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Seismic Retrofit of Welded Steel Moment Connections with Highly Composite Slabs

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Abstract

In the 1994 Northridge earthquake, connection damage initiated from the beam bottom flange was prevalent. The presence of a concrete slab and resulting composite action was speculated as one of the critical causes of the prevalent bottom flange fracture. Close review of past experimental studies recently conducted by the author clearly indicated that conventional seismic steel moment connections with highly composite slabs were much more vulnerable to the bottom flange fracture. In this study, seismic retrofit schemes are presented for welded steel moment connections with highly composite floor slabs typical of existing steel moment frames in Korea. Because top flange modification of existing beams is not feasible due to the presence of a concrete floor slab, bottom flange modifications by using welded triangular or straight haunches, and by beam web strengthening with heavy shear tab were cyclically tested and analyzed. Test results of this study showed that the retrofit schemes used is effective in eliminating the detrimental effects associated with high composite action and can ensure excellent connection plastic rotation exceeding 4% rad. Side effects as a result of each retrofit scheme as well as design recommendations are also presented.

Keywords: Seismic retrofit; Welded steel moment connection; Slab effect; Heavy shear tab; Straight haunch; Triangular haunch

1. Introduction

Most of steel moment frame buildings have composite floor slabs which usually consist of steel beam, shear studs, metal decks, steel rebars or wire meshes, and topping concrete. As was previously discussed by the authors [1], understanding their influence on seismic performance is very important for both new construction and retrofit. Composite floor slab acts as diaphragm and shear studs are provided to help transfer diaphragm loads to the steel moment frame, to laterally support the beams, and to increase overall structural continuity. For gravity beams with simple connection at both ends, it is usual practice and more economical to design them as fully composite. In this case, an adequate number of shear studs must be provided to accomplish full composite action according to the well-established design procedure. It is the moment frame beams that much attention should be paid to. In the past design and construction practice in Korea, more shear studs than needed to transfer diaphragm forces were provided for steel moment frame beams. For example, the use of a pair of shear studs of 16~19mm diameter per deck rib (or about 200 mm on center) for moment frame beams was very common even when they were designed as non-composite beams, or when composite action was not explicitly considered in computing lateral stiffness and strength of the moment frame. Typical slab details in many countries appear to be empirical and vary somewhat depending on locality. For example, providing 19-mm diameter shear studs spaced 300 mm on center is a popular US west coast practice to transfer seismic forces from the slab to steel frame.

The effects of composite slabs on cyclic seismic behavior of welded steel moment connections received only limited attention before the 1994 Northridge earthquake and most of cyclic seismic connection tests were conducted for bare steel beam-column subassemblies. However, unprecedented and widespread brittle failures were observed in connections of welded steel moment frames in the 1994 Northridge earthquake. Numerous causes were conjectured as potentially contributing to the poor seismic performance of the pre-Northridge steel moment connections. Especially, the majority of reported damages occurred in the beam bottom flange [2]. Other than the flaws recognized in the welding and detailing issues like the use of low toughness welding electrode and notch-like condition present in the bottom flange, the upward shift of the neutral axis due to the unintended composite action between steel beam and concrete slab was speculated as one of the critical causes for the prevalent bottom flange fracture.

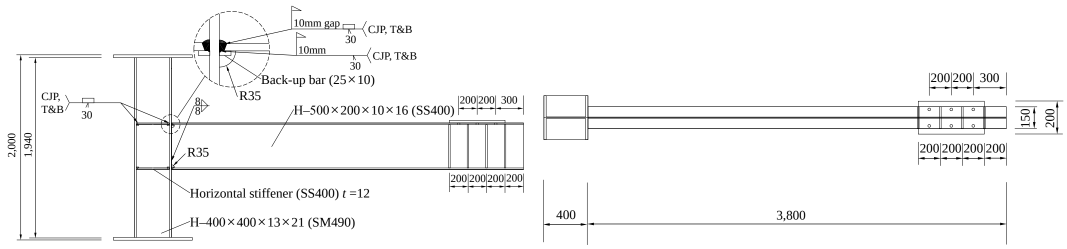
After the earthquake, the effects of a concrete slab on seismic performance were investigated by many researchers. However different observations and conclusions were made among different researchers for slab effect on seismic performance depending upon the details of beam-to-column connections and floor slabs. Recently the authors conducted a close review of extensive relevant experimental studies conducted after the 1994 Northridge earthquake [1]. Based on the review, the authors were able to draw the following conclusions:

