DUCTILE DESIGN OF INTERMEDIATE HORIZONTAL BOUNDARY ELEMENTS IN STEEL PLATE SHEAR WALLS

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ABSTRACT :

Multi-story steel plate shear walls (SPSW) are progressively being used as the primary lateral force resisting system in buildings. However, insufficient information exists on the behavior of intermediate beams in this structural system as well as the performance of such beams having reduced beam sections (RBS). Using the results of an MCEER/NCREE collaborative experimental investigation on a full scale two-story SPSW specimen subjected to pseudo-dynamic load and subsequent cyclic load to failure, analytical models are developed to better understand intermediate beam behavior. In particular, the observed yield patterns and failure modes of the intermediate beam, which are inconsistent with predictions per simple model, are briefly described in this paper. In addition to the above, this paper also presents analysis capable of capturing the characteristic and behavior of intermediate beams with RBS connections. Based on capacity design concept, innovative design procedures and recommendations are provided to ensure the ductile behavior of intermediate beams in SPSW.

KEYWORDS: Ductile Design, Steel Plate Shear Walls, Horizontal Boundary Elements, Reduced Beam Sections

1. INTRODUCTION

Steel Plate Shear Walls (SPSWs) consist of unstiffened infill steel panels surrounded by columns, called Vertical Boundary Elements (VBEs), on both sides, and beams, called Horizontal Boundary Elements (HBEs), above and below. These infill steel panels are allowed to buckle in shear and subsequently form a diagonal tension field. SPSWs are progressively being used as the primary lateral force resisting systems in buildings (Sabelli and Bruneau 2006).

Past monotonic, cyclic and shaking table tests on SPSWs in the United States, Canada, Japan, Taiwan and other countries have shown that this type of structural system can exhibit high initial stiffness, behave in a ductile manner and dissipate significant amounts of hysteretic energy, which make it a suitable option for the design of new buildings as well as for the retrofit of existing constructions (Berman and Bruneau 2003a). Analytical research on SPSW has also validated useful models for design and analysis of this lateral load resisting system (Thorburn *et al.*1983; Driver *et al.* 1997; Berman and Bruneau 2003b). Recent design procedures for SPSW are provided by the CSA Limit States Design of Steel Structures (CSA 2003) and the AISC Seismic Provision for Structural Steel Buildings (AISC 2005). Innovative SPSW designs have also been proposed and experimentally validated to expand the range of applicability of SPSWs (Berman and Bruneau 2003a, Vian and Bruneau 2005). However, some impediments still exist that may limit the widespread acceptance of SPSWs. For example, little experimental information exists on the behavior of intermediate HBEs in SPSWs as well as the performance of such HBEs having reduced beam section (RBS) connections and composite behavior. Note that intermediate HBEs are those to which are welded infill steel panels above and below, by opposition to anchor HBEs that have steel panels only below or above. To further address the pressing concerns regarding behavior and design of intermediate HBEs, a two-phase experimental program was developed to test a two-story SPSW specimen having an intermediate composite beam with RBS connections under the collaboration of the Multidisciplinary Center for Earthquake Engineering Research (MCEER) in the U.S. and the National Center for Research on Earthquake Engineering (NCREE) in Taipei, Taiwan.

In this paper, following a brief review of the experimental observations from the MCEER/NCREE testing, the design recommendations will be presented, followed by examinations and explanations on the observed failure of the intermediate HBE.

