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PROOF OF CONCEPT TESTING OF NARROW STEEL PLATE SHEAR WALL WITH TENSION BRACING FOR RAPID SEISMIC REAHABILITATION

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ABSTRACT

The use of a thin steel plate as a supplemental lateral load resisting system for relatively small seismic rehabilitation projects is described. The plate and surrounding boundary elements are installed in the middle of the bay, separate from existing columns. The proposed geometry intends to eliminate the need to strengthen the existing columns, as these elements typically would have been designed for the combined forces of gravity and wind only. The main objectives of the system are to reduce the influence on existing frame systems, to minimize construction work and impact on the environment, and to provide adequate energy dissipation with enhanced deformation capacity. The system performance was evaluated through two half-scale scale tests. The system achieved stable hysteretic behavior without showing strength deterioration until large story drifts were reached. As intended in design, the damage was solely concentrated in the thin steel plate and boundary elements.

Introduction

The development of a seismic rehabilitation strategy which is rapid and aimed at relatively small rehabilitation projects is described. Current rehabilitation strategies are intended for large-size rehabilitation projects, mostly multi-story mixed-use buildings, and are not necessarily optimal for small size projects. The proposed strategy adopted a unique approach to design supplemental systems using tension-only elements. The key features of the tension-only system includes (1) elimination of undesirable global and local buckling, (2) rational implementation of a strict capacity design (over-strength is known or capped), (3) being scalable and adaptable to many bay geometries, (4) use of simple connection with rapid and adjustable installation, and (5) minimal disruption to occupants. Such rehabilitation strategies are also suitable for a multi-staged incremental seismic rehabilitation strategy proposed by FEMA (e.g. FEMA, 2002), where a series of discretized actions can be made to coincide with regularly scheduled building repairs, and maintenance or capital improvement so that both investment and losses due to business

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interruptions are minimized.

A steel thin plate attached to exiting framing as a shear wall works as a tension-only element by developing tension field action after the onset of global shear buckling. Such a system, labeled a Special Plate Shear Wall (SPSW), is lighter and more ductile than a reinforced concrete shear wall and is applicable for new design or retrofit projects (Astaneh-Asl, 2001). Addition of a properly designed and detailed unstiffened thin steel plate to a steel moment frame can provide the system with a substantial increase in stiffness, load-carrying capacity, and energy dissipation (Sabelli, 2006). In order to utilize a thin steel plate as a supplemental lateral load resisting element in a rapid rehabilitation scheme, an alternative geometry where a plate with surrounding boundary elements is installed at the middle of the bay, separate from existing columns, is proposed herein. The proposed geometry intends to reduce the need to strengthen the existing columns and minimize installation problems which impact on indirect construction costs such as tenant disruption in occupied buildings; fire safety and hazardous material issues: relocation of occupants; overnight work fees; and noise problems. In most cases, these columns would have been designed for the combined forces of gravity and wind only and will not be able to provide the necessary resistance associated with a full bay shear wall.

The main objectives of the proposed system are to: i) reduce influence on existing frame systems, ii) minimize construction work and impact to environment, and iii) provide adequate energy dissipation capacity with enhanced deformation capacity. Such a system, developed within a rapid rehabilitation scheme, can have sustainability benefits both in terms of providing a more resilient building stock for the community as well as minimizing environmental and economical impacts and social consequences during the rehabilitation project.

Narrow Steel Plate Shear Wall with Tension Bracing

Proposed Geometry

In a conventional special plate shear wall system (SPSW), inward flexural forces induced by the tension field action in a thin steel plate are resisted by the flexural bending of beams and columns. When this system is applied in the seismic rehabilitation for which only gravity forces had been considered originally, the existing columns, known as vertical boundary elements (VBEs), will not have enough flexural resistance and will be susceptible to deterioration due to local buckling of the flanges. These columns are subjected to the axial and shear force induced by the overturning moment as well as the inward flexural force induced by tension field action in the infill panel. Once the failure mode of a SPSW system is governed by the capacity of VBEs, only a negligible increase in system strength is achieved and the excess plate material used is wasted. Lubell *et al*. (2000) reported such test results, where inward flexure of boundary elements resulted in an hourglass effect with only a limited area of tension field action developing in the infill panel. Behbahanifard *et al*. (2003) also reported test results which involved severe damage in VBEs due to local flange buckling. Qu and Bruneau (2008) reported on flexibility limits for VBEs design by reviewing the derivation of a flexibility factor in plate girder theory and how that factor was incorporated into current codes. Based on this review, they developed analytical models for preventing shear yielding and estimation of the out-of-plane buckling strength of VBEs of SPSWs.