i) When composite connections are composed of conventional welded steel moment connections (similar to the PN type) and relatively shallow beams with a high degree of composite action, composite slab effect is quite detrimental to seismic performance because of significant upward shift of the neutral axis under positive moment, often leading to premature fracture of the beam bottom flange.

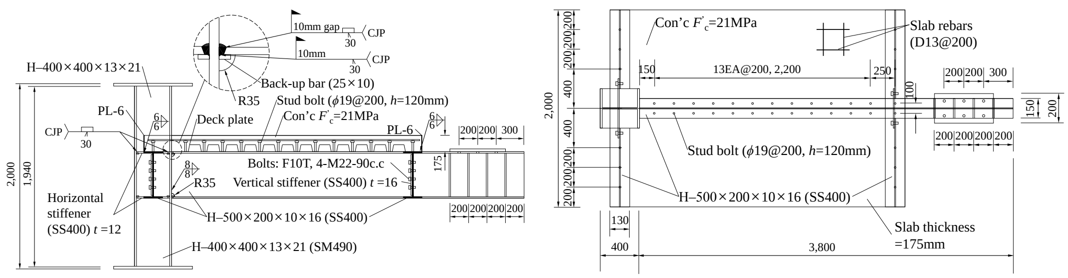
ii) Composite specimens with deep beams, a low degree of composite action, and connection modification by using RBS (reduced beam section) or haunch showed little or no negative effects due to the presence of a composite slab. The presence of a composite slab did not cause any significant shift of the neutral axis and increased strain demand on the bottom flange. The dominance of the beam bottom flange fracture was not observed. Instead, the composite slab was beneficial and helped improve seismic performance because local and lateral instability of the beams was decreased due to the bracing effect of the slab. However, several experiments also showed that composite slab effect could be detrimental to the bottom flange even in RBS connections when composite action is very strong.

iii) Connection improvement by using modification scheme like RBS or haunch provides a structural fuse at the critical section of the beam. In such connections with a promoted plastic zone, which can help relieve increased strain demand, the slab effect on the bottom flange is expected to be less critical than in PN type connections.

iv) The bottom flange in a composite beam is always more strained than that in a bare steel beam for a given beam rotation demand, and therefore welded seismic steel moment connections and concrete slab should be



(a) PN500



(b) PN500C

Fig. 1 – Side view and top view of PN500 and PN500C

designed and constructed to minimize the composite action, if possible, especially when shallow beams and conventional connections are involved.

In this paper, several seismic retrofit schemes to salvage conventional welded steel moment connections with highly composite floor slabs and their seismic performance are presented based on full-scale cyclic test results. A total of five full-scale specimens fabricated by using all-welded beam-to-column connection per the past practice in Korea were tested. By comparing with the cyclic performance of two benchmark test specimens, the detrimental effect of the composite action caused by the presence of the floor slab was clearly shown. Further, the pros and cons as well as the effectiveness of the three retrofitting schemes involving triangular or straight haunches and heavy shear tab were analyzed and discussed in details.

2. Test Programs

2.1. Design of specimens

When modifying existing connections for improved seismic performance, the presence of a concrete floor slab often dictates modification in the bottom flange only. The bottom flange modification by using welded triangular haunch is well described in the AISC Design Guide 12 [3]. The bottom flange modification by using welded

Table 1. Test specimens

Specimen	Slab	Retrofit scheme
PN500	None	None
PN500C	Yes	None
PN500C-HST	Yes	Heavy shear tab
PN500C-SH	Yes	Straight haunch
PN500C-TH	Yes	Triangular haunch

straight haunch was also proposed based on the analytical and experimental studies by Lee and Uang [4] and Lee et al. [5].

