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SHAKING TABLE TESTING OF A THREE STOREY STEEL FRAME BUILDING INCORPORATING FRICTION-BASED CONNECTIONS: STRUCTURAL DESIGN AND DETAILING

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Abstract

Recent severe earthquakes, such as Christchurch earthquake series, worldwide have put emphasis on building resilience. In resilient systems, not only life is protected, but also undesirable economic effects of building repair or replacement are minimized following a severe earthquake. Friction connections are one way of providing structure resilience. These include the sliding hinge joint with asymmetric friction connections (SHJAFCs) in beam-to-column connections of the moment resisting steel frames (MRSFs), and the symmetric friction connections (SFCs) in braces of the braced frames. Experimental and numerical studies on components have been conducted internationally. However, actual building performance depends on the many interactions, occurring within a whole building system, which may be difficult to determine accurately by numerical modelling or testing of structural components alone. Dynamic inelastic testing of a full-scale multi-storey composite floor building with full range of non-structural elements (NSEs) has not yet been performed, so it is unclear if surprises are likely to occur in such a system.

A 9 m tall three-storey configurable steel framed composite floor building incorporating friction-based connections is to be tested using two linked bi-directional shake tables at the International joint research Laboratory of Earthquake Engineering (ILEE) facilities, Shanghai, China. Beams and columns are designed to remain elastic during an earthquake event, with all non-linear behaviour occurring through stable sliding frictional behaviour, dissipating energy by SHJAFCs used in MRFs and SFCs in braced frames, with and without Belleville springs. Structural systems are configurable, allowing different moment and braced frame structural systems to be tested in two horizontal directions. In some cases, these systems interact with rocking frame or rocking column system in orthogonal directions subjected to unidirectional and bidirectional horizontal shaking. The structure is designed and detailed to undergo, at worst, minor damage under series of severe earthquakes. NSEs applied include precast-concrete panels, glass curtain walling, internal partitions, suspended ceilings, fire sprinkler piping as well as some other common contents. Some of the key design considerations are presented and discussed herein.

Keywords: Resilience, SHJAFCs, SFCs, ILEE, NSEs

1. Introduction

Severe earthquakes occur infrequently but place very high demands on structures. To economically allow for this, the concept of designing for controlled damage in a severe earthquake has been well developed and implemented for several decades [1]. However, experience from severe earthquakes has been that, while this approach is excellent for preserving life safety, the repair costs and downtime resulting from the controlled damage is very high [2]. To reduce the damage and downtime, there is a need to develop a low damage structural system which can be occupied immediately following an ultimate limit state (ULS) earthquake and should be repairable with low cost in a short time when subjected to more severe earthquakes. The 2010-2011 Canterbury earthquake sequence shows that the performance of controlled damage designed steel structures is very good, with these either not needing repair or able to be readily and rapidly repaired [3]. Development of such systems has been underway in New Zealand before Canterbury earthquakes and has been continuing well [1] [2].

Clifton [4] initially developed the SHJAFC and proposed a plastic theory based mathematical bolt model to predict the sliding shear capacity which is defined as the amount of the shear force per bolt required to undergo stable sliding in the AFCs. This bolt model is then modified by MacRae et al. [5] and then further modified by Yeung et al. [6]. Ramhormozian et al. [7] [8] [9] further investigate the AFC with Belleville Springs (BeSs) in the SHJ showing improved dynamic self-centring property and retained elastic strength and stiffness of the joint.

Traditional concentric braces dissipate energy by yielding in tension and buckling and yielding in compression. Because of the different strength in tension and compression, they are not often permitted to be major energy dissipating element in tall structures according to worldwide codes [10]. The concept of SFC braces used as energy dissipaters in framed buildings is initially introduced by Pall and Marsh [11], showing a square shape repeatable hysteresis loop. They showed that the brake lining pads exhibit a negligible degradation when subjected to a number of cycles comparable to the cycles that a brace can undergo during a severe earthquake. The SFCs can now be considered as efficient components to dissipate energy because they are characterized by stable hysteretic behaviour, low strength degradation and assembling cost comparable to conventional construction [12] [13].