To follow a capacity design approach, where damage is limited to primary energy dissipating components, the proposed shear wall system is installed apart from existing columns

[Fig. 1]. In this system, additional relatively weak, pin-ended VBEs are supported by tension bracing elements and provide enough strength and stiffness relative to the infill steel plate. This results in efficient use of the stiffness and strength of the thin shear wall and avoids strengthening of the existing beams at the location where the VBEs are attached. In this configuration, the tension bracing rods attract a large amount of the inward force in the VBEs and transfer them to beam elements of existing framings. A proper design for strength and stiffness of the tension rods enables the VBEs to remain in the elastic range until the infill panel reaches its shear yielding strength.

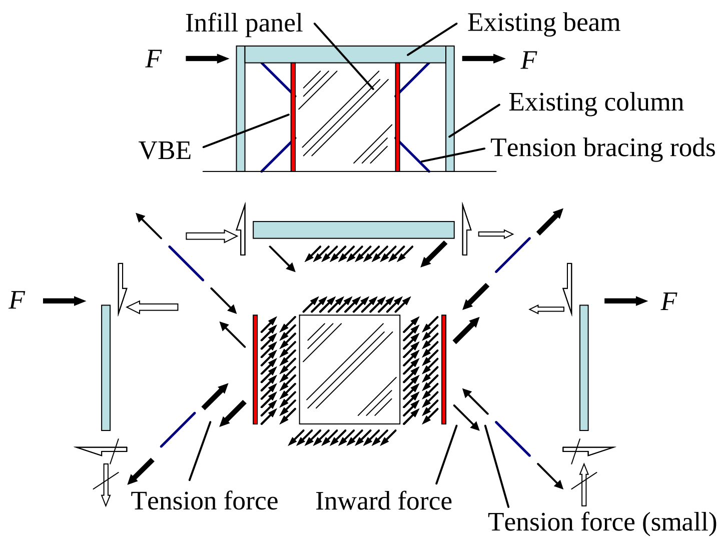


Figure 1. Concept and free-body diagram of Narrow Steel Plate Shear Wall with Tension Bracing (NSPW-TB).

Design Approach

A main design constraint for the prototype system is the requirement that yielding of the infill panel should occur prior to yielding of the boundary elements. The VBEs are subjected to inelastic deformation later due to the inward flexural force induced by the tension field developed in the infill panel. The design of the VBE requires an iterative procedure since its behavior interacts with the behavior of the tension-only rod and the local geometry of the arm. The strength and stiffness of the VBEs greatly affect the global behavior of the system while the behavior of the Horizontal Boundary Elements (HBE) does not influence the global behavior as long as the VBEs are designed to be stronger than the infill panel. The proper design of the HBEs is a relatively easy task since they are directly attached to the top and bottom beams of a frame. The tension-only elements are designed to remain elastic until very large deformation.

The performance goal of the prototype was set to achieve a total system shear strength of about 700kN, which is approximately equal to the shear force carried by three columns with standard section in low-to-mid size steel moment resisting frames when columns were assumed to have fixed-fixed end conditions; one column with a Japanese standard section, H-300x300x10x15 (section modulus, Z_x = 1350cm^3, roughly equivalent to an American W12x58 section), can sustain approximately 236kN shear force if it reaches its plastic moment capacity (M_p) at both ends, if the expected yield strength is taken as 305MPa (1.3 times its nominal value) and the story height of a frame to be rehabilitated is assumed to be 4m (clear story height = 3.5m). The aspect ratio of the infill panel is arbitrarily taken as 4:3 (height to width).