Test specimens were designed to simulate one-sided (or exterior) moment connections. Five specimens were tested in this study as shown in Table 1 and Figs. 1 and 2. The steel beam and column sizes of all the specimens were identical. The grade of steel for the beams was SS400 with a specified minimum yield strength of 235 MPa; the steel used for the columns was SM490 with a specified minimum yield strength was 325 MPa. SN490 (or seismic steel recently developed in Korea) was used for retrofit steel plates. The tensile test results based on the average of three coupons taken from the flanges and the webs are reported in Table 2. Each specimen consisted of H-500×200×10×16 (SS400) beam and H-400×400×13×21 (SM490) column. All specimens was fabricated by using all-welded beam-to-column connection per the past practice in Korea. A notch-tough welding electrode with a specified minimum Charpy V-Notch (CVN) toughness of 26.7 Joule at -28.9°C (20 ft-lb at -20°F) and a tensile strength of 490MPa was specified for flux-cored arc welding in all specimens.

Nominally identical 175 mm-thick composite slab with rebars (13mm diameter @200 in both directions) and 100 mm concrete topping on 75mm-deep corrugated steel deck was provided to all the specimens except PN500. The average 28-day compressive strength of three cylinder specimens taken from the concrete topping was $f_c' =$ 21MPa. The flutes of the deck were oriented perpendicular to the beam axis. A pair of shear studs of 19mm diameter per deck rib (or 200 mm on center) were attached to the beam top flange of all the composite slab specimens following the past practice in Korea.

The specified minimum tensile strength of shear stud (F_u) was 400MPa. Two cross beams (with one row of shear studs of 19mm diameter 200mm on center) were also provided to simulate the presence of the gravity beams. The shear studs provided corresponded to 92% fully composite action. The degree of composite action was calculated per Eq. (1) as the shear strength of shear studs (ΣQ_n) divided by tensile yielding strength of the steel beam ($A_s F_y$). The shear strength of a stud was calculated according to Eq. (2) following the AISC Specification [6].

$$\text{The degree of composite action} = \frac{\sum Q_n}{A_s F_y} \tag{1}$$

Table 2. Measured tensile mechanical properties of steels

Specimen		Yield stress (MPa)	Tensile stress (MPa)	Yield ratio (%)	Elongation (%)
H-500×200×10×16 (SS400)	flange	308	442	69	24
	web	306	442	69	26
H-400×400×13×21 (SM490)	flange	373	523	71	29
	web	370	531	70	24
Heavy shear tab		412	565	73	24
Straight haunch (SN490)	flange	375	545	69	23
	web	379	546	69	25
Triangular haunch (SN490)	flange	363	546	66	24
	web	367	550	67	23

$$Q_n = 0.5A_{sa}\sqrt{f_c'E_c} \le R_g R_p A_{sa} F_u \quad (2)$$

where ΣQ_n = sum of nominal shear strength of shear studs, A_{sa} = cross-sectional area of shear stud, f_c' = specified compressive strength of concrete, E_c = modulus of elasticity of concrete, R_g = coefficient to account for group effect., R_p = position effect factor for shear studs, F_u= specified minimum tensile strength of a shear stud.

Table 3 summarizes the relative strength of the beam, the panel zone and the column of specimen PN500 based on the measured yield strength of steels. The column was much stronger than the beam, but the panel zone strength including the column flange contribution factor was comparable to the beam strength and thus varying degrees of panel zone yielding were expected among specimens because of different connection details and different shear demand on the panel zone under cyclic loading.