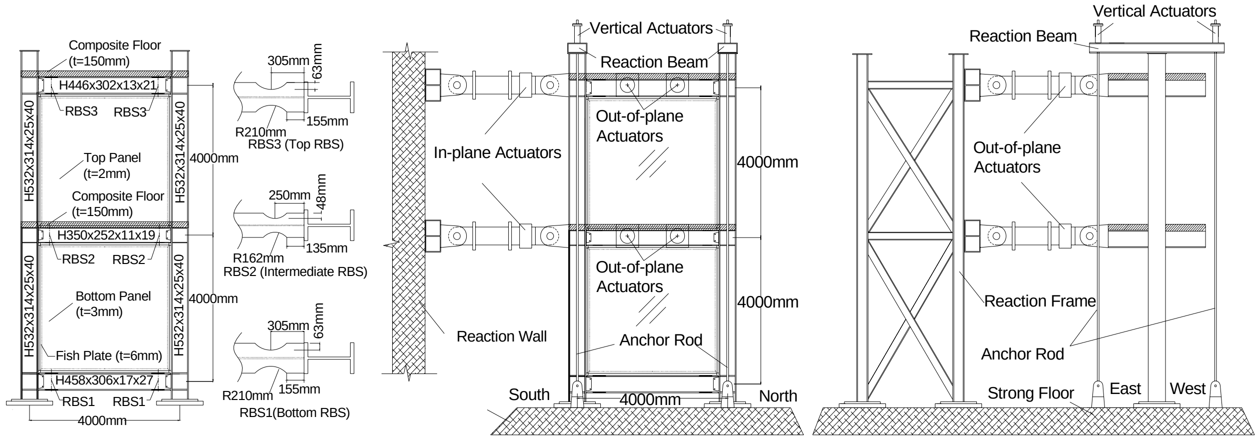
2. MCEER/NCREE TESTING

A full scale two-story one-bay SPSW specimen was fabricated in Taiwan and a two-phase experimental program (Phase I and II tests) was conducted at the laboratory of NCREE. The specimen with equal height and width panels at each story was measured 8000 mm high and 4000 mm wide between boundary frame member centerlines. HBE and VBE were of A572 Gr.50 steel members. Infill panels were specified to be SS400 steel which is similar to ASTM A36 steel in this case. The RBS connection design procedure proposed by FEMA 350 (FEMA 2000) was used to detail the HBE-to-VBE connections at top, intermediate and bottom levels respectively. The infill panels were designed to be 3mm and 2mm thick at the first and second story respectively. Prior to the Phase II tests, the buckled infill panels were removed and replaced by new panels.

The specimen was mounted on the strong floor. In-plane (south-north) servo controlled hydraulic actuators were mounted between the specimen and a reaction wall. Three 1000kN hydraulic actuators were employed to apply in-plane (south-north) lateral load on the specimen at each story. Two hydraulic actuators were used to avoid out-of-plane (east-west) displacement at floor levels. A vertical load of 1400 kN was applied by a reaction beam at the top of each column to simulate the gravity loads. The specimen schematic and test setup were illustrated in Figure 1. The designation of H shapes correspond to U.S. designation W shapes reflecting the depth, flange width, as well as web and flange thicknesses.

In Phase I, the specimen was tested under three pseudo-dynamic loads using the Chi-Chi earthquake record

(TCU082EW) scaled up to levels of excitations representative of seismic hazards having 2%, 10% and 50% probabilities of exceedances in 50 years, subjecting the wall to earthquakes of progressively decreasing intensity. No fracture was found in the boundary frame and it was deemed to be in satisfactory condition allowing for the replacement of infill panels. The buckled infill steel panels were replaced by new ones prior to submitting the specimen to the subsequent phase of testing. Detailed information about the results from the Phase I tests are presented elsewhere (Lin *et al.* 2007).



(a) Specimen schematic (b) In-plane setup (c) Out-of-plane setup

Figure 1 Specimen and test setup

In the first stage of Phase II, the specimen was tested under pseudo-dynamic load corresponding to the Chi-Chi earthquake record (TCU082EW) scaled up to the seismic hazard of 2% probability of occurrence in 50 years which was equivalent to the first earthquake record considered in the Phase I tests (Qu *et al.* 2007). Figure 2 shows the plastic deformations at the ends of the intermediate HBE observed during the test. As shown, the center of the yielded zone, which can be deemed to be the location of the lumped plastic hinge, moved toward the VBE face. This observation is different from those for a beam having RBS connections in a conventional moment frame, in which plastic behavior of the flange usually concentrates at the center of the RBS (i.e. where the beam flange is reduced most severely). Both the first and second story exhibited stable displacement-force behavior, with some pinching of the hysteretic loops as the magnitude of drifts increased, particularly after the development of a small fracture along the bottom of the shear tab at the north end of the intermediate beam at drifts of 2.6% and 2.3% at the first and second story respectively.