A configurable three-storey steel frame composite floor building incorporating friction-based moment and braced frame connections will be tested at the International joint research Laboratory of Earthquake Engineering (ILEE) facilities, Shanghai, China. The purpose is to develop and/or examine damage avoidance design of steel structures based on a complete building system, in which precast concrete panels, glazing curtain walls, suspended ceilings, internal partition walls and fire sprinkler system will all be included through shake table testing. The building response in terms of dynamic characteristics, residual drift, post-earthquake loss of stiffness and influence of NSEs between numerical models and actual structure are to be analysed and compared which cannot be fully understood in a component level test.

Four parts are related to this ILEE testing of RObust BUilding SysTem (ROBUST), 1) an overview of ROBUST [14], 2) design and detailing of NSEs, 3) design and detailing of general structural systems and 4) design and detailing of specialized structural systems [15] [16]. In this paper, the general structural parts including the detailing of friction type connections (i.e. SHJAFC and SFC) as well as important design considerations have been presented and discussed. The testing is to be conducted in early August 2020, by then, a more comprehensive understanding of building's seismic performance will be gained for future facilitating the engineering applications.

2. Building information

The building considered in this project (see Fig. 1) is a 3-storey steel frame building with plan dimensions of 7250 mm by 4750 mm (from centre to centre, see Fig. 2) and an inter-storey height of 3 m. The building is considered to be of normal importance (Importance Level 2 as per NZS 1170.0 [17]) and located on shallow soil (Subsoil Class C as per NZS 1170.5 [18]) in the Wellington CBD within 8 km of the nearest fault. The floor system is designed as steel metal deck with concrete topping (ComFlor 80 details applied). The seismic force resisting system is MRF incorporating SHJAFC and CBF V-braced system with braces effective in compression and tension using SFC in the long (marked as X) and short direction (marked as Y), respectively. To be noted, these seismic resisting systems are fully configurable and can be changed into other systems simply by undoing the bolts and adding new components. The centre column is designed and detailed to carry gravity load only but remains capable of undergoing imparted seismic drift deformations when adopting a rocking system. Several improvements have been made to the proposed structure to be tested since last time reported by Yan et al. [19]. The details of major design considerations are discussed in the following section.

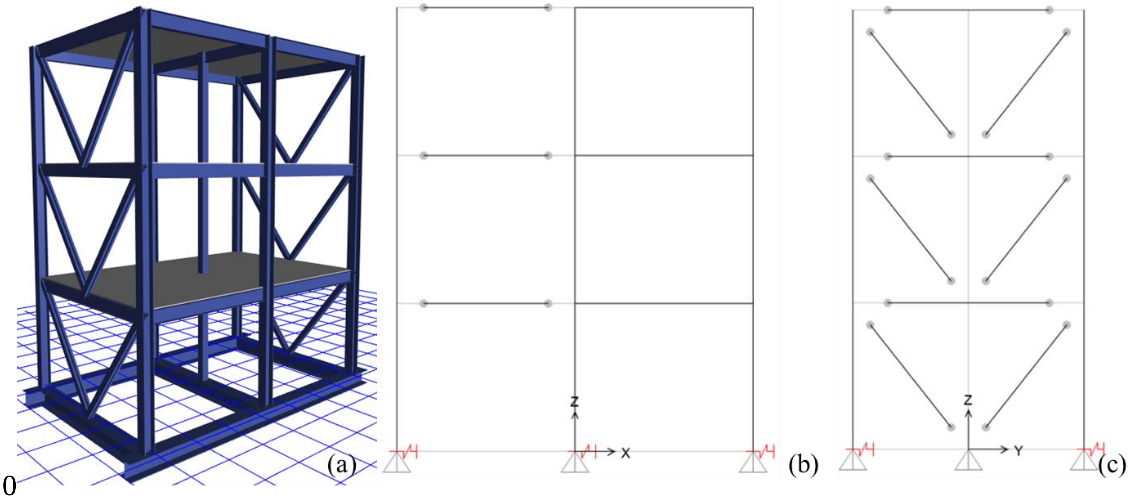


Fig. 1 – (a) 3D View, Elevation in (b) Long Direction and (c) Short Direction of Proposed Structure