Test Specimen

The prototype was scaled to approximately 50% in a dimension for a proof-of-concept testing due to size limitations of the existing load frame. The target shear strength of the specimen was scaled down to 25% of the prototype strength, 175kN. Fig. 2(a) shows the geometry of the specimen determined based on parametric analyses. In the analyses, the infill panel was modeled using a strip model in which an infill panel is represented by a series of inclined pin-ended tension-only members (Thorburn *et al.*, 1983; Driver *et al.*, 1997; AISC, 2007). The thickness of the infill panel was selected as a 1.6mm thin plate of low carbon mild steel (SHPC) with a dimension of 1250mmx1673mm. The required section for VBE was defined as CT-75x100x6x9 (SS400, I_x = 51.7cm^4, Z_x = 8.8cm^3). The section for HBE was CT-87.5x175x7.5x11 (SS400, I_x = 115cm^4, Z_x = 15.9cm^3). The required tension-rods were M16 with at yield strength of 222kN; two tension rods were placed at each diagonal, sandwiching the VBEs with the inclination of 45 degrees; eight tension-rods were used in total. The other ends of the tension rods are anchored to beams at the position outside of protected zones (one half of beam depth away from column surfaces). The arms (brackets) shown in the figure were designed with the intention for limiting force demand in the tension rods through their rotational movement involving inelastic deformation at their connection to the VBEs. The bracket consisted of mild steel plates (SS400) with thickness of 6 mm and 9 mm. The connection between the VBE and the HBE was assumed as a pinned connection and the corners of the infill panel were cut to avoid interference with the connectors. A series of small holes (ϕ = 10mm) along the edge of the infill panel were used for welding inside of the holes to attach the infill panel and boundary members.

Design Flowchart

The flowchart shown in Fig. 2(c) schematically describes the design procedure. Given the geometry of a frame and a target shear force (V_n) for the proposed system, the approximate thickness of an infill panel is estimated using the formula specified in the U.S. and Canadian seismic codes for the calculation of the nominal shear strength of infill panel for SPSW system (CSA, 2006; AISC, 2007). This formula includes an overstrength factor of 1.2;

$$\boldsymbol{t_w = V_n / 0.42 F_y L \sin 2\alpha} \qquad (1)$$

where, F_y = yielding stress of the infill panel, t_w = thickness of the infill panel, L = distance between the centerlines of VBEs and α = the inclination of the tension field. It should be noted here that this equation is valid when the aspect ratio of the infill panel is larger than 0.8 and the boundary elements satisfy the specified stiffness limitation (Rezai, 1999; Qu and Bruneau, 2008). The shear strength of the infill panel is used as a rough estimate of the total shear strength of the system since the pin connected boundary elements do not carry shear force. The inclination of the tension field is assumed to be 45 degrees.

Once the thickness of the infill panel has been selected, a trial section is picked for the VBEs. In the first nonlinear static pushover analysis, the behavior of tension-rods is assumed to be elastic. Reasonable dimensions are assigned to tension-rods and arms. Using the analysis results, the requirement that the infill panel yields prior to the VBEs is checked. This judgment is rather arbitrary since not all areas of the infill panel yield even with very stiff and strong VBEs. For the design of the prototype, the criterion adopted was whether most of the middle part of the infill panel had yielded or not. When the criterion is satisfied, the diameter of the tension-rods is selected based on the force at 1% story drift. A second analysis is then executed to see if the total shear strength of the system is acceptable. If the error is within a reasonable tolerance, the dimension of the arm is determined in an iterative manner to ensure that the tension-rod remains elastic to at least a 2.5% story drift; otherwise the thickness of the infill panel need to be updated.

