The Asymmetric Friction Connection with Belleville springs in the Sliding Hinge Joint

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ABSTRACT: The Sliding Hinge Joint (SHJ) is a joint for moment resisting steel frame (MRSF) seismic-resisting systems. It is ideally intended to be rigid under serviceability limit state (SLS) conditions. For greater shaking, up to the ultimate limit state (ULS), rotation between the column and beam is expected. At the end of the shaking the joint is expected to seize up and become rigid again. A key component in the sliding hinge joint is the asymmetric friction connection (AFC) which allows large beam-column rotation with minimal damage through sliding. However, previous research has shown that, after a severe earthquake, the post sliding strength and stiffness of the SHJ connection as currently applied is considerably reduced, such that re-tightening or replacement of the bolts is likely to be needed. This is because the AFC bolts lose part of their initial tension during joint sliding. Hence the joint falls short of meeting one of the key original low damage performance requirements of not requiring any structural intervention following a severe earthquake. A remedy to this shortcoming could be using Belleville springs in the AFC. However, the introduction of Belleville Springs potentially reduces the self centering ability of the building and the impact of this effect must be considered.

Belleville springs are truncated conical washer-type elements that compress elastically to an accurately defined level of force. When the bolt relaxes, the Belleville spring pushes out to maintain very close to the installed level of tension. The behaviour and sliding shear capacity of the AFC with Belleville springs is to be investigated in this project and will be presented in this paper considering different configurations using a plastic theory based bolt model. A method of installing the bolts using Belleville springs is then proposed. The predicted effects of using the AFC with Belleville springs on the SHJ characteristics such as dynamic self centering properties are also discussed. Finally, the proposed research program to determine the performance of AFC's with Belleville springs and their influence on the overall building performance is outlined.

1 INTRODUCTION

Moment resisting frames (MRFs) are one of the most commonly used lateral force resisting systems for steel buildings. MRFs resist lateral loads through rigid frame action principally by the development of bending moment and shear force in the beams and columns.

A key benefit of MRFs compared with braces or shear walls is their architectural versatility providing maximum flexibility in space usage through their openness. The other benefit of MRFs is their very high structural ductility capacity if are appropriately designed and detailed. On the other hand, a disadvantage of MRFs compared to braced frames, is their lower stiffness, requiring larger members to satisfy specified drift limits. In a rigid framed MRF, stiffness and strength are coupled so that increasing the member sizes to control stiffness will also make the frames stronger than desired. This in turn will impose higher demands on the substructure/foundation system.

Although steel MRFs have a history of good performance in past earthquakes, with few reported collapses and little loss of life related to these systems, their performance has not been entirely as expected in strong earthquakes. Fractures observed at welded moment connections in both the 1994 Northridge Earthquake (Hamburger and Frank 1994) and the 1995 Kobe Earthquake (Park *et al.* 1995) showed vulnerabilities in steel MRF systems. In the abovementioned earthquakes, many traditional rigid welded steel connections suffered unexpected brittle fracture, mostly in the beam bottom flange to column flange welds. However, these partial failures did not lead to building collapse and demonstrated that semi-rigid MRFs, where the connections are weaker than the beams, can deliver satisfactory earthquake performance.

2 IMPROVEMENTS IN STEEL MRFS' SEISMIC PERFORMANCE

2.1 The capacity design philosophy

The experience of the 1994 Northridge Earthquake, followed by a similar experience in the 1995 Kobe Earthquake, showed a basic shortcoming in beam-to-column connections in steel MRFs (Engelhardt 2001) and led to the development of procedures to avoid weld failure in steel connections. These methods, such as the reduced beam section and bolted flange plate connections (Roeder 2002) are typically based on capacity design principles. The concept of capacity design is comparable with the electrical wiring in a building. The electrical wiring is protected by a fuse "which is necessarily weaker than the wiring", so that in case of an electrical overload, the fuse burns out before the wiring is damaged.

In a seismic-resistant frame which is designed based on the capacity design philosophy, the designer must first choose the frame locations where yielding is intended to occur, i.e., the locations of plastic hinges "fuses" which in a rigid MRF are at the beam ends, and the other frame elements "columns and connections" are designed to develop the capacity design derived actions from the beams. In an earthquake, the plastic hinges limit the forces that can be transferred to the remainder of the frame, thereby suppressing inelastic demand in the remainder of the frame.