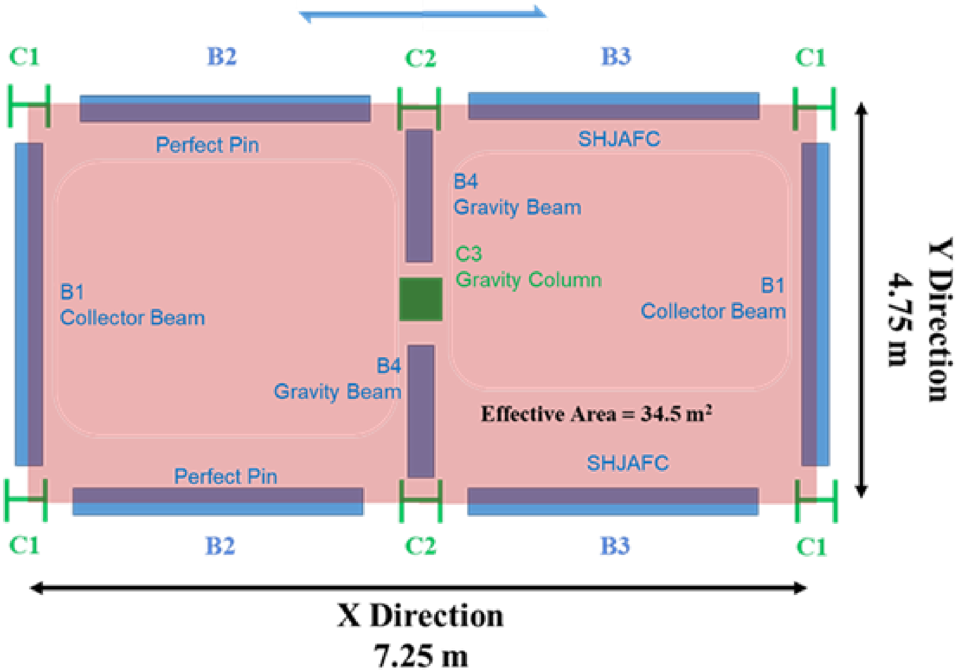


Fig. 2 – Plan View of Proposed Structure

The elevations in two horizontal directions (X and Y) of MRF-SHJAFC and CBF V-braced system using SFC braces are given in Fig. 3 (a) and (b), respectively. The design and detailing of key connections (i.e. SHJAFC and SFC) have been reported by Yan et al. [19], thus not repeated herein. As can be seen in Figure 3, the collector beams (B1) and longitudinal beams (B2 and B3), unlike what normally is the case in practice, are not fully covered by the slab, especially around the column, which initially aims to help with changeover activity between different structural systems, making it possible to test different concepts within one main frame. However, by doing so the stability of the structure needs to be carefully considered with the absence of part of the slab. To be further noted here, such gap is not required for repairing and replacement in an actual building but to provide flexibility for changeover activities of different structural systems.