Bare steel specimen PN500 was nominally identical to specimen PN500C except the absence of a composite slab. Both may be thought as benchmark specimens. Triangular and straight haunch specimens were designed according to the procedure in the AISC Design Guide 12 [3] and by Lee-Uang [4] and Lee et al. [5], respectively. The unique load transfer mechanism of haunch connections and the design procedures for haunch connections are not presented here because of space limitations. The motivation of including the heavy shear tab specimen PN500C-HST is as follows. As mentioned previously, when modifying existing connections for improved seismic performance, the presence of a concrete floor slab often dictates modification in the bottom flange only. However, strengthening existing beam web by using heavy shear tab may be a viable alternative, when even the bottom flange modification is not feasible. As was shown by Lee [7] and Lee and Kim [8], the load transfer mechanism close to the beam to column welded joint is completely different from the engineering beam theory, and the flange near the welded joint should transfer a significant portion of the beam shear as well and is overstrained. Further, the flange near the welded joint becomes brittle due to the welding heat affection and the tri-axial strain restraint condition there. Thus, the purpose of strengthening the beam web with heavy shear tab is two-fold; i) to reduce tensile strain demand on the beam flange by increasing the plastic section modulus of the retrofitted beam web, and ii) to push the plastic hinging away from the brittle welding area to the inside “ductile’ area.

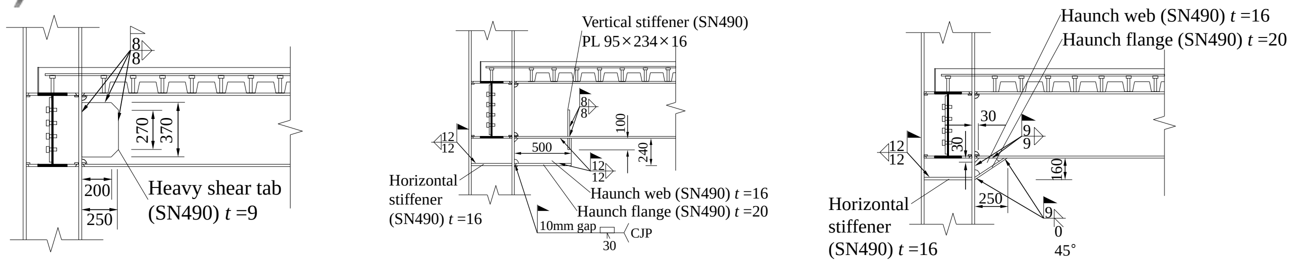
Table 3. Relative strength of beam, panel zone and column of PN500

Beam (SS400)	Column (SM490)	Panel zone (SM490)	SC-WB requirement	PZ requirement
$M_{pb}=F_{yb}Z_b$	$M_{pc}=F_{yc}Z_c$	$R_v=0.60F_{yc}d_ct_w\times[1+(3b_{cf}t_{cf}^2/d_cd_bt_w)]$	$\Sigma M_{pc}/\Sigma M_{pb}$	$R_v/(M_{pb}/d_b)$
671.4kN-m	1368.9kN-m	1400.6kN	2.04>1.0	1.04

Note: All the strengths were calculated using the measured yield strength reported in Table 2; the panel zone strength R_v was computed per Eq. (J10-11) in the 2010 AISC Specification.

2.2. Test setup and loading

The specimens were mounted to a strong floor and a strong wall. An overall view of the test setup is shown in Fig. 3. The column, which was pinned at the top and bottom, were 3,500 mm high. The distance between the column centerline and the actuator loading point was 3,500mm. Thus, the beam tip displacement corresponding to 1.0% story drift was 35mm. Lateral bracing was provided at a distance of 2,500mm from the column centerline. The specimens were tested statically according to the AISC cyclic seismic loading protocol [7]. The test specimens were instrumented with a combination of displacement transducers and strain gages to measure global and local responses.



