floor systems, which is work in progress as of February 2004.

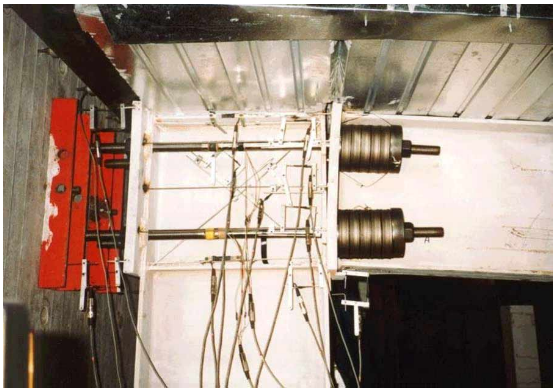

Fig 1 Ring Spring Joint (RSJ) Under Test

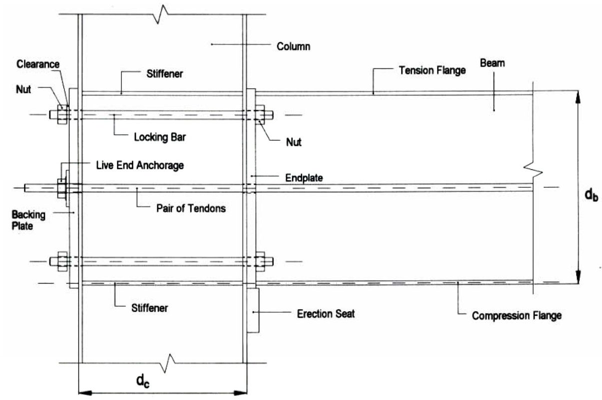

Fig 2 Post-Tensioned Joint (PTJ) in Unloaded Condition

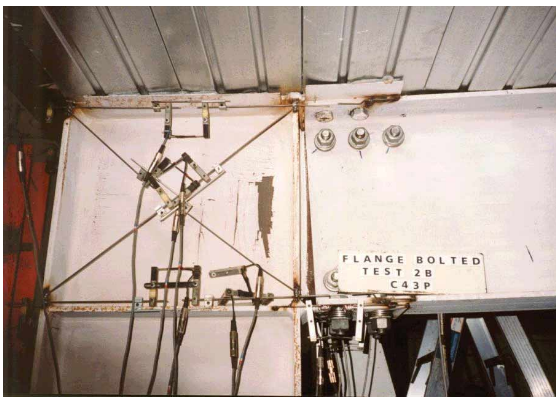

Fig 3 Flange Bolted Joint (FBJ) Under Test

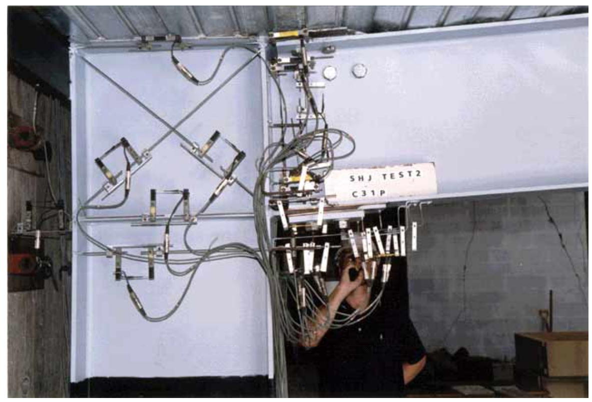

Fig 4 Sliding Hinge Joint (SHJ) Under Test

This paper gives an overview of the project, concentrating on the two potentially practicable damage resistant systems arising from the work, namely the MRSF with SHJs and the MRSF with FBJs and partially isolated floors.

2. DESIGN PHILOSOPHY AND TARGET PERFORMANCE REQUIREMENTS

Design Philosophy

The general design philosophy that has been developed and applied for these damage-resistant systems aims to establish dependable behavioural characteristics for the semi-rigid MRSF seismic-resisting system for two levels of ultimate limit state earthquake event. The first is the design ultimate limit state earthquake (DLE), as specified by the New Zealand Loadings Standard (currently NZS 4203 [3] but to be replaced in 2004 with DR1170.4 [4]) and the second is a defined maximum considered earthquake (MCE).

Under the design level ultimate limit state earthquake, the MRSF is expected to respond with minimal or no structural damage.

Under the maximum considered earthquake, the MRSF is expected to retain its integrity, to allow evacuation and post-earthquake assessment, but to suffer controlled structural damage. In this instance, repair is still to be a practicable proposition.

In terms of the force-based seismic design philosophy, for which the design procedures presented herein have been developed, it is intended that the design procedures developed for these semi-rigid systems utilise the equivalent static method or modal response method of [3,4], in conjunction with NZS 3404 [5] and, where appropriate, the established seismic design procedures for steel structures given in HERA Report R4-76 [6]. This is to ensure maximum ease of use.