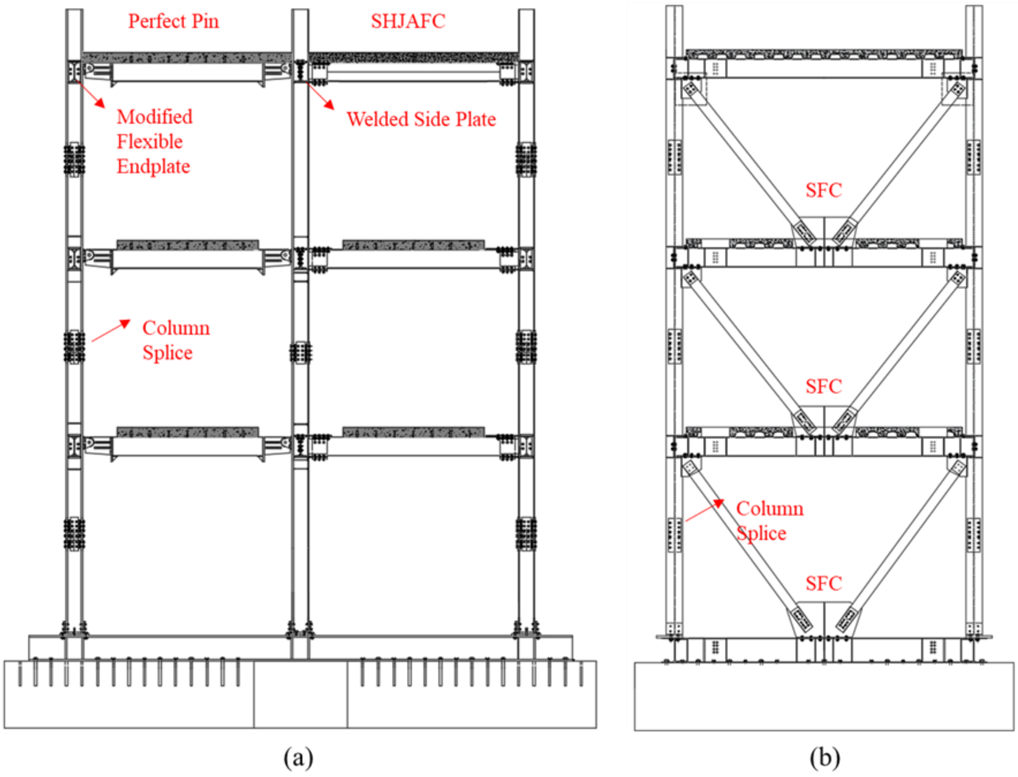


Fig. 3 – (a) MRF-SHJAFC in X Direction and (b) CBF V-braced System using SFC Braces in Y Direction

3. Design considerations

3.1 Detailing of SHJAFC

3D view of SHJAFC adopted in MRF in the long direction is shown in Fig. 4, looking from two different views.

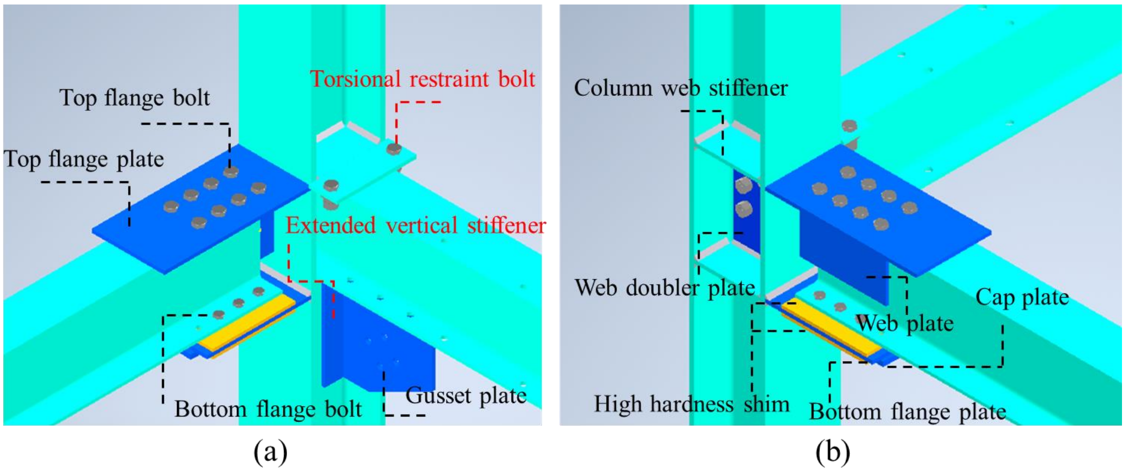


Fig. 4 – 3D View of SHJAFC (a) Inside and (b) Outside of the Frame

The SHJAFC herein only keeps the bottom flange bolt group (circled in red dashed line) with no web bottom bolt group from typical layout of SHJAFC [4]. One of the reasons is to reduce the capacity of the joint due to a limitation of the shake table capacity (or to put it in other way, to reduce the strength of the connection keeping realistic size of the structural bolts) and the fact that the structure aims to be tested as far as possible even over maximum considered shaking level. The effects of using BeSs against hardened washers and NSEs in terms of cumulated sliding distance, post-earthquake loss of stiffness as well as other key parameters can be then verified.