Despite being effective in providing safety and preventing collapse, these systems are usually associated with irrecoverable plastic deformation in the beams or joints. This can cause heavy undesirable economic losses both in the post-disaster repair, and downtime due to closure of the building (Khoo 2013).

Additionally, because strength and stiffness are coupled in a rigid jointed MRF and the stiffness is often the governing factor for beam size selection, the frames can end up stronger than desired which in turn puts large actions into the foundation system, requiring increased costs of construction to avoid the risk of failures in the foundations.

2.2 Low damage steel MRFs

To avoid the undesirable abovementioned economic effects of earthquakes, the global tendency has been changing toward the development and implementation of low damage seismic resisting systems. In addition to preventing building collapse, the aim is to make the building operational rapidly or, ideally, immediately after a major earthquake and to fix any probable minor damage easily and cheaply.

A number of low damage alternatives to plastic beam hinging in steel MRFs have been developed, all of which involve making the beam to column connections semi-rigid with the earthquake-induced rotation being accommodated within the connection. Such systems include slotted bolted connection, post-tensioned steel tendon, shape memory alloy systems, and the Sliding Hinge Joint connection to minimise permanent deformation and residual drifts in buildings resulting from earthquake shaking. Low damage steel MRFs involve beam end connections providing nonlinear resistance that dissipate energy with negligible damage, while the structural members remain elastic. Beam stiffness and connection strength are decoupled meaning beams can be made stiffer to control lateral deflections without increasing the overstrength actions going into the columns.

The slotted bolted connection (SBC) is a friction damper that dissipates energy through relative slip between the interfaces of bolted plates providing a non-linear behaviour whereby slip occurs at a predetermined friction force based on the level of clamping force and coefficient of friction. Popov and Yang (1995) proposed and tested a rotational beam-column moment-resisting connection with SBCs in the top and bottom flanges of the beam. It was shown to provide ductility with limited degradation through sliding in the SBCs, but this system is expensive and difficult to be fabricated and erected. Additionally, the performance of the SBC in the top flange is affected by the floor slab, meaning that lateral movement at floor slab level relative to the supporting column should be minimised in order to reduce the slab participation. The Sliding Hinge Joint by Clifton (2005) adopted the concept of pinning the beam to column at top flange, utilising the Asymmetric Friction Connection (AFC) at bottom flange and bottom web level instead of the SBC.

Danner and Clifton (1995) studied the feasibility of post-tensioning beams to the columns being inspired by the beam to column PRESSS system used in concrete structures (Priestley and MacRae 1996).These systems were involved in a gap-opening at the beam-column interface during large storey drifts, resulting in severe damage to the overlaying floor slab. Thus, they were recommended only to be used in the column bases in presence of the ring springs. USA researchers (Herning *et al.* 2012) have developed this concept further, with proposed detailing of the floor system to accommodate the gap opening, but they have not solved all the design and detailing issues associated with this form of construction.

A number of researchers have proposed using the unique characteristic of shape memory alloys (SMAs) not present in most traditional materials to recover large nonlinear deformations in seismic resistant systems through undergoing reversible micromechanical phase transition processes by changing their crystallographic structure (Fugazza 2005). Various beam-column connections using SMA bars to develop joint moment capacity, dissipate energy, and provide a re-centering property have been proposed, but the implementation has yet been limited by either the high cost and difficulties in machining or through the less than complete recovery of elongation or bar fracture (Khoo 2013).

3 THE SLIDING HINGE JOINT CONNECTION (SHJ)

The Sliding Hinge Joint (SHJ) is a low-damage beam-column connection developed by (Clifton 2005) used in steel moment-resisting frames. It allows large beam-column rotation with minimal damage through sliding in asymmetric friction components (AFCs) that are located at the web bottom bolt and bottom flange level, as shown in Figure 1. The SHJ is ideally intended to be rigid under SLS conditions, become semi-rigid allowing column to beam rotation to occur in a major ULS earthquake and, at the end of the earthquake, seize up and become rigid again.