Target Performance Requirements

The target performance behavioural requirements from each system for the two levels of earthquake described in the previous section are as follows:

(1) For the design level earthquake (involving a 500 year return period for buildings of normal importance from DR 1170.4 [4]);

- (i) Negligible inelastic demand in the beams
- (ii) Minimal or preferably no inelastic demand in the columns at base level (such that the column bases will be readily repairable) and none at higher levels
- (iii) The rotation demand on the joints not to exceed that associated with easy assessment and, where necessary, rapid and straightforward repair
- (iv) Column panel zones to remain essentially elastic
- (v) Lateral drift not to exceed 2%
- (vi) Lateral stiffness at the end of the elastic range of behaviour to be sufficiently great to minimise $P - \Delta$ effects

(2) For the maximum credible earthquake (i.e. based on a 2000 to 2500 year return period for buildings of normal importance from [4]);

- (i) Negligible inelastic demand in the beams, except in the vicinity of bolts to beam flange and web elements for the FBJ and SHJ
- (ii) Inelastic demand in the columns to be able to be dependably resisted (this applies especially at the base)
- (iii) Joint rotation demand may cause significant local element damage, but no overall joint failure and with repair still to be possible.
- (iv) Panel zones may yield to accommodate increased joint moments from (iii) above.
- (v) Lateral drift to be within sustainable limits, including the influence of $P - \Delta$ effects.

3. MOMENT-RESISTING FRAMES WITH SEMI-RIGID FLANGE BOLTED AND SLIDING HINGE JOINTS

Background to the Semi-Rigid Joint Development

In 1994, the Northridge Earthquake caused large numbers of partial failures in rigid welded joints of moment-resisting steel frames. These failures turned the rigid joints into semi-rigid joints, thus developing an unintended strong column/weak joint behaviour. Buildings with MRSFs that suffered these failures typically still performed well, in terms of overall response to this earthquake

In 1995, the Hyogo-Ken Nanbu earthquake caused damage to a wide range of steel framed buildings, principally older medium-rise commercial and industrial buildings. The pattern of damage showed that three factors were important in order to achieve good performance of a building with semi-rigid connections in the inelastic range. These are:

1. The beam to column connections retain their integrity, with regard to carrying shear and axial force, when their moment capacity is reduced by inelastic demand
2. Inelastic demand is minimized in the columns, both demand due to general plastic hinging and demand due to local buckling or crippling
3. The inelastic response is essentially symmetrical in nature and does not lead to a progressive movement of the building in one direction

These concepts were incorporated into the semi-rigid joints shown in Figs 1 to 4 and the MRSF systems developed incorporating these joints. Of those 4 joints and systems considered:

- The MRSF with RSJs (Fig 1) was shown to generally meet the target performance criteria given above, however the RSJ is too expensive and difficult to construct for the building superstructure
- The PTJ was taken to design development but was not a practical joint to develop further
- The MRSF with FBJs has been taken to full development and is now in use
- The MRSF with SHJs has been taken to full development but is not currently in use. Once the design and detailing provisions given in Clifton [2] are reviewed, its use will be promoted

Of these 4 systems, both the FBJ and the SHJ have been developed to meet the target performance criteria given above, which include damage resistance. The concept of each joint and modes of operation are covered in more detail below.

Concept of the Flange Bolted Joint

The FBJ involves connecting the beam to the column through plates welded to the column flange and bolted to the beam top and bottom flanges and a similar plate welded to the column flange and bolted to the beam web. The pattern of bolts in the flange plates is typical of any flange bolted beam to column connection. The pattern of bolts in the beam web/web plate is unconventional and comprises two horizontal bolt rows, one near the top of the beam web/web plate and the other near the bottom. Fig 3 shows these details, especially the web top bolts.

The FBJ is designed and detailed for high strength, low ductility demand applications. It is very simple to fabricate and erect, has a low inelastic rotation damage threshold but is capable of withstanding high levels of inelastic rotation demand through undergoing a planned mode of failure that retains the integrity and vertical, axial load carrying capacity of the joint while the moment resistance decreases with increasing rotation demand.

When developing the FBJ, the detailing of the joint and the strength hierarchy developed within it have been chosen such that:

(i) The joint remains rigid at the serviceability earthquake level, as defined by NZS 4203 [3].

(ii) At the design severe seismic level of rotation demand, bolts can force elongation into the bolt holes and the plates connected to the column, through bearing yielding of the plate/beam elements. This elongation is not to be sufficient to require plate or bolt replacement or significant loss of bolt tension.

(iii) The behaviour of the joint at the design severe seismic level of rotation demand will be maintained up to at least 1.5 times that level of rotation (with increasing yield in the plates but with the bolts retaining their integrity) and repair of the joint at that point will still be straightforward to effect.

(iv) At the MCE level of rotation demand, extensive plate/beam element yielding is expected, but bolt fracture does not occur. If the (bottom) flange plate fractures, the horizontal line of web bolts adjacent to the flange provides an alternative horizontal load path for the beam moment-induced axial actions, maintaining a reasonable moment capacity at high rotation demand.

Concept of the Sliding Hinge Joint

At the time of commencing FBJ design, initial consideration was given to whether it would be feasible to develop a joint that was still practicable to fabricate and erect, but which could withstand fully ductile levels of rotation demand (e.g. 30 milliradians of rotation) with minimum damage and loss of function. For that to occur, the joint would have to possess the following attributes when subjected to inelastic rotation demand:

(1) Be laterally pinned at top flange level, so the effect of rotation on the floor slab was minimised

(2) Be able to slide in a controlled manner at bottom flange level when the imposed moment exceeded a threshold level.
(3) Not slide at bottom flange level when the imposed moment was below the threshold level, so that it remains rigid under serviceability limit state conditions.
(4) Be able to carry the imposed vertical loading into the column without compromising the sliding mechanism.

The result is the Sliding Hinge Joint, the layout and component details of which are shown in a large scale experimental test in Fig 4 and in isometric and exploded form in Fig 5.