(a) PN500C-HST (b) PN500C-SH (c) PN500C-TH

Fig. 2 – Details of retrofitted specimens

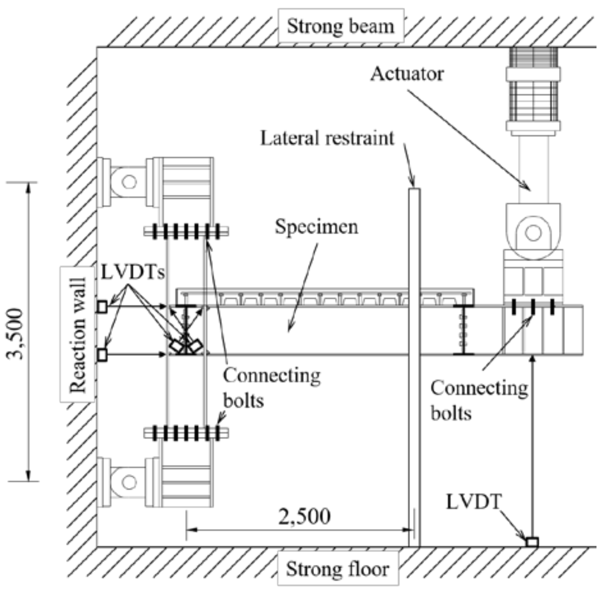
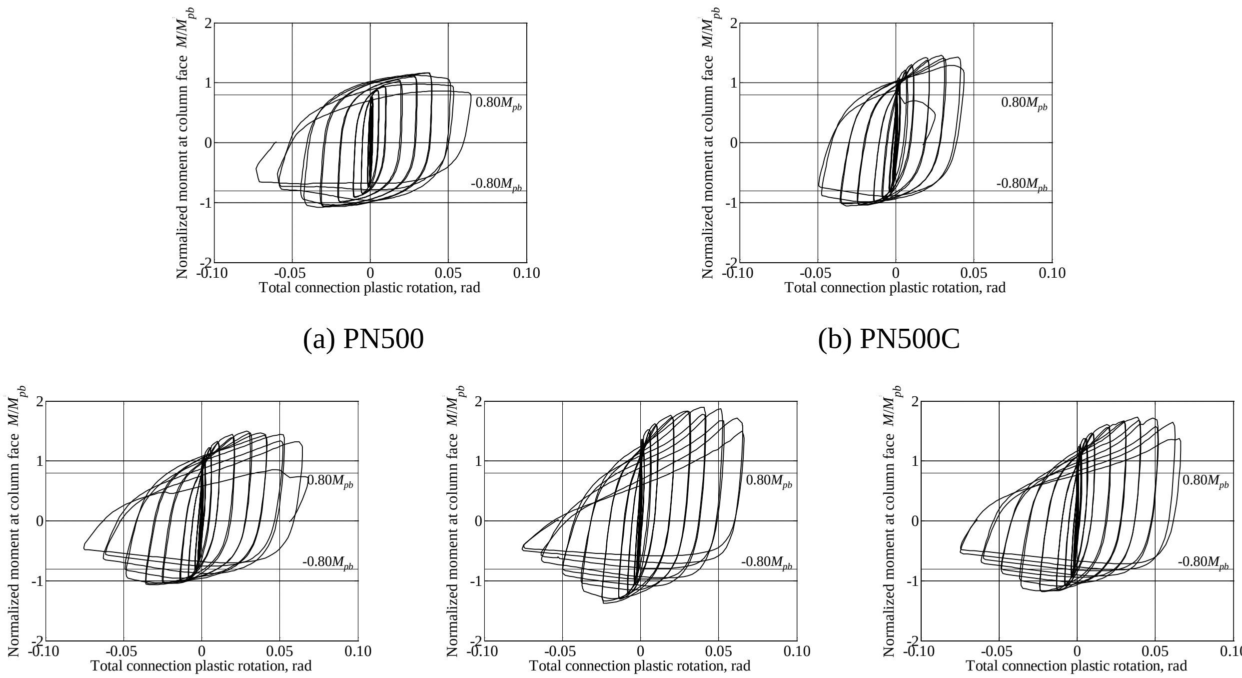