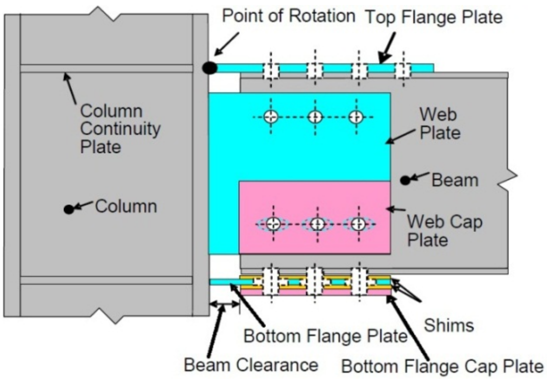

Figure 1. SHJ Setup with AFC components (not to scale) (MacRae *et al.* 2010)

When subjected to an increasing moment, the SHJ initially behaves like a rigid connection, until the moment in the beam overcomes the frictional resistance of the bottom web and bottom flange AFCs. When this occurs, the beam rotates about the top flange plate (relative to the column) through sliding in the AFCs while dissipating energy through friction. Figure 2 shows the joint rotation under positive and negative moments. Only the bottom flange friction surfaces are shown for simplicity. The rotation about the top flange plate effectively isolates the floor slab limiting additional demands to the beam, column and slab under large nonlinear rotations. The SHJ decouples joint strength and stiffness, limits inelastic demands in the beams and columns, and confines yielding to the bolts, in which the latter is intended to be improved or ideally avoided by the current research.

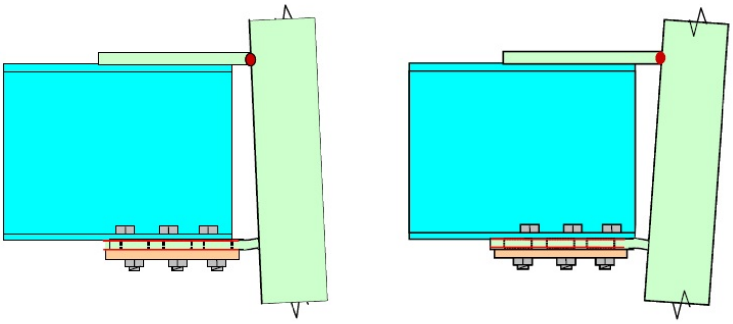

Figure 2. Positive (left) and negative (right) rotation of the SHJ

4 THE ASYMMETRIC FRICTION CONNECTION IN THE SHJ

At the onset of sliding, the SHJ moment-rotational behaviour is dependent on the AFC sliding characteristics. The configuration of the AFC in the beam bottom flange is shown in Figure 3(a). It consists of five components, including the beam bottom flange, bottom flange plate (cleat), cap plate, and two shims all clamped by the pre tensioned bolts.

The original form of the SHJ connection developed by Clifton (2005) used brass shims. Further extensive development work has been undertaken by Khoo (2013) and by Yeung *et al.* (2013) led to the replacement of the brass shims by high hardness steel shims.

The cleat is sandwiched between the beam bottom flange and cap plate, with shims in between. The web plate AFC similarly consists of the web plate sandwiched by the beam web and cap plate, with shims in between. The cleat and web plate have elongated holes to allow sliding, with standard sized bolt holes in the beam flange, cap plate and shims. Based on the design procedure that has been developed for the SHJ prior to the current research, the five plates are bolted with high strength Property Class 8.8 bolts, which are fully tensioned at installation (i.e. yielded) with the turn-of-nut method in accordance to the New Zealand Steel Structures Standard, NZS 3404 (NZS3404 1997/2001/2007).

The AFC has two sliding surfaces. The first is the bottom flange/web plate and the upper/inner shim interface, and the second is the bottom flange/web plate and the lower/outer shim plate interface. The rotational behaviour of the SHJ is dependent on the two sliding surfaces of the AFC. The idealised force-displacement behaviour of the AFC, which is schematically same as the idealized moment-relative rotation behaviour of the SHJ, is shown in Figure 3(b).