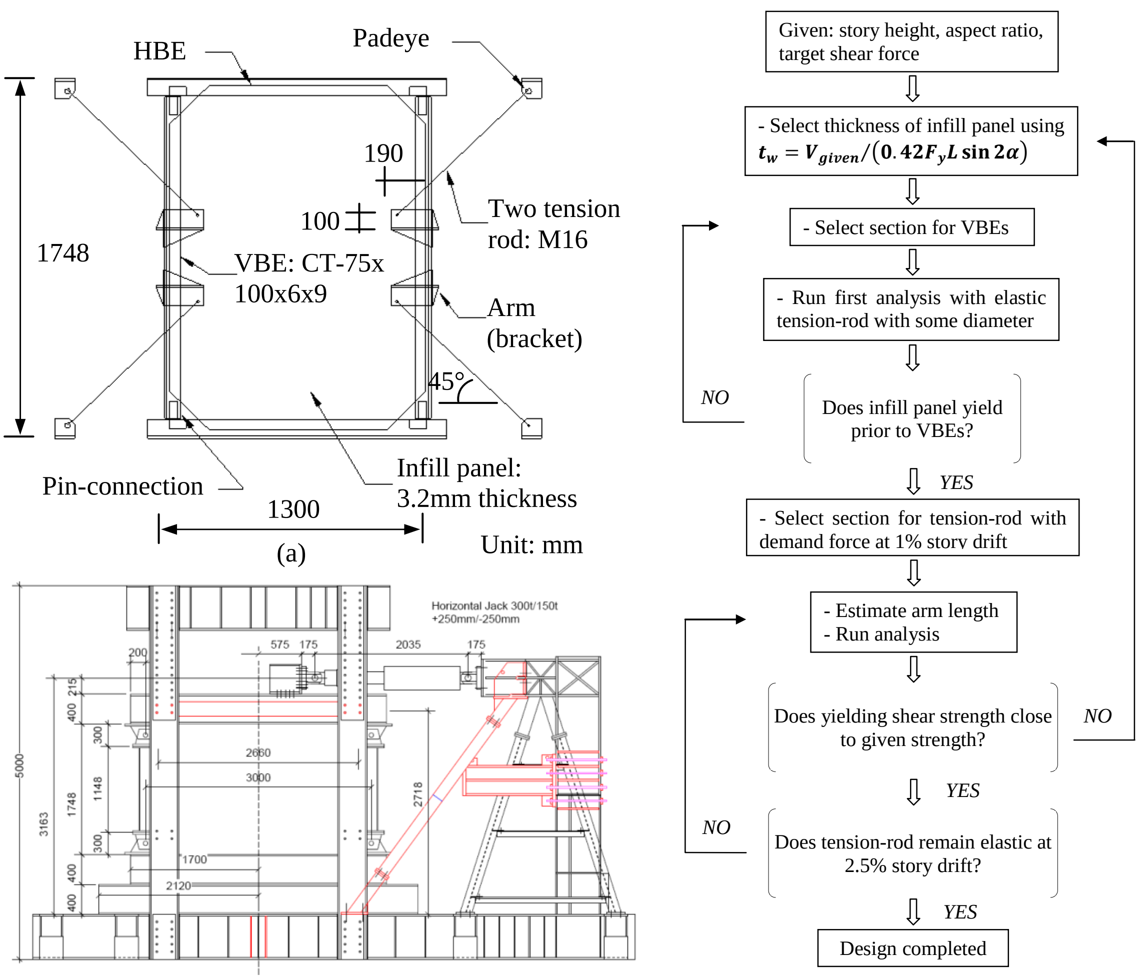


Figure 2. Specimen design (a) geometry and components (b) reaction frame and testing setup (c) design flowchart.

Proof-of-Concept Testing

Testing Setup and Specimen Assembly

The performance of the proposed system was evaluated through large scale testing at the structural laboratory in Kyoto University, Japan. In the experimental program, two shear wall specimens were tested. One specimen had tension bracing (Specimen 1) and the other did not

(Specimen 2) in order to evaluate the effects of the bracing on the global and local behavior of the prototype. The test setup and specimens were fabricated using members and materials specified in Japanese standards. A new testing setup was installed into an existing load reaction frame at the structural laboratory in Kyoto University [Fig. 2(b)]. The testing setup is a portal frame with four pins at each corner and has an inter-story height of 1,748mm and a column centerline spacing of 3,000mm. The main components of the test bed were as follows: (a) top and bottom H-400x400x13x21 beams (b) two H-250x250x9x16 columns (c) four pin-clevis subassemblies with load carrying capacity of 900kN each (d) a fixed support for the actuator loading. The assembly is capable of applying a horizontal force more than 750kN which is determined by slip critical force at bolted connections. The deformation of the test setup was restrained to in-plane deformations using out-of-plane restrainers and guiding beams.

The typical assembly of a specimen was as follows: (1) Horizontal Boundary Elements (HBE) were attached to the top and bottom beams of the test setup with high strength bolts, (2) Vertiacl Boundary Elements (VBE) were pin-connected to HBEs with L-shape plates and F10T-M16 high strength bolts, and (3) an infill thin steel panel was welded to the HBEs and VBEs. For the specimen with tension-only bracing (Speciemen 2), the assembly continued as follows: (4) four steel brackets were installed to the VBEs using F10T-M14 high strength bolts after welding of the infill panel (5) four pad-eyes were installed to the top and bottom beams (6) eight tension-only braces, composed of a M16 steel threaded rod and a M16 steel turnbuckle, were connected the steel brackets and the padeyes.