Fig. 3 – Typical test set-up

(a) PN500 (b) PN500C (c) PN500C-HST (d) PN500C-SH (e) PN500C-TH

Fig. 4 – Photographs showing plastic hinge region after completion of testing



(a) PN500

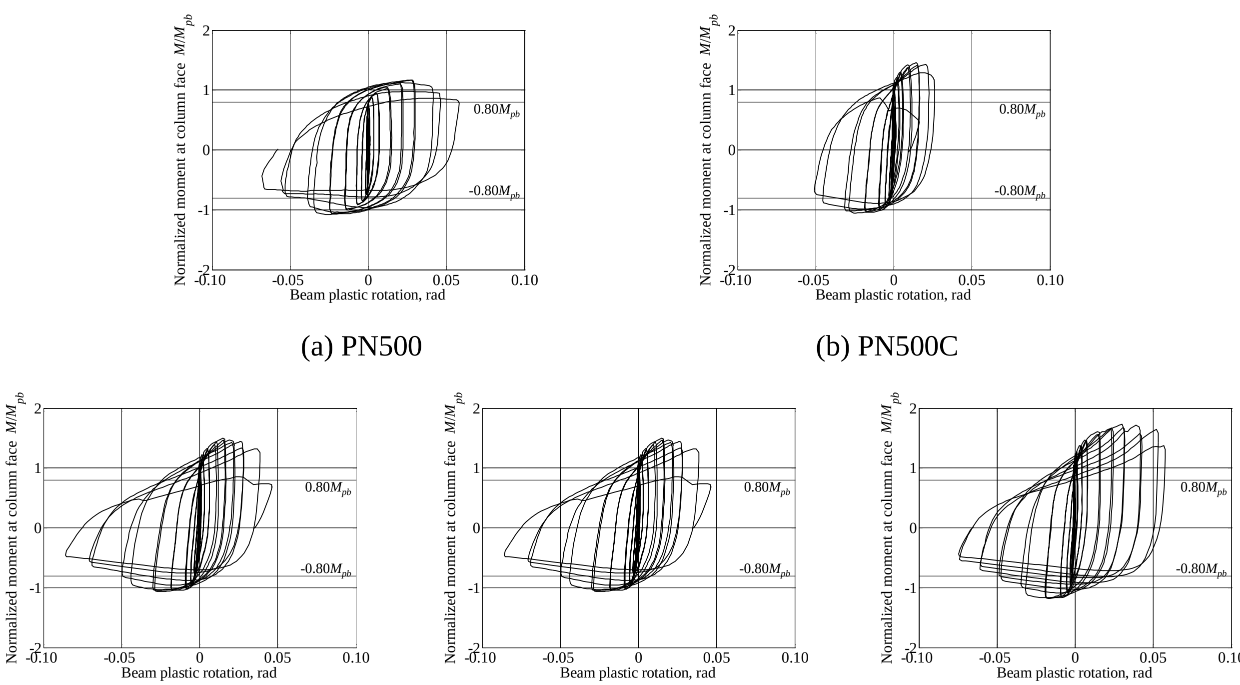
(b) PN500C

(c) PN500C-HST

(d) PN500C-SH

(e) PN500C-TH

Fig. 5 – Total connection plastic rotation