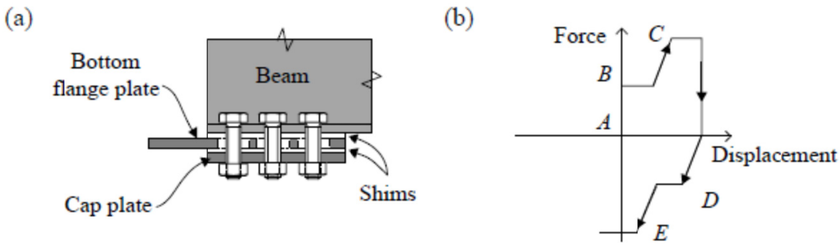

Figure 3. (a) AFC in the bottom flange plate and (b) AFC idealised force-displacement behaviour (Khoo 2013)

When the force overcomes the frictional resistance of the first sliding surface, this surface starts to slide (i.e. beam moving relative to the bottom flange/web plate), while the cap plate remains fixed to the bottom flange/web plate (shown as B). After a very short time or distance of movement along the first sliding surface, the cap plate starts to also move relative to the bottom flange/web plate, i.e. becomes fixed in position relative to the beam. From this point onwards for that direction of rotation, the bottom flange/web plate slides between two sliding surfaces, which doubles the sliding shear developed by the AFC (shown as C). At this stage, the bolts are in the double curvature state. Under load removal, the AFC retains its maximum displacement, and under load reversal shows a lower stiffness than its initial stiffness. The sliding then occurs on the first interface (shown as *D*) followed by the second interface (shown as *E*) pushing the bolts again into double curvature but in the opposite direction. This gives the SHJ a "pinched" hysteretic behaviour, which is desirable for self centering of the system, but on the other hand is not desirable for the post-earthquake retention of stiffness of the SHJ, which is the focus of this research and is discussed below.

5 THE PLASTIC THEORY BASED BOLT MODEL IN THE AFC

The AFC bolts are subjected to a moment, shear and axial force (MVP) interaction from bending in double curvature during joint sliding. The ideal bolt deformation, bearing areas on the bolt shank, and the related bolt moment diagram are shown in Figure 4.

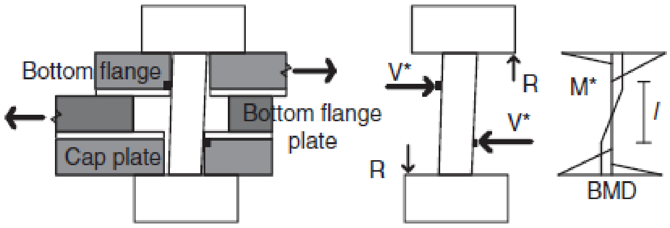

Figure 4. Idealization of bolt behaviour (MacRae *et al.* 2010)

Clifton (2005) initially proposed a plastic theory based mathematical bolt model to predict the sliding shear capacity (V_{ss}) which is defined as the amount of the shear force per bolt required to initiate a stable level of sliding in the AFC. The bolt model was then modified by MacRae *et al.* (2010) then further by Yeung *et al.* (2013) to give the current model which is presented in the equations 1-6.

$$\left(\frac{2M^*}{M_{rfn,i}+M_{rfn,j}}\right)+\left(\frac{2V^*}{V_{fn,i}+V_{fn,j}}\right)<1\ ;\ \begin{cases} i \rightarrow shank, j \rightarrow thread; If\ the\ bearing\ areas\ are\ shank/thread \\ i,j \rightarrow shank; If\ the\ bearing\ areas\ are\ shank/shank \end{cases} \quad (1)$$

$$V^* = \mu N \quad (2)$$

$$M^* = \frac{V^* l}{2} = \frac{\mu N l}{2} \quad (3)$$

$$M_{rfn,i} = S_{fn,i}\left(1-\left(\frac{N}{N_{tf,i}}\right)^c\right)f_{uf} \approx 0.1665 d_i^{\,3}\left(1-\left(\frac{N}{0.56 d_i^{\,2} f_{uf}}\right)^c\right)f_{uf}\ ;\ i \rightarrow shank, thread \quad (4)$$

$$V_{fn,i} \approx 0.62 f_{uf} \times 0.56 d_i^{\,2}\ ;\ i \rightarrow shank, thread \quad (5)$$

$$V_{ss} = 2\mu N \quad (6)$$

where: M^*=bolt moment demand; $M_{rfn,shank}$=bolt moment capacity at the shank considering axial force interaction; $M_{rfn,thread}$=bolt moment capacity at the threads considering axial force interaction; V^* =bolt shear demand; $V_{fn,shank}$=bolt shear capacity at the shank considering no axial force interaction; $V_{fn,thread}$= bolt shear capacity at the threads considering no axial force interaction; d_{shank}=nominal diameter; d_{thread}=threaded area effective diameter; N=stable sliding bolt tension; l=lever arm between points of bearing on bolt; μ=coefficient of friction between sliding surfaces; f_{uf}=average ultimate tensile stress of the bolt; c=constant taken as 1 or 2, without or with Belleville springs; $S_{fn,shank}$=plastic section modulus of the shank diameter; $S_{fn,thread}$=plastic section modulus of the threaded area effective diameter.