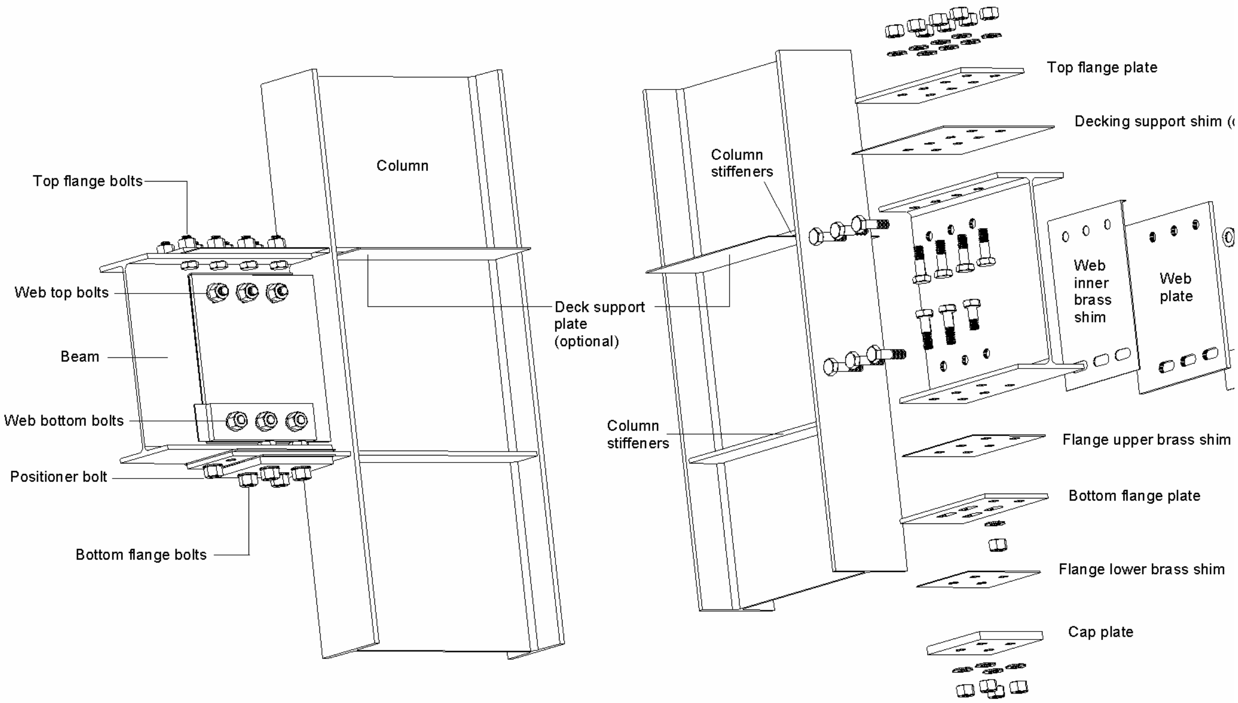

Fig 5 Isometric and Exploded Views of the Sliding Hinge Joint

The mode of operation of the SHJ is relatively simple. The beam is pinned laterally at the top flange level, using nominal sized bolt holes and FBJ details. This keeps lateral movement in the floor slab to 2-3 mm, thus minimising undesirable floor slab participation, slab damage and suppressing slab reinforcement fracture. Joint rotation is achieved through sliding at the bottom flange and the web bottom bolt level (see Fig. 5 for the location of these components).

The sliding details are shown in the isometric view of Fig. 5. The sliding layers are between the brass shims and plate (web plate, bottom bolts and the bottom flange plate). The holes for the web bottom bolts in the web plate and for the bottom flange bolts in the bottom flange plate are slotted to allow this sliding to occur. The beam flange or web and the associated cap plates all have nominal sized holes.

When the moment demand on the SHJ from earthquake generates internal beam axial forces which exceed the sliding resistance available through the bottom flange bolts and web bottom bolts, the joint will slide, allowing beam rotation to occur. As sliding occurs, the cap plate is anchored in position relative to the beam flange or web by the bolts, allowing the cap plate to also slide relative to these surfaces. Once the imposed moment reduces, there comes a point where the sliding stops and the joint becomes rigid again. This is illustrated in Fig. 6, which shows the joint rotation versus moment from the large-scale test 3. More details are given in Clifton [2].

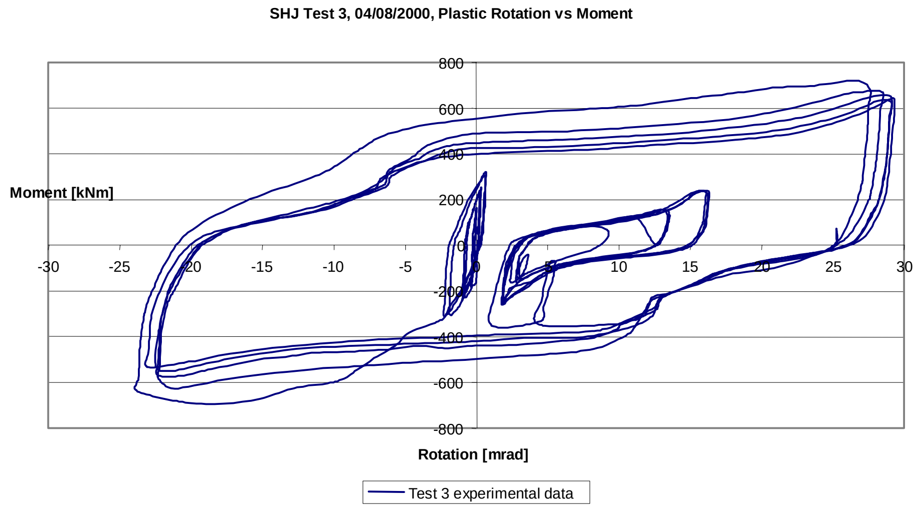

Fig 6 Moment Versus Plastic Rotation Curve for Large Scale Sliding Hinge Joint Test

On rotation reversal, the joint unloads abruptly, then the moment capacity builds up in the reverse direction, as shown in Fig. 6. The increase in moment with increasing reverse rotation occurs in two stages; one as sliding occurs along the first interface (beam to plate) and then with a further increase in sliding capacity as the second interface (plate to cap plate) is activated.