The mechanical properties of the infill panel, the web of the HBE, the flange and web of the VBE were obtained from tensile coupon tests and summarized in Table 1. The shape of the coupons followed the Japanese Industry Standard (JIS Z2201, 2005).

Table 1. Mechanical properties

Name	Element	JIS Z2201 Coupon Shape	*Thickness* t (mm)	*Yield strength* σ_y (Mpa)	*Tensile strength* σ_u (MPa)	*Elongation* *EL* (%)
C-A	Infill panel	JIS 1B	1.60	201.9	329.6	34.4
C-B	VBE web	JIS 1A	5.61	328.9	438.1	28.0
C-C	HBE web	JIS 1B	6.88	343.4	482.1	19.9
C-D	VBE flange	JIS 1A	8.63	299.7	430.1	28.7

Loading Ptotocol and Measurement Plan

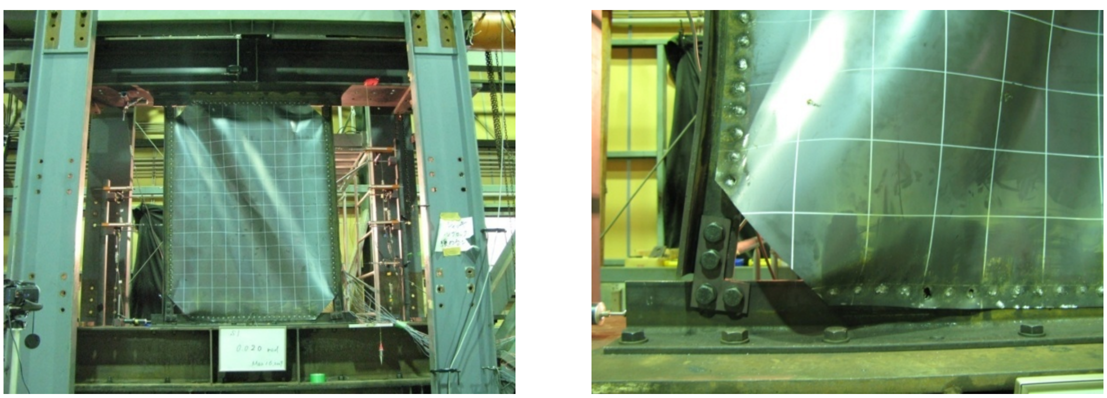
The loading protocol used in the test consisted of 3 cyles at story drifts of 0.375%, 0.5% and 0.075%, followed by 2 cylces at 1%, 2%, 3% and 4% story drift. The applied loading protocol was determined after a review of the loading protocols used in previous tests and the design guidelines (Vian and Bruneau, 2005; AISC, 2007).

The local deformation of the specimens were measured with twelve potentiometers and four LVDTs. The strain histories for the infill panel and the VBE were measured using rosette strain gauges and uniaxial strain gauges. For the specimen with tension bracing (Specimen 2), forces in the tension bracing were monitored using uniaxial strain gauges attached on the turnbuckles. The relationship between the strain and the axial load in the turnbuckles were calibrated from preliminary tension tests so that the turnbuckles could be used as load cells. The yielding strength of turnbuckles were obtained by tensile loading tests after the tests were completed.

Test Results and Discussion

Specimen 1

Fig. 3 shows the deformed shape of Specimen 1 at the 0.02rad. cycle loading. Global buckling of the infill plate took place immediately in the first load cycle with the amplitude of 0.00375rad. with the development of tension-field action in the infill plate. The buckling of the plate involved a single low tone sound following a series of high tone sounds accompanied by vibration of the infill plate. The main wave line in the global buckling ran exactly diagonally across the corners in the infill panel. The infill plate at the corner was subjected to compressive stress and was deformed out-of-plane. This out-of-plane deformation caused the weld metal at the boundary to be loaded under combined tension and shear stress. The two welds at the left bottom and the right top of the boundaries fractured during the second half cycle of the 0.01rad. At the 0.02rad. loading cycle, the VBEs deformed inelastically due to the inward force generated by the tension-field action of the infill panel. At the left bottom corners of the infill panel, most of fractures occurred in the welds while some of them occurred in the base material (infill plate). Under further loading, fractures at the boundary connection propagated rapidly, with the bottom boundary of the infill panel almost disconnected at the end of the 0.04rad. loading. After the scheduled loading cycles were completed, a monotonic loading was applied to the specimen until the bottom boundary of the infill panel completely disconnected at the amplitude of 0.053rad.