3.2 Detailing of CBF using SFC brace

3.2.1 Consideration of Cs factor

The design seismic coefficient is the product of the lateral force coefficient, C (T) in accordance with NZS 1170.5 [14] which is calculated based on the chosen structural ductility factor, μ and the factor C_s from NZS 3404 [16]. The C_s factor presented in NZS 3404 [10] takes the less-than-ideal inelastic behaviour of CBF systems into account including 1) the departure of the CBF system from the optimum O-mechanism system (the whole structure undergoes some inelastic displacement and plastic hinges are spread throughout several levels of the structure), 2) the less than ideal hysteretic behaviour of the CBF system and 3) the deterioration in inelastic performance. With the presence of the SFC, the brace has a similar capacity in both compression and tension. The inelastic behaviour only occurs at the joint while the brace remains elastic during the stable sliding stage. Therefore, the C_s is taken as 1 regardless of the effect of the compression brace slenderness ratio. The seismic design action can be obtained from C (T) directly.

3.2.2 Detailing of SFC

The SFC is designed and detailed at the brace bottom end to the gusset plate joint as shown in Fig. 5. To form a perfect symmetric friction connection, the slotted holes are formed within the gusset plate which then sandwiched by two rectangular hollow sections (RHSs). High hardness shims are placed between the gusset plate and the brace body for a stable sliding condition. The length of slotted holes (in the gusset plate) is calculated based on the maximum brace extension/ compression at 3% drift herein this project. The gusset plate is designed and detailed following a notional load yield line (NLYL) method [20]. The most critical case occurs when the brace is extended to the most outer place and being compressed back.

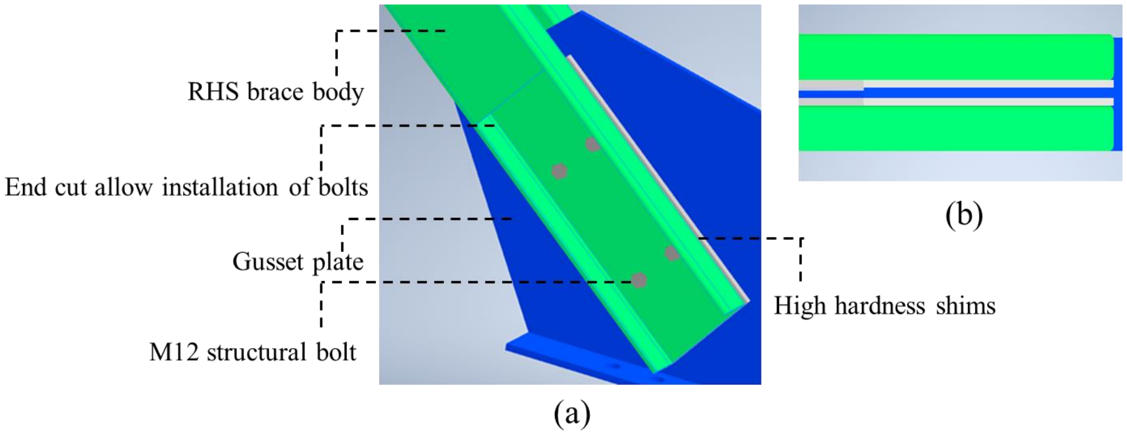


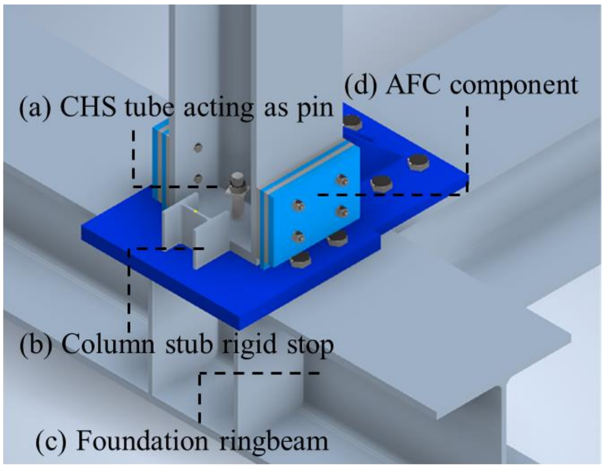
Fig. 5 – SFC at Brace to Gusset Plate Joint (a) Oblique View and (b) Plan View