The slotted holes in the flange and web plates are designed to accommodate a joint rotation of 30 mrad (radians x 10-3) multiplied by an over rotation factor of 1.25; if the inelastic rotation demand exceeds this, the joint undergoes inelastic behaviour through flange plate yielding, in the same manner as for the FBJ.

Under the design level ULS earthquake, inelastic rotation demand has been shown, from the numerical integration time history (NITH) analyses undertaken, to be not greater than 37.5 mrad. This is accommodated within the slotted holes. At this level of rotation demand, minimum joint degradation will occur and only minor slab cracking, such that no post-earthquake repair is required. Details are given in section 5.3.2 of Clifton [2]. Figs 7 and 8 show the condition of the joint and floor slab, respectively, following testing to approximately the design level of inelastic rotation demand.

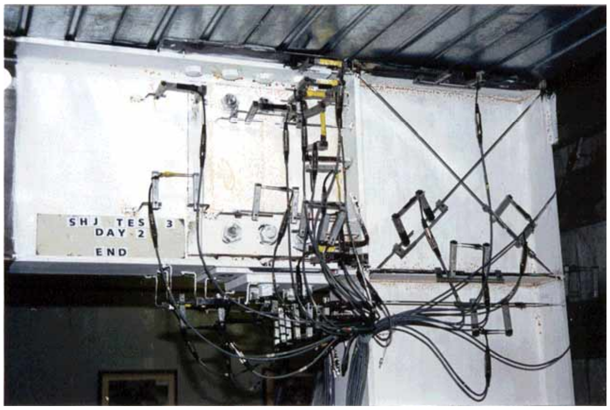

Fig 7 Large Scale SHJ at Completion of Test Loading Shown in Fig 6

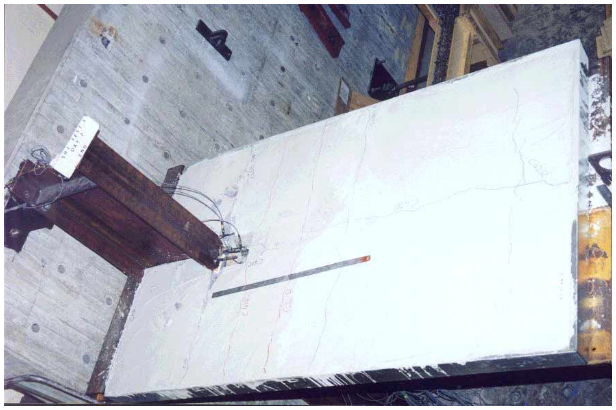

Fig 8 Slab Above SHJ at Completion of Test Loading Shown in Fig 6

Under the maximum considered earthquake (MCE), the MRSF with SHJs will retain its integrity, to allow evacuation and post-earthquake assessment, but will suffer controlled joint damage, which may necessitate replacement of components. However, the results from the NITH studies show that, in most instances, little or no reinstatement would be needed after most maximum considered events, especially for buildings not subject to near fault action.

Experimental Testing of the FBJ and the SHJ

There were 4 large scale tests undertaken on the SHJ and 19 small scale tests undertaken on one of the critical components, namely the bottom flange sliding assembly. A similar extent of testing was undertaken on the FBJ.

Figs 7 and 8 show the SHJ at the completion of the third large scale test. Fig 6 shows the nominal plastic moment-rotation curve from this test.

The four tests involved two tests each on two separate assemblages - tests 1 and 2 on the first assemblage and tests 3 and 4 on the second assemblage. Each assemblage was designed to represent, as closely as practicable, a representative full size SHJ in a MRSF. Each test specimen therefore included the effective width of concrete slab, cast onto profiled metal deck and containing the normal specified mesh reinforcement. All tests were undertaken at pseudo-static rates of loading.

The results from these SHJ tests were complemented by 19 component tests, undertaken during the second half of 2000 and first half of 2001. These focused on the behaviour of the flange plate to beam flange connection and took place in a specially built test rig powered by a 300 kN capacity servo controlled dynamic actuator. Details are given in Clifton [2]. These component tests allowed a variety of bolt and plate arrangements to be rapidly tested, at both pseudo-static and dynamic rates of loading.

The overall objective of the experimental tests was to determine the performance of the SHJ under inelastic cyclic loading, including the influence of the floor slab, in order to develop a dependable design procedure and detailing requirements for its use in a MRSF.