(a) PN500

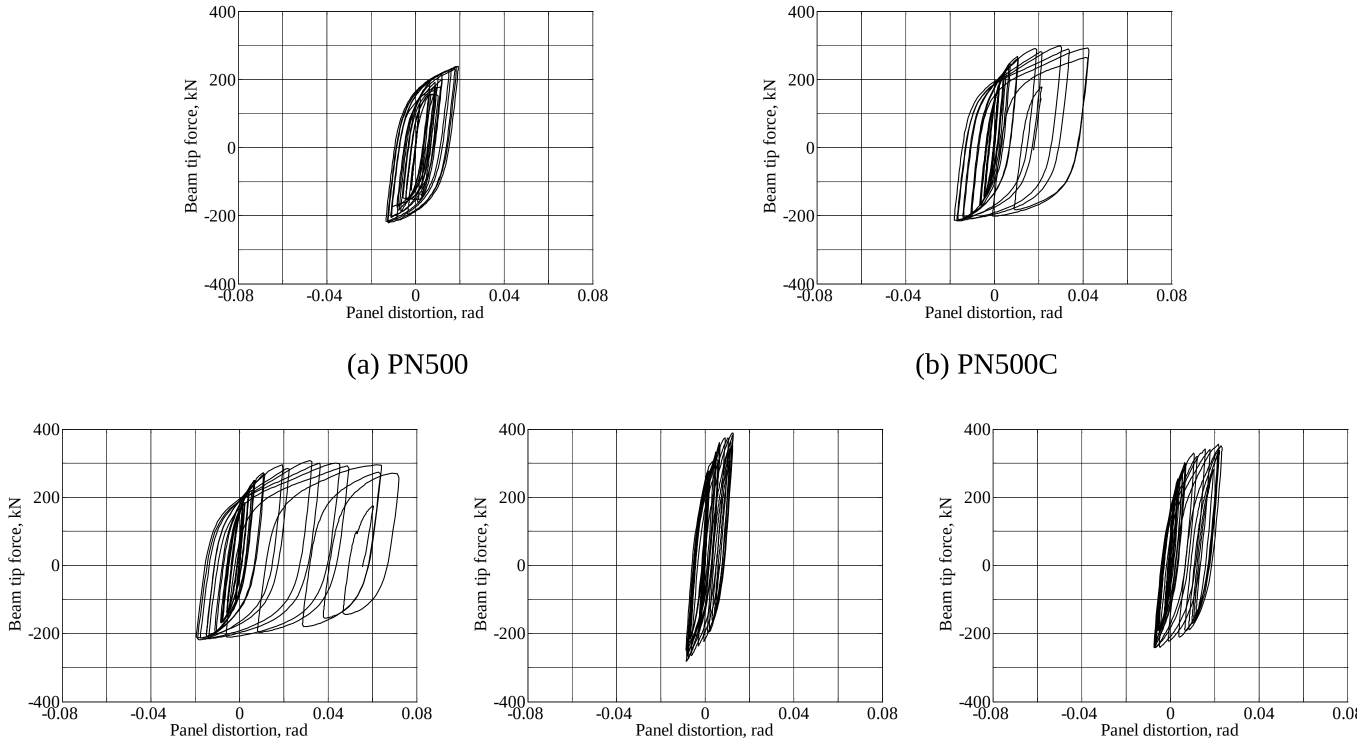
(b) PN500C

(c) PN500C-HST

(d) PN500C-SH

(e) PN500C-TH

Fig. 6 – Beam plastic rotation



(a) PN500 (b) PN500C

(c) PN500C-HST (d) PN500C-SH (e) PN500C-TH

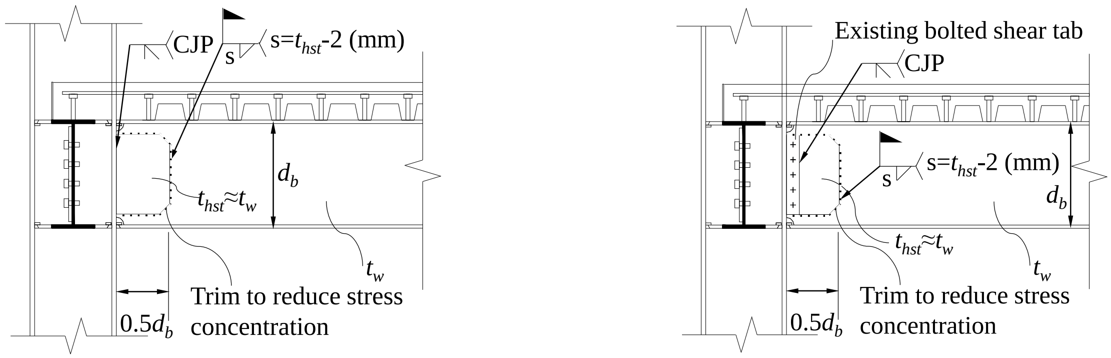
Fig. 7 – Panel shear distortion of specimens