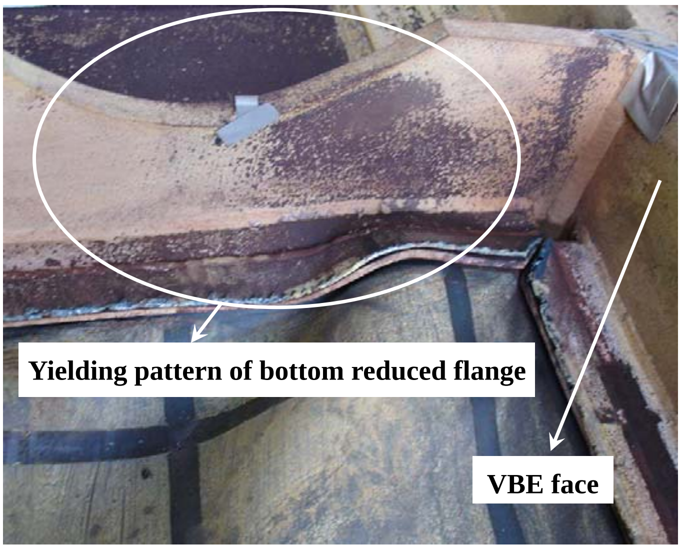


Figure 2 Yielding pattern at the end of intermediate HBE

The next stage of Phase II tests involved cyclic test on the SPSW specimen in order to investigate the ultimate behavior of intermediate HBE. As mentioned in the observations of Phase II pseudo-dynamic test, the boundary frame members were in good condition after the pseudo-dynamic test except for a small fracture was found along the bottom of the shear tab at the north end of the intermediate beam. To correct this limited damage and get a better assessment of the possible ultimate capacity of the SPSW, the damaged shear tab was replaced by a new one prior to conducting the cyclic test. A displacement-controlled scheme was selected for the cyclic test. Hysteretic loops of the specimen were then full until drifts of 2.8% and 2.6% at the first and second story respectively, when a complete fracture occurred along the shear tab at the north end of the intermediate HBE. A similar fracture developed along the shear tab at the south end of the intermediate HBE when the specimen was pulled towards to the reaction wall in the same cycle. At drifts of 3.3% and 3.1% at the first and second story respectively, the bottom flange at the north end of the intermediate HBE fractured as shown in Figure 3. However, no fractures developed in the reduced beam flange regions of the intermediate HBE.