Analytical Modelling: Finite Element Analysis of the Sliding Component

The mode of operation of this joint under severe earthquakes involves the joint being pinned at the top flange level and able to undergo controlled friction sliding at the bottom flange and bottom row of web bolts levels. To achieve this, the bottom flange and web bottom bolts must undergo asymmetric sliding along the two sliding planes on either side of the flange plates containing the slotted holes. This requires the bolts to undergo combined tension, moment and shear actions, during which they lose a significant amount of their initial pre-tension. A design model was developed to predict the bolt sliding shear capacity. The accuracy of this model was confirmed by finite element analysis (FEA) of the sliding component. Details are in Clifton [2] and Mago [7]. This FEA modelling established the following:

- The stress distribution in the bolt under sliding conditions
- The loss of bolt pretension once active sliding occurs
- The influence of bolt loss of pretension prior to earthquake attack
- The influence of different mechanical properties in the brass shims against which the sliding occurs
- The influence of misalignment of the components on the sliding shear capacity
- What happens to the bolt tension when the end of the slotted hole is reached

Analytical Modelling of the Seismic-Resisting System

As described in items (2.1) to (2.4) of section 1, the analytical modelling is undertaken in order to determine how well the proposed frame/joint systems are expected to meet the target performance requirements specified in section 2. The analytical modelling has been undertaken on the perimeter frames for five and ten storey, rectangular in plan buildings of 35 x 21 metre footprint. They carry loads typical Clifton [2] of an office building. Each building is supported laterally by a perimeter MRSF along each wall. Fig. 9 shows an elevation of one such frame for the five storey semi-rigid joint system, illustrating the features incorporated into the analytical model.

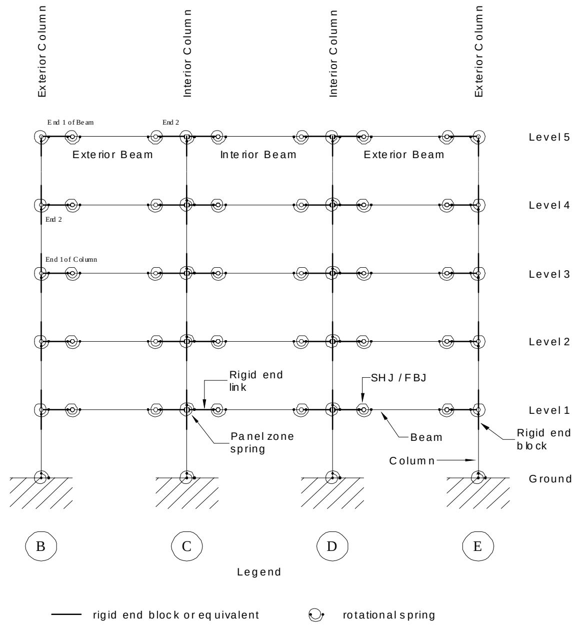

Fig 9 Analytical Model of Five Storey Frame With Semi-Rigid Connections

Over 200 analyses on a range of representative FBJ and SHJ systems have been undertaken, covering the range of seismicity and soil conditions typical of New Zealand [3,4]. Selection and scaling of earthquake records has been to [4]; Fig 10 shows the scaled spectra used for the soft soil analyses. Details are given in a number of publications, most comprehensively Clifton [2]. The results have shown that the systems meet the target performance criteria of section 2 and that the MRSF with SHJ, in particular, is expected to require little or no structural repair following a design level earthquake.

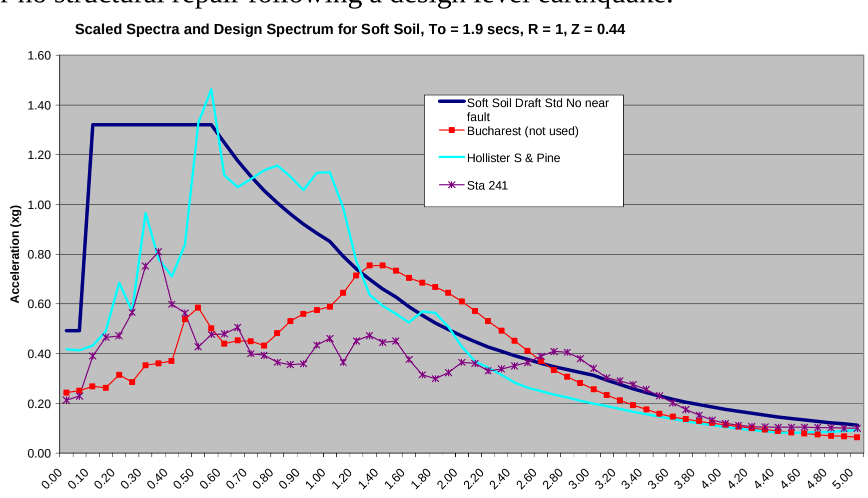

Fig 10 Scaled Earthquake Spectra Used in Time History Analyses of SHJ and FISSER Systems

Softening of the MRSF With Sliding Hinge Joints After Ultimate Limit State Earthquake Attack

An important aspect of the SHJ study has been to answer the question: what is the effect of an ultimate limit state earthquake event on the subsequent serviceability limit state stiffness of the building? This has been answered to a limited extent through two NITH studies, which have subjected the MRSF to the following:

- a serviceability limit state earthquake, followed by:
- an ultimate limit state DLE earthquake, followed by:
- the serviceability limit state earthquake.