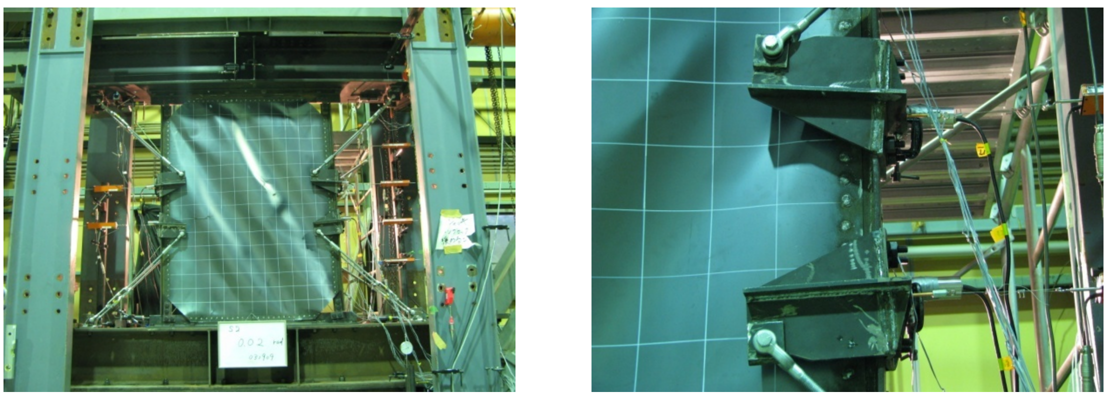


(a) (b)

Figure 3. Behavior of S1 (a) deformation at 0.02 rad. (b) condition of left bottom corner

Specimen 2

Fig. 4 shows the deformed shape of Specimen 2 at the 0.02rad. cycle loading. As for Specimen 1, global buckling of the infill plate took place immediately in the first load cycle at the 0.00375rad. with the development of tension-field action in the infill plate. However, the deformed shape exhibited more short wave lines for Specimen 2 than for Specimen 1, which indicated that the buckling mode was higher for Specimen 2 than for Specimen 1. The main wave line in the global buckling ran diagonally across the corners in the infill panel while two other wave lines initiated from the location of the steel brackets. Again, the buckling of the plate was accompanied by a single low tone sound following a series of high tone sounds with vibration of the infill plate. The rotation at the HBE-VBE connection was smaller than that in Specimen 1, and the deformation at the corners of the infill panel was not large. As deformations became larger, the number of wave lines increased. This is because the tension-only bracing started to resist against the inward deformation of the VBEs and the tension load paths in the infill panel changed. The deformations at the corners of the infill panel were still limited as a benefit of the tension bracing. No damage was observed at the boundary connections at end of the 0.01rad. loading cycle. At the 0.01rad. loading cycle, large deformations were observed in the steel brackets. Yielding of the steel brackets and the VBE occurred due to the large local force input from the tension-only braces in the 0.015rad. loading cycle [Fig. 4(b)]. The steel bracket located at left bottom fractured during the 0.03rad. loading cycles. This undesirable fracture resulted in a slight buckling of the tension bracing located at the bottom left. The yielding of all steel brackets affected the mode of the inelastic buckling and the deformed shape of the infill panel. This increased the force input in the corner of the infill panel and resulted in fractures at the bottom boundary connection. Under further loading, fractures at the bottom boundary increased excessively.



(a) (b)

Figure 4. Behavior of S2 (a) deformation at 0.02 rad. (b) condition of brackets

Hysteresis Behavior

The global behavior before the specimens degraded are shown in Fig. 5. The strength of Specimen 1 reached its maximum value, 108kN, during the first loading cycle at 2.0% story drift. The strength started to deteriorate slightly earlier (1.5% story drift) for the negative loading due to the fractures that propagated at the left bottom of the boundary connections. The strength of Specimen 2 reached its maximum value, 175kN, during the first loading cycle at 3.0% story drift. The strength stopped increasing earlier in the positive loading due to early, undesirable yielding of the steel brackets at their left top and right bottom locations. The design of the steel brackets should be reexamined to prevent this early termination of the strength increase. At the 3% story drift, the strength deterioration in the positive loading direction became significant due to the fracture of the steel brackets and the development of the yielding line in the flange of the VBEs. In the negative loading direction, the deterioration was milder where the steel brackets deformed in a rather ductile manner without fracture at the welds. When the two specimens were compared, the maximum strength was 62% larger in specimen 2.