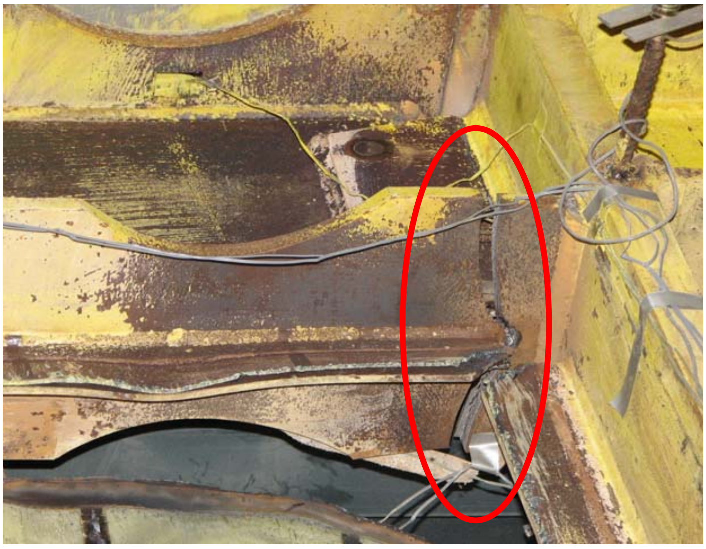


Figure 3 Ruptures at the end of the intermediate HBE

3. MOMENT DEMAND AT VBE FACES

Although many effects may have contributed to the unexpected failure at the ends of the intermediate HBE in the MCEER/NCREE SPSW specimen, flexural strength deficiency at the VBE face is a factor worthy of investigation. The original design of the intermediate HBE assumed all inelastic beam action concentrate at RBS centers and used a simple free body diagram as shown in Figure 4 to calculate the flexural demand at VBE face. In the free body diagram, L represents the span of the HBE, d represents the depth of the HBE, e represents the distance between plastic hinge to VBE face, distributed loads (i.e. ω_{ybi}, ω_{xbi}, ω_{ybi+1}, and ω_{xbi+1}) represent the infill panel yield forces; P_R and P_L represent axial forces at the right and left ends of the HBE; M_R and M_L represent moment demands at the right and left VBE faces; V_R and V_L represent shear forces at the right and left VBE faces; P_{RBSR} and P_{RBSL} represent axial forces at the right and left plastic hinges; V_{RBSR} and V_{RBSL} represent shear forces at the right and left plastic hinges; and M_{RBSR} and M_{RBSL} represent the plastic moments at the right and left plastic hinges respectively. For analysis purpose, the HBE is divided into three segments, the middle segment between two plastic hinges, and the right and left segments outside of the plastic hinges.