The key points to note are that:

- The building self centres following the DLE event and the lateral deflection response with time at roof level under the post-DLE serviceability event is the same as that under the pre-DLE serviceability event. This is shown in Fig 11.
- The SHJ does soften following the DLE event, however, the maximum joint rotation increases from 2 milliradians to only 3.5 milliradians, which is not significant in regard to overall building response. This is shown in Fig 12.
- There is no inelastic demand on the columns under the DLE event, which assists in the building self centering

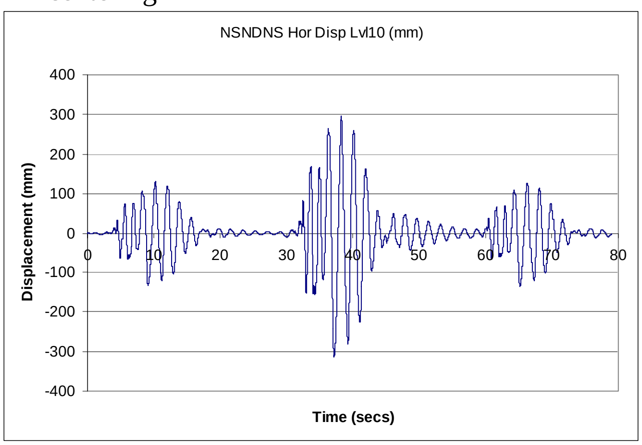

Fig 11 Lateral Deflection of Top Floor of 10 Storey MRSF During Serviceability then Ultimate then Serviceability Limit State Earthquakes

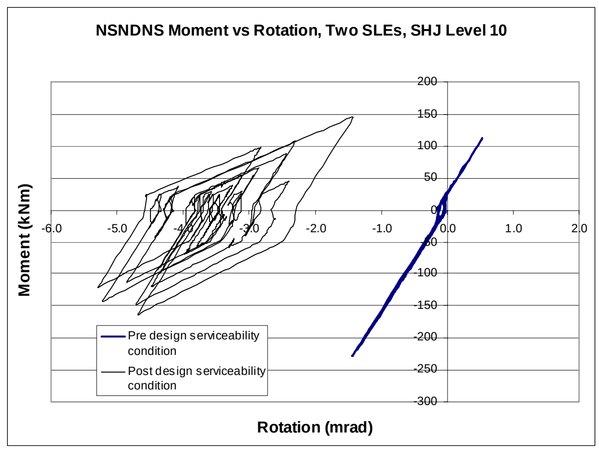

Fig 12 Rotation of Sliding Hinge Joint at Top Level Before and After DL Earthquake

Detailing and Design Procedure Available for the Frame and Joints

Comprehensive provisions for the MRSF with SHJ are given in sections 5.8 to 5.10 of Clifton [2], with a SHJ design example in section 5.11. Similar coverage is given for the FBJ.

Self Centering Sliding Hinge Joint

As has been described above, the Sliding Hinge Joint moment-resisting steel framed system (MRSF with SHJ) has a high damage threshold and reasonable self centring ability. However, following a DLE, the building will not fully self centre and the SHJ is softened by excursions into the active sliding range, as shown in Fig 12.

By replacing the bottom flange plate sliding assembly (Fig. 5) with a dual direction acting ring spring system, the self-centering ability of the SHJ should be enhanced. This concept is shown in Fig. 13. By suitable proportioning of the components, this system should be able to develop a self centering SHJ with large rotation capability. Such a self centering SHJ will represent the furthest towards a full Damage

Avoidance Design (DAD) concept possible from any moment framed system considered to date. Further research into the Self Centering Sliding Hinge Joint (SSHJ) is dependent on funding and resources being found.

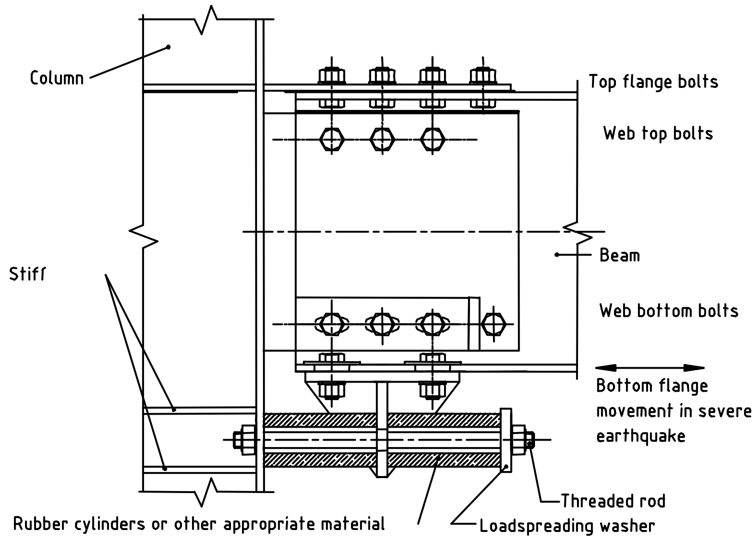

Fig 13 Self Centering Sliding Hinge Joint Concept

4. MOMENT-RESISTING FRAMES WITH PARTIALLY ISOLATED FLOORS (FISSER)

Concept

A typical multi story structure comprises a gravity load carrying system and a separate earthquake load resisting system. The former is designed to be sufficiently flexible to resist the earthquake imposed rotations without loss of gravity load carrying function, while the latter is designed with the high strength, stiffness and ductility required to resist the earthquake actions and to ensure the overall building stability. This is as applicable to the semi-rigid systems described in section 3 herein as it is to a traditional rigid framed system.

While the building is designed for earthquake action by means of an imposed lateral load, the impact of the earthquake on the structure actually comes from the ground movement reacting against the inertial mass of the building, almost all of which is in the floor system at each level. The floor system at each level is tied into the seismic system, so that the inplane diaphragm forces generated by the inertial mass at each level can get into/out of the seismic system and from the building superstructure into the ground. The lateral load resisting system must resist the forces resulting from this interaction.

The idea behind the Floor Isolating System for Superior Earthquake Response (FISSER), is to partially isolate the floor from the seismic system at each level. This will reduce the actions generated within the seismic resisting system and reduce the acceleration on the floors of the building, as well as reducing the actions transmitted through the foundations. It should deliver some of the benefits of base isolation (but being applied through each floor) with considerably less cost.