3.2.3 Consideration of unbalanced force for CBF using SFC braces

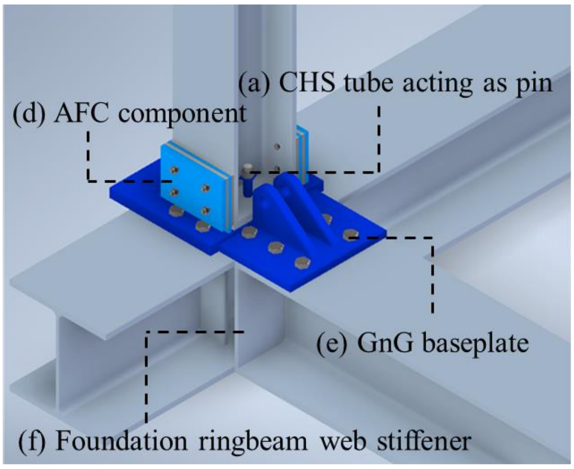
For a conventional CBF system, the compression force that a brace can resist after buckling is a function of the brace slenderness ratio [10]. The brace post-buckling compression capacity is estimated by multiplying the 'pre-buckling' compression capacity by the term, α_c' [10]. Such reduction in the capacity of the compression member will result in an unbalanced force from the tension brace at the midspan gusset plate to collector beam joint. However, due to the presence of the SFC, once the sliding force of the connection is reached, the bolts slide along the slot in the gusset plate. Especially that the braces are designed to remain elastic and not to buckle, based on the SFC's overstrength sliding shear force under stable sliding stage. Hence, for braces with SFC, the unbalanced force will not come from the brace buckling but the degradation of SFC. It is reported that for SFC the degradation is no more than 25% after a cumulated sliding distance of 6000 mm (Xie et al., 2018). α_c' is thus taken as 0.75 for design and detailing of CBF using SFC braces.

3.3 Column base connection at corner column

The detail of column base connection at corner column C1 is shown in Fig. 6. This detail is designed to be able to act as 1) fixed known as strong axis-aligned asymmetric friction connection (SAFC base), 2) effectively pinned (for CBF system, preventing uplift) and 3) free to rock (for rocking frame incorporating Grip and Grab (GnG) device [16], allowing uplift). The authors believe that such concurrent column detail should be avoided in practice, however, to accommodate different structural system (i.e. rocking frame concept) making the best use of the main structure, such compromise is essential and won't be an issue with proper detailing.



(a)



(b)

Fig. 6 – Column Base Connection at Concurrent Column (a) and (b)

For the fixed condition, the column base is designed and detailed as column base strong axis-aligned asymmetric friction connection (SAFC Base). For the pinned case, the AFC bolts group will be removed and a CHS tube will be added to prevent the uplift. A short column section is welded on the left-hand side of the column acting as a column stub rigid stop while on the other side the hold down device for GnG concept will do the same work. This is to assure that the column will not slide more than 1 to 2 mm and reduce the demand on the centre shear rod. With such detail, the shear rod will act as a hold down device instead of carrying horizontal shear force which may be undesirable. For the rocking case, the column is expected to rock/rotate without any limitation besides the gravity load. So, the bolts will be removed leaving a clear space for the column to uplift. To be noted, one of the reasons that this shear rod is not fully threaded but with a shank area is to protect the rod making sure the damage part will not be on the threaded parts. The reason is once the threads are damaged then the nuts could no longer fit.