Table 1 shows the values of the bolt proof load (the greatest load that can be applied to a bolt without straining the bolt beyond the elastic limit), stable sliding bolt tension (N), and the sliding shear capacity (V_{ss}) for different configurations of the AFC with and without Belleville springs. The results are based on solving the equations 1-6 using MATLAB codes written by the first author in which the coefficient of friction (μ) and the lever arm (l) are considered "0.40" and [(cleat thickness)+(2×shim thickness)] respectively as suggested by Khoo *et al.* (2014). Shim thickness is considered 5mm. The average ultimate tensile stress of bolts (f_{uf}) is considered "1.12×830=930 MPa" where 1.12 is the ratio of average to nominal strengths (Clifton 2005). Because of this use of the average ultimate tensile strength of the bolts, the results are given the subscript av in Table 1. The table is also based on one contact surface being at the shank and one at the threads, which will be the case for almost all applications in practice.

Table 1. Results of solving the equations 1-6 for different AFC configurations

Bolt	d_{thread} (mm)	Cleat thickness (mm)	Bolt proof load (kN)	Without Belleville springs		With Belleville springs	
				N_{av} (kN)	$V_{ss,av}$ (kN)	N_{av} (kN)	$V_{ss,av}$ (kN)
M16	14.14	12	91.1	50.75	40.60	60.49	48.39
		16		46.78	37.42	55.85	44.68
		20		43.43	34.75	51.87	41.49
M20	17.66	12	147	87.62	70.1	103.92	83.13
		16		81.33	65.07	96.84	77.47
		20		75.99	60.79	90.67	72.54
M24	21.2	12	211.8	136.12	108.89	160.49	128.39
		16		127.05	101.64	150.62	120.50
		20		119.28	95.43	141.92	113.54
M30	26.73	20	336.6	207.06	165.65	244.94	195.96
		25		193.88	155.11	230.38	184.31

6 LOSING THE AFC BOLT TENSION AND ITS EFFECTS ON THE POST-EARTHQUAKE JOINT BEHAVIOUR

The post-earthquake behaviour of the SHJ has been significantly different from its pre earthquake behaviour. During sliding, the bolts lose some of their installed tension, causing the point at which the joint commences to slide to reduce after severe earthquake induced sliding. This is because when the joint starts to slide, the moment in the bolt combines with the high axial force and plasticize the bolt. In addition, due to the very high clamping force, very high localized bearing stress makes the whole plates and shims thinner which also drops the bolts out of the plastic range. Experimental tests confirmed that once the joint is forced into the sliding state, the AFC bolts lose installed tension and is associated with a decrease in the V_{ss} (Yeung *et al.* 2013). This can be seen also in Table 1 which shows that the stable sliding bolts' tensions are appreciably lower than their proof loads.

Thus, to restore the joint to its pre-earthquake condition, the bolts will need replacing and retightening, depending on the magnitude of sliding that has occurred. This will impose the post-earthquake restoration costs and is highly desirable and beneficial to be avoided. A remedy to this shortcoming could be using Belleville springs in the AFC to help the bolts to retain their installed tension.

7 USING BELLEVILLE SPRINGS IN THE AFC

Belleville springs, known also as "disc springs, Belleville washers, and conical compression washers" are truncated conical washer-type elements of very high strength steel that compress elastically to a flat disk under a closely defined level of force (Davet 1997). When the bolt tries to relax during sliding, the Belleville spring pushes out to maintain most of the installed level of tension. A typical Belleville spring design is shown in Figure 5 where: S=thickness; d=inside diameter; D=outside diameter; h+S=overall height; h=maximum deflection. Belleville springs can be assembled in various ways as demonstrated in Figure 6 to generate elastically compressible elements. They come in a wide range of rated strength and sizes and are similar size to standard washers so can be used in conventional layout bolted connections. When stacked up to 5 deep in parallel they can develop a compression load equal to the installed bolt tension of a high strength structural bolt.