The partial floor isolation concept involves a perimeter frame using FBJs. The partial isolation of the floor diaphragm from the perimeter frame is achieved by using a compound member of the type shown in Fig 14. The I section is part of the perimeter frame seismic-resisting system (PMRSF). The floor slab is connected rigidly into the channel, but the channel is not welded to the beam. The two are connected only by the weight of the floor slab tributary to the channel, which keeps the two members together. When the earthquake occurs, the stiff seismic-resisting system will be able to move relative to the flexible floor system at each level. The system is given resistance to impact and self-centring ability by putting a spring into the gap, see Fig 15. This spring must have high strength, stiffness but be compact and able to withstand full compression without damage. Representative frame trial designs and studies to date, from the RSJ research (Clifton [2]) and current studies have shown that Ring Spring elements [8] are well suited to this application. There would need to be a gap left between the floor slab and the column in the in-plane direction to allow the design floor movement to occur.

A much smaller gap would be required in the out of plane direction, as the seismic resisting system's flexibility in this direction will allow it to deform as required, without generating large actions in the members and system. The only exception may be at the column bases where some additional detailing might be rquired or else a ring spring type base, as developed in Clifton [2], could be used.

The connections between beams and columns in the gravity load carrying system would be designed to provide an effective pinned connection. Suggested details are given in Zaki [9].

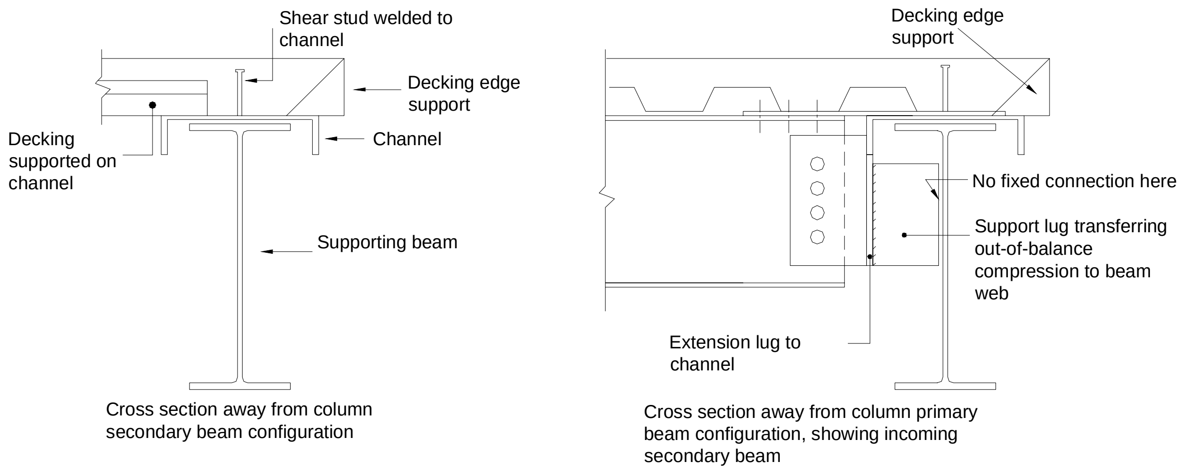

Fig 14 Cross Sections of Floor Isolating System onto Supporting MRSF Beam Configurations

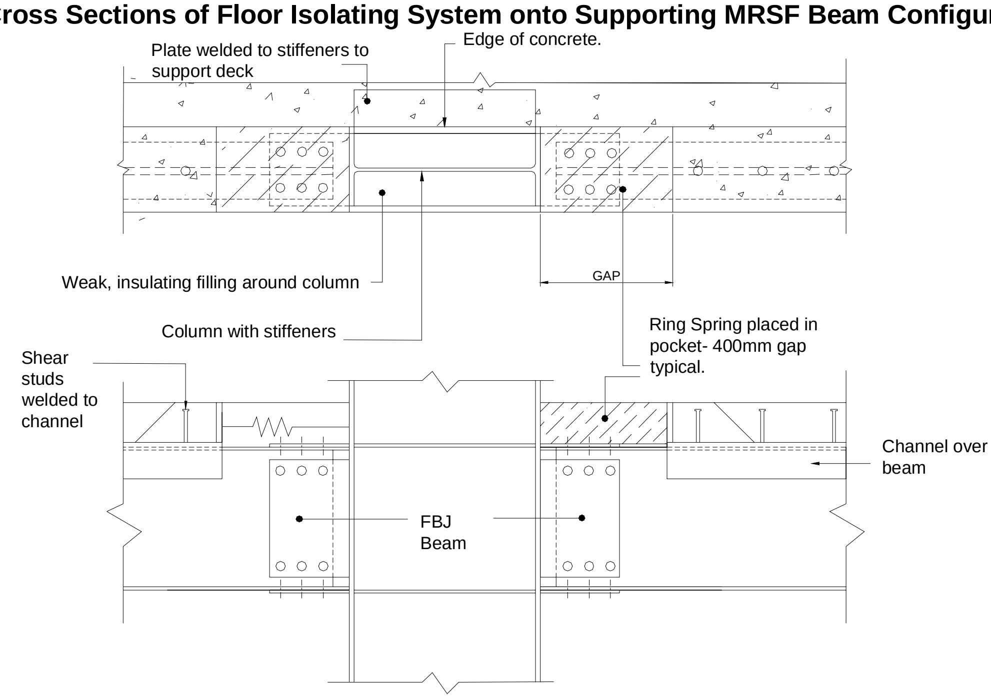

Fig 15 Floor Isolation System for Superior Earthquake Response: Details around perimeter frame beam and column

Analytical Model and Preliminary Results

During 2003, the 10 storey FBJ model was converted into a FISSER system through the addition of a representative gravity system. Details are presented in Zaki [9].