The ductility of the specimens, defined as the deformation where strength deteriorated to 80% of the maximum strength divided by the deformation at yield, were roughly estimated as 10 (2.5%/0.25%) for Specimen 1 and 14 (3.5%/0.25%) for Specimen 2. The yield and maximum strength of both systems were around 10-15% smaller than the analyses prediction mainly due to damage in the welds at boundary elements which were treated as rigid in the blind analyses.

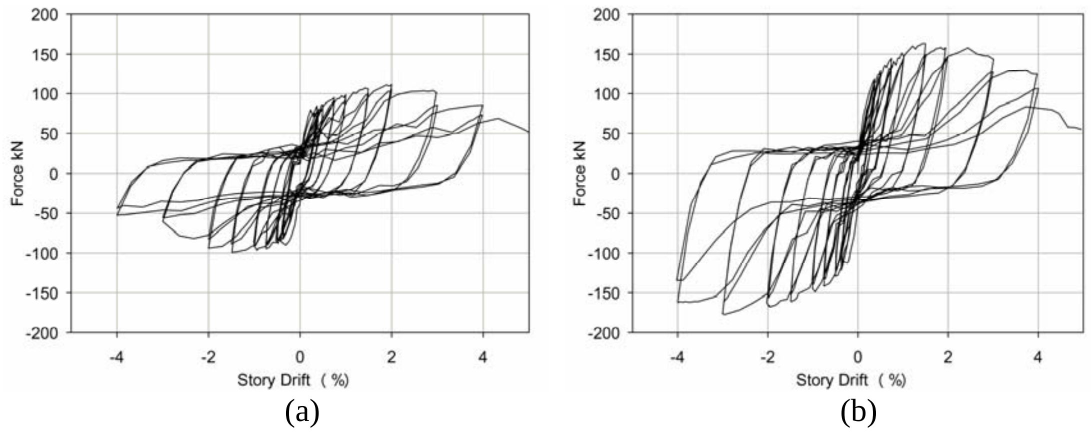


Figure 5. Global behavior until end of loading (a) specimen 1 (b) specimen 2

Conclusions

Addition of a properly designed and detailed unstiffened thin steel plate to a steel moment frame can give the system a substantial increase in stiffness, load-carrying capacity, and energy adsorption. In order to utilize a thin steel plate as a supplemental lateral load resisting system for relatively small seismic rehabilitation projects, a geometry where a plate with surrounding boundary elements is installed separately from existing columns is proposed. The proposed geometry intends to minimize installation impact and to reduce the need to strengthen the existing columns. The prototype geometry was defined following the proposed design approach and design flowchart. The scaled specimen was examined through a proof-of-concept testing. The key findings and conclusions are summarized below:

- Specimens with and without tension bracing both showed nonlinear hysteresis curves immediately after the loading started. This was mainly caused by the global buckling of the infill plate.
- The yield strength and initial secant stiffness were increased by approximately 45% by the addition of tension braces. Both systems showed pinched behavior after the second loading cycle at the 0.00375 rad. The pinching was more severe for second and/or third cycles with the same amplitude. With the presence of the tension bracing, the shear strength at the 0.01 rad. and at peak increased by 67% and 62%, respectively.
- Strength started to deteriorate much later with the presence of tension bracing. In the case without the tension bracing, the strength started to deteriorate slightly at the 1.5% story drift due to the fractures that propagated at the left bottom of the boundary connections. In the case of the tension bracing, the strength deterioration in the positive loading direction became significant at the 3% story drift, due to the fracture of steel brackets and the development of the yield lines in the flange of the VBEs.

- The ductility of the specimens, defined as the deformation where strength deteriorated to 80% of the peak strength divided by the deformation at yielding, were roughly estimated as 10 and 14 for specimens with and without tension bracing, respectively.

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