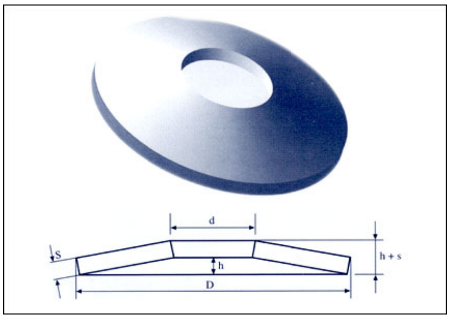

Figure 5. A typical Belleville spring design

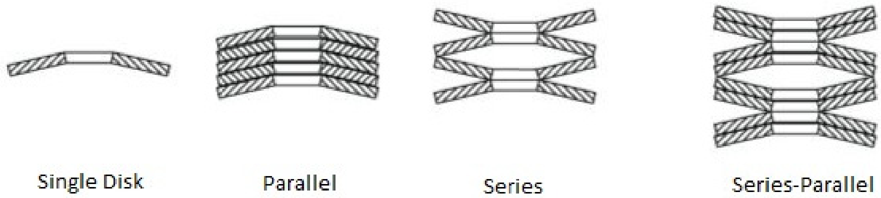

Figure 6. Various configurations of Belleville springs

Figure 7 demonstrates the bolt tension loss during a displacement controlled test on the SHJ moving the cleat relative to the beam flange (Clifton 2005). During sliding in the positive and negative directions, the bolt considerably losses its installed tension which was slightly under 350kN, until it reaches the stable sliding tension which is slightly above 175kN.

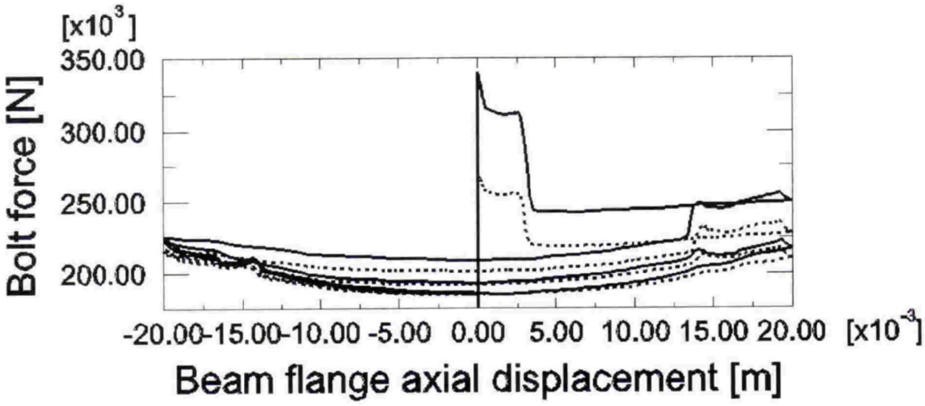

Figure 7. Bolt tension *vs* relative slip between beam flange and cleat (Clifton 2005)

The aim of the current research is to ideally prevent any loss of the AFC bolt tension during sliding. An approach to achieve this could be to install the bolts initially under their predicted stable sliding tension, as well as using the Belleville springs to compensate any probable loss of the bolt tension. One of the concerns then will be choosing the number of Belleville springs along with the method of installing them. Supposing that n is the number of Belleville springs in flat parallel stacking configuration, needed to generate a force equal to the predicted stable sliding bolt tension N, the Equations 7-11 could be a primary guide to calculate the amount of turn “rotation” of the wrench “or nut” needed to just flatten the Belleville washers from the uncompressed state.

$$n = \frac{N}{K_{BeS} \times h} \quad (7)$$

$$K_{bolt} = \frac{AE}{L} \quad (8)$$

$$\Delta_e = \frac{N}{K_{bolt}} = \frac{NL}{AE} \quad (9)$$

$$\Delta_t = \Delta_e + h \quad (10)$$

$$R = \frac{\Delta_t}{P} \times 2\pi \quad (11)$$

where: K_{BeS}=stiffness of each Belleville spring; K_{bolt}=longitudinal stiffness of the bolt; h=maximum deflection of each Belleville spring shown in Figure 5; A=cross sectional area of the bolt; E=elastic modulus of the bolt material; Δ_e=elongation of the bolt due to the force N; Δ_t=total vertical displacement of the nut necessary to just flatten the stack of Belleville springs; R=amount of rotation of the nut in radians to just flatten the stack of Belleville springs; P=pitch of the bolt threads; L=effective length of the bolt=[beam flange thickness+cleat thickness+cap plate thickness+(2×shim thickness)+hardened washer thickness+(n×S)].