A range of FISSER spring stiffnesses and other important parameters have been incorporated into the analyses. The results showed that the concept is feasible and reduces the demands on the FBJ under the design level event (DLE) to the extent that little or no remedial repair following this event would be required. This holds even for records incorporating near fault forward directivity effects. An example is shown in Fig 16; the FBJ system with ductility 2 completely self centres even after the design level soft soil earthquake record Hollister and Pine (see Fig 10 for the spectrum of this record) which generates significant response in the 10 storey system.

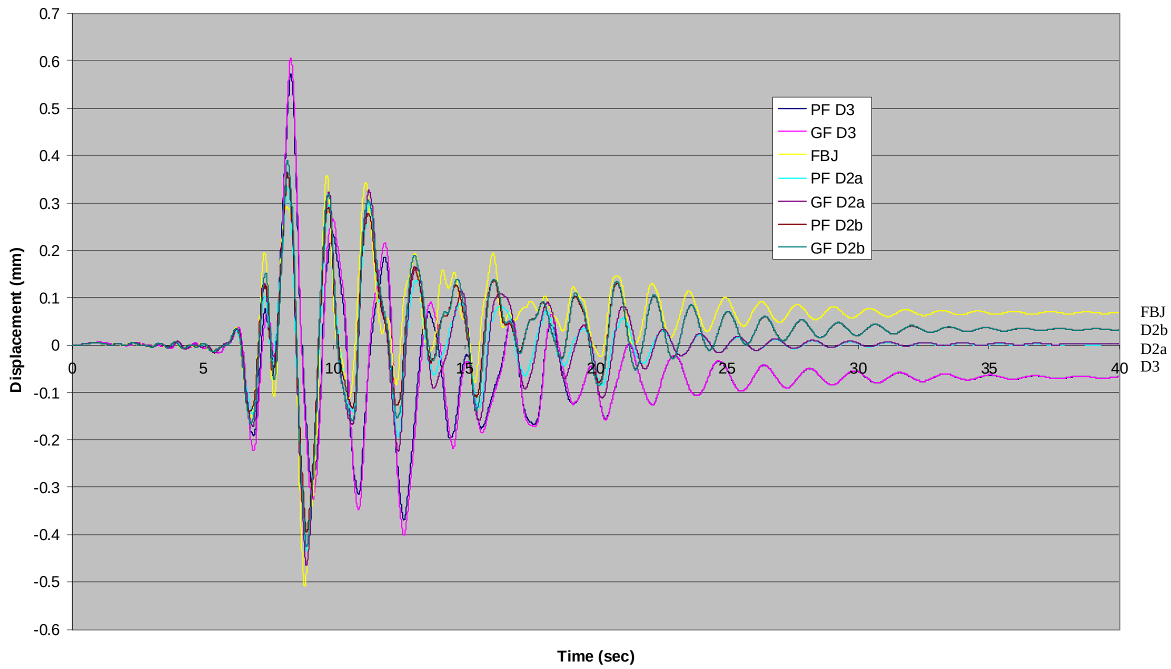

Fig 16 Lateral Deflection of Level 10 with Time, 10 Storey MRSF with Floor Isolation System

Notes: PF = perimeter moment-resisting frame in the FISSER system
GF = gravity frame in the FISSER system
FBJ = moment-resisting frame in the non-FISSER system (i.e. where the floor is integral with the frame)
The 40 seconds comprises 20 seconds of earthquake record followed by 20 seconds of free vibration

By using a FBJ model with ductility 2 as a control model, different FISSER models were subject to numerical integration time history analyses to determine their behaviour and to come up with the best model in regard to constructability and earthquake response. The key aspect of earthquake response being sought is minimum residual displacement at the end of the event, as this is a direct measure of structural damage sustained during the event. Some results are shown in Fig 16. The FBJ system, which is designed for ductility 2, provides the benchmark against which to assess the effectiveness of the FISSER systems.

The FISSER model with ductility 3 (D3) gave good results, since the residual displacement is less than the FBJ, even though the design ductility is greater. However, model D2a, which is FISSER designed for ductility 2, has produced the best results, as there is no residual displacement and the frame returns to its original place. This option would require painting the contact surfaces between beam and channel to achieve the coefficient of friction used in the analysis of 0.1 but the additional cost of this is minimal. FISSER D2b (also ductility 2 but with a coefficient of friction of 0.35 which is appropriate for unpainted contact surfaces) also gave good results in comparison to the FBJ. It did, however, have a small residual displacement, which is still less that the FBJ but more than FISSER option D2a. It is just outside the construction tolerance requirements of NZS 3404 [5]. Refer to Zaki [10] for more details.

5. CONCLUSIONS

The conclusions from the research reported on herein are that:

1. The concept of a semi-rigid moment-resisting steel framed seismic-resisting system based on a weak joint, strong column philosophy, has significant advantages over a conventional rigid MRSF for both normal design and for damage resistant design.
2. Two suitable semi-rigid joint details have been developed; one of these (FBJ) is simple to fabricate and erect but has a low damage threshold, the other (SHJ) is also simple to fabricate, less simple to erect but has a high damage threshold.
3. A partial isolation system to apply at a floor by floor level is feasible and delivers some benefits in terms of increased damage resistance under design ultimate limit state earthquake attack.

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