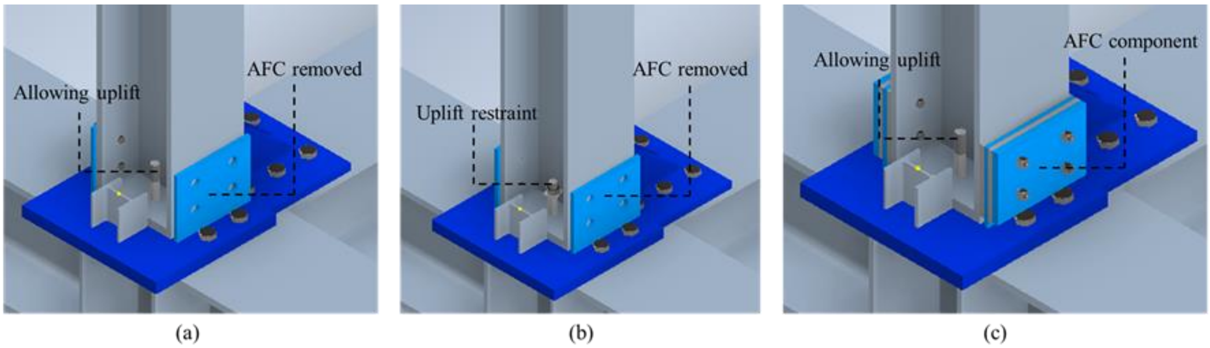


Fig. 7 – Column Base (a) Free to Rock, (b) Effectively Pinned and (c) Fixed at Concurrent Column

3.4 Consideration of structure stability

Due to the need of testing several different low-damage seismic resisting systems within one main structure, the key connection parts need to be exposed to allow changeover activities, which also means the floor slab does not cover where it usually covers in an actual building. The stability of the system needs to be carefully checked.

3.4.1 Stability of columns

Column twisting is also taken into consideration. The stability of the column due to seismic actions is normally ensured by the beam framing into the columns and by the torsional and the lateral restraint provided by the concrete slab being poured up against the sides of the column. However, in this project, all the columns of the seismic resisting system (C1 and C2) are isolated from the floor slab. This means the torsional and lateral stability of the columns due to seismic actions must be shown to be adequate by calculation. The stability of the column is initially checked to resist the following:

- 2.5% of $N^*_{c,max}$ on the column in the global X and in the global Y directions. $N^*_{c,max}$ is the maximum force generated by the seismic loading which can be determined as the capacity design derived column compression force
- Twist of the column which can be determined by 2.5% of N^*_{max} taken back to the point of attachment of the beam onto the column providing resistance to that twist

However, due to the unusual demands on the corner columns (concurrent columns) and lack of normal system restraint, the stability of corner columns is further checked under 5% of $N^*_{c,max}$.

3.4.2 Modified flexible endplate connection at B1 to C1

For the columns C1 adjacent to SHJAFCs, the lateral restraint can be provided by the beams framing in from each direction. The twist restraint can be developed through the top flange plate of the SHJAFCs in beams B3 (see also Figure 4).

The most critical case occurs at the corner column where a perfectly pinned connection and flexible endplate connection meets. The twist restraint is improved from the following parts:

1) Extended full depth endplate (see Figure 8)

   Moment resistance of detail (b) will be stronger than detail (a) due the change of the dimension of the endplate, which is negligible, and the connection can still be considered as effectively pinned.

2) Extended vertical stiffener of the top gusset plate (see Figure 4)

3) Bolts installed at extended column web stiffener acting as beam lateral moment restraint (see Figure 4)

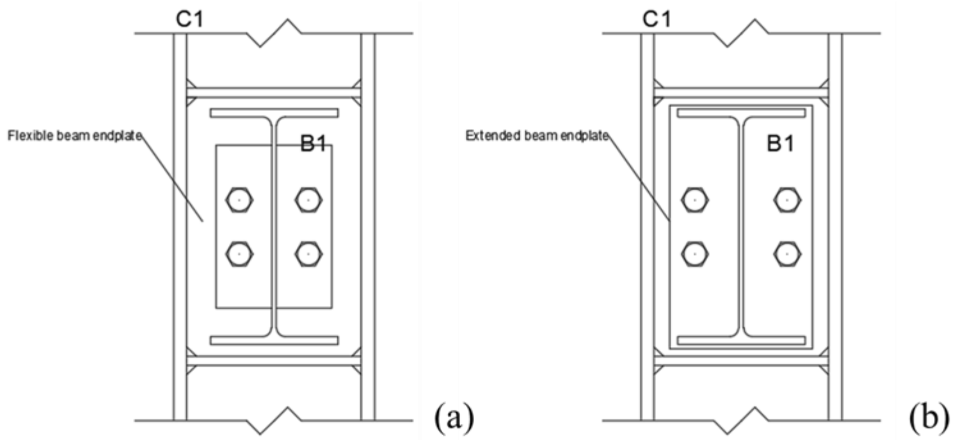


Fig. 8 –Beam to Column Connection with (a)Conventional and (b) Extended Flexible Endplate