If the bolts are initially installed with Belleville springs based on their stable sliding tension, which is in the elastic range, they are unlikely to lose much of this tension during sliding in a severe earthquake. This is because they are under an installed tension force which is less than the proof load and not the very large initial tension force associated with the part turn method which would delay sliding on the second interface and lead to the rapid drop in bolt tension shown in Figure 7. Additionally, any reduction in tension due to a decrease in the effective bolt length will be compensated for by the elastic stored energy of the Belleville springs. Thus, it is expected that the AFC, and as a result the SHJ, will show the same or very similar behaviour after the earthquake as before the earthquake without any considerable changes in their stiffness and hysteresis curve. Khoo (2013) found that Belleville Springs also have the potential to reduce variability in sliding shear capacity (V_{ss}) due to their ability to control, retain and moderate bolt tension, as the V_{ss} is a function of the installed bolt tension (*N*) as determined from the bolt model and which is less than the proof load and the coefficient of friction between the sliding surfaces (μ).

Consequently, the AFC with Belleville Springs can meet the original objective of the SHJ, which is to return to effectively rigid following a major earthquake with sufficient residual joint strength that enables it to be rigid in a subsequent serviceability limit state event. However, beside the beneficial aspects of using Belleville springs in the AFC, there is an important possible disadvantage that should be considered. The idealized hysteresis curve of the AFC without Belleville springs is closer to the flag shaped hysteresis curve which would provide for dynamic self centering capability for the SHJ. However, using Belleville springs in the AFC is expected to provide a closer to square hysteresis curve for the AFC which negatively affects the self centring capability of the SHJ. The aim of this research is to reach an optimum balance between the two abovementioned characteristics i.e. post-earthquake strength and self centering capability of the SHJ and to look to the influence of other elements in the system to generate the self centering ability.

Khoo (2013) suggested using the ring springs at the bottom of selected joints in a building helping the whole structure to be self centered. However, this is costly and creates the architectural limitations. An appropriate approach to solve this problem could be designing the column bases in the elastic range helping to improve the self centering ability of the whole building.

8 SCOPE OF THE RESEARCH

The current research is aiming to investigate the following concepts:

- The instruction of installing the bolts with Belleville springs considering the possible options such as calculating the amount of nut rotation or using a micrometre to measure the Belleville spring stack overall height.
- The exact lever arm in the bolt model in presence of the Belleville springs. As it is probable

that using the Belleville springs increases the lever arm.

- The accurate value of the coefficient of friction (μ).
- The optimum amount of the installed bolt tension and its effect on the AFC behaviour.
- The optimum location of the Belleville springs. To find out whether they should be put under the nut, under the bolt head, or both.
- The accuracy of the bolt model with Belleville springs, especially for the stronger bolts e.g. M30.
- The effect of the number of bolt rows on the AFC behaviour considering the prying effect. The minimum of two rows is advised by Clifton (2005).

9 CONCLUSIONS

This paper briefly describes the moment resisting frames and their features. A background of the MRFs' performance in the past earthquakes is presented which led to proposing the capacity design philosophy and then the low damage moment resisting frames. The sliding hinge joint (SHJ) as a low damage system in MRFs and its behaviour is explained followed by a discussion on the asymmetric friction connection (AFC) as a key component of the SHJ.

A key issue of the SHJ which is losing the bolt tension in the AFC during sliding is addressed along with its reasons which are (MVP) interaction as well as abrasion of the AFC components causing the bolt to become shorter. Installing the bolts in the elastic range and the use of Belleville springs as the remedy to these issues are explained and the results of solving the plastic theory based bolt model for different configurations of the AFC are presented. A method of installing Belleville springs based on the nut's turn is also formulated. The probable negative effect of using Belleville springs which is deteriorating the self centering capability of the building by retaining the SHJ's stiffness and strength is mentioned and designing the base column in the elastic range is mentioned as a primary solution to solve this problem. Finally, the general scope of the research is outlined.

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