3. Test results and discussions

Figure 4 shows deformation of the plastic hinge region near the connection. Figure 5 summarizes the total connection plastic rotations (or plastic story drift ratios) of the five specimens. First all, specimens exhibited excellent plastic rotation capacity exceeding 4% rad. Relatively shallow beam depth (500mm) and all-welded quality welding with notch-tough welding electrode are deemed to have contributed to this superior performance. However as can be observed in Figs. 5a and 5b, the composite slab specimen PN500C showed seismic performance inferior to the bare steel counterpart PN500 because of low cycle fatigue fracture of the beam bottom flange at 5% plastic drift cycle. Strain measurement showed that the natural axis of PN500C was shifted upward by about 10% of the beam depth (not shown here). As a result of strengthening the connection with bottom haunches or heavy shear tab, the positive moment significantly increased by about 1.5 times (heavy shear tab) and 1.8 times (bottom haunches) that of the bare steel section plastic moment (M_{pb}). Surely this is a side effect that should be considered in retrofit design of existing frames when checking the strong column-weak beam condition.

Figures 6 and 7 respectively present the beam plastic rotations and the panel zone shear distortions measured in the test. In the two specimens PN500C and PN500C-HST, the panel zone shear distortion is increased asymmetrically or further increased under the positive moment because the asymmetric topology of the moment demand on the panel zone. This implies that neglecting the presence of composite floor slabs can lead to the underestimation of panel zone shear distortion.

Whereas, in the haunch specimens (see Figs. 7a, 7d and 7e), the panel zone shear distortion is reduced, especially in the straight haunch specimen, because of the effect of enlarged (or dual) panel zone. This is surely a side benefit of haunch retrofit. Please refer to Lee-Uang [4] and Lee-Uang [10] for the behavior and mechanical modeling of dual panel zone of haunch connections. Haunch specimens tend to show rather severe pinching at



(a) All-welded

(b) Bolted web and welded flange

Fig. 8. Sizing and detail of heavy shear tab recommended

high drift cycles over 5% as a result of severe local buckling outside haunch-reinforced region (see Figs. 4d and 4e, and Fig. 6e and 6f).

Retrofit design by using triangular and straight haunches is well described in AISC Design Guide 12 [3] and Lee et al. [4, 5]. Although test data is limited, simple guides for sizing and detailing heavy shear tabs are illustrated in Fig. 8 based on successful test results of specimen PN500C-HST. Note that the web plastic section modulus of PN500 and PN500C-HST was respectively 25% and 40% of the plastic modulus of the whole beam section.

4. Summary and Conclusions

This experimental study investigated retrofit schemes for PN-type welded steel moment connections with highly composite slabs by using the beam bottom flange or beam web modification. The results of this study can be summarized as follows.

i) This study again confirmed that high composite action of a floor slab is detrimental to seismic performance of PN-type welded steel moment connections, showing seismic performance inferior to bare steel counterparts and eventually leading to the beam bottom flange fracture.

ii) All the three retrofitted specimens by using triangular or straight haunches and heavy shear tab effectively pushed plastic hinging outside the strengthened region and exhibited excellent total plastic rotation capacity exceeding 4% rad. Although the test data of heavy shear tab retrofit is very limited, simple retrofit design guides are also recommended.

iii) Generally, asymmetric topology of hysteretic response was pronounced in the highly composite slab specimens of this study. The maximum positive moments observed in heavy shear tap and haunch specimens were about 1.5 and 1.8 times the bare steel plastic moment, respectively. This side effect should be considered in retrofit design when checking the strong column-weak beam condition. This asymmetric topology of hysteretic response should be also properly incorporated when seismic performance evaluation of the retrofitted frames is conducted based on inelastic static or dynamic seismic analysis.

iv) The heavy shear tab retrofit tended to increase the panel zone yielding compared to PN-type connections. However, haunch retrofit significantly reduced the panel zone yielding, less than PN-type counterparts, because of the enlarged (or dual) panel zone effect. Both triangular and straight haunch specimens exhibited comparable seismic performance. They all showed rather severe pinching under unloading stage toward the positive moment after 5% story drift cycle as a result of severe local buckling at the plastic hinge.

5. Acknowledgements

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