3.4.3 Weld side plate connection at B4 to C2

The conventional weld side plate provides limited torsional restraint, especially for this case where the floor slab is not fully around the column. The column web stiffener adjacent to the beam top flange is extended with 3 rows of bolts installed to connect with the beam top flange as shown in Figure 9.

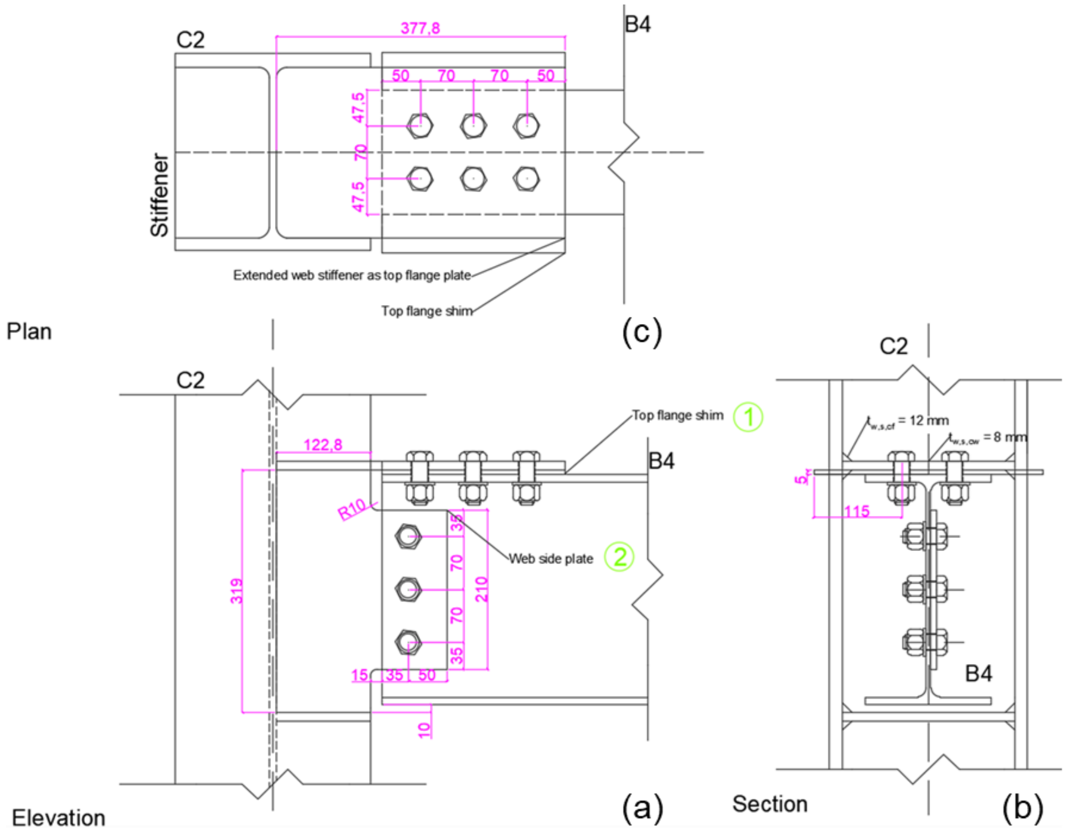


Fig. 9 – (a) Elevation, (b) Section and (c) Plan View of Weld Side Plate Connection at B4 to C2

4. Conclusions

This paper presents the current design and detailing of proposed structure incorporating both SHJAFCs in MRF and SFC braces in CBF system, addressing the need for conducting full scale shake table testing of such building system. Design considerations which enable a number of structure configurations to be tested with one basic frame are reported. The testing is expected to be conducted in August 2020 at ILEE facilities, Shanghai, China. This test aims at providing an exemplar of how economic resilient technology may protect the whole building against severe earthquakes.

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