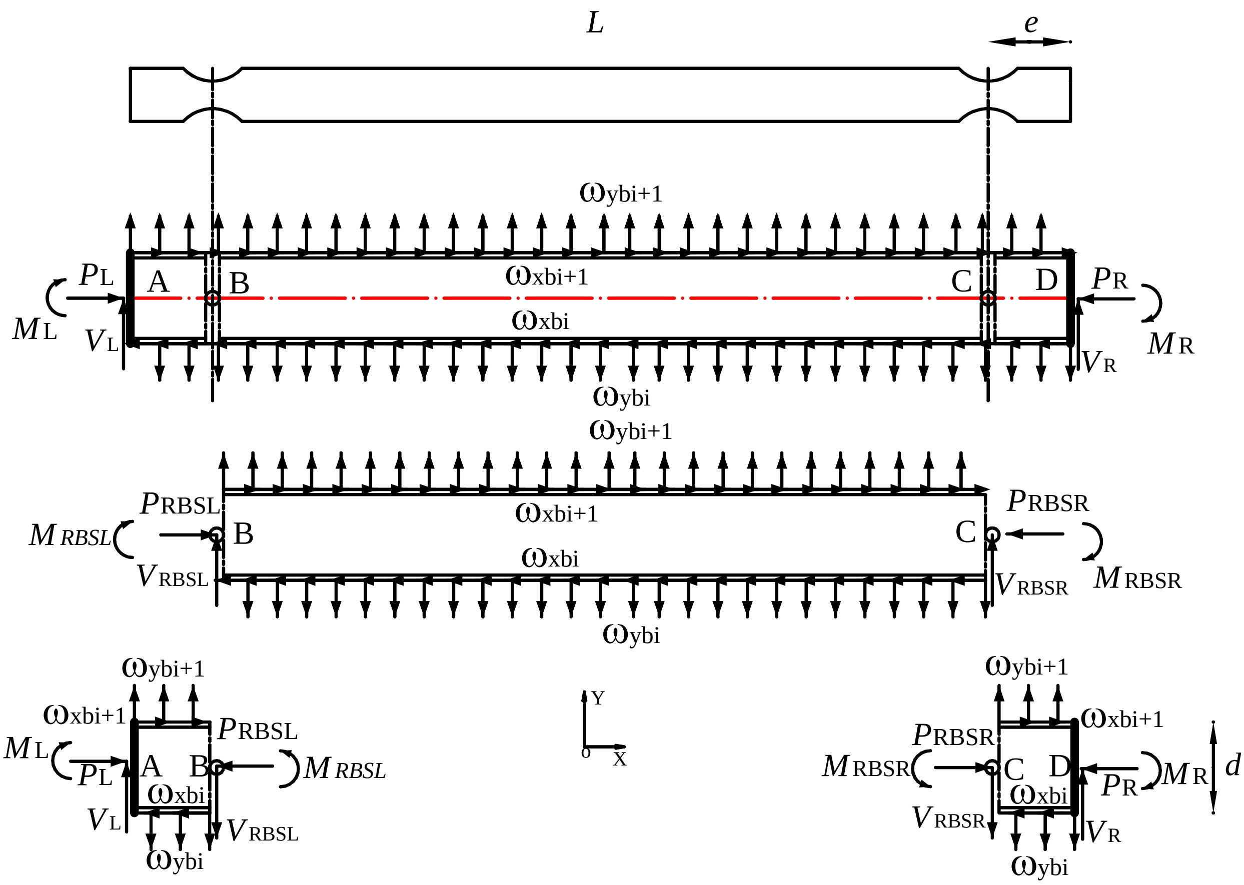


Figure 4 Free body diagram of intermediate HBE

For the middle portion of the beam (i.e. segment BC shown in Figure 4), the moment equilibrium to the left plastic hinge (i.e. point B) gives

$$M_{RBSR} + M_{RBSL} + \frac{\left(\omega_{ybi} - \omega_{ybi+1}\right)\left(L-2e\right)^2}{2} + \frac{\left(\omega_{xbi} + \omega_{xbi+1}\right)\left(L-2e\right)d}{2} - V_{RBSR}\left(L-2e\right) = 0 \quad (3.1)$$

Solving for V_{RBSR}:

$$V_{RBSR} = \frac{M_{RBSR} + M_{RBSL}}{L-2e} + \frac{\left(\omega_{ybi} - \omega_{ybi+1}\right)\left(L-2e\right)}{2} + \frac{\left(\omega_{xbi} + \omega_{xbi+1}\right)d}{2} \quad (3.2)$$

For the right end of the HBE (i.e. beam segment CD shown in Figure 4), the moment equilibrium to the right VBE face (i.e. point D) gives:

$$M_R + \frac{\left(\omega_{xbi} + \omega_{xbi+1}\right)de}{2} - V_{RBSR}e - \frac{\left(\omega_{ybi} - \omega_{ybi+1}\right)e^2}{2} - M_{RBSR} = 0 \quad (3.3)$$

Solving for M_R:

$$M_R = M_{RBSR} + V_{RBSR}e + \frac{(\omega_{ybi} - \omega_{ybi+1})e^2}{2} - \frac{(\omega_{xbi} + \omega_{xbi+1})de}{2} \tag{3.4}$$

Similarly, the shear force at the left plastic hinge, V_{RBSL}, and the moment demand at the left VBE face, M_L, can be determined:

$$V_{RBSL} = \frac{(\omega_{ybi} - \omega_{ybi+1})(L-2e)}{2} - \frac{M_{RBSR} + M_{RBSL}}{L-2e} - \frac{(\omega_{xbi} + \omega_{xbi+1})d}{2} \tag{3.5}$$

$$M_L = M_{RBSL} - V_{RBSL}e - \frac{(\omega_{ybi} - \omega_{ybi+1})e^2}{2} - \frac{(\omega_{xbi} + \omega_{xbi+1})de}{2} \tag{3.6}$$

The free body diagrams shown in Figure 4 produce reasonable results for beams having RBS connections in conventional moment frame. However, they may be inadequate for intermediate HBEs having RBS connections in SPSWs. The yielding pattern at the end of intermediate HBE shown in Figure 2 suggested that the center of the yielded zone, which can be deemed to be the location of lumped plastic hinge, moves towards the VBE face rather than occurs at the RBS centers. This effect can be ascribed to the presences of large axial and shear forces that vary along the HBE, and the presence of vertical stresses in HBE web due to infill panel forces (Qu and Bruneau 2008a; Qu and Bruneau 2008b).

For design purpose, it is recommended to assume that the actual plastic hinge moves toward VBE face and have a plastic section modulus, Z_{RBS}, equal to the average of the plastic section moduli of the unreduced part of the HBE and that at the RBS center (i.e. Z and Z_{center} respectively), which is :

$$Z_{RBS} = \frac{Z_{center} + Z}{2} \tag{3.7}$$

The moment resistance at the plastic hinge is reduced by the axial and shear forces in the HBE, and the vertical stresses in HBE web. This reduction effect can be considered by incorporating the cross-section plastic moment reduction factors, β_{RBSR} and β_{RBSL}, into the determination of moment resistances of plastic hinges:

$$M_{RBSR} = \beta_{RBSR} R_y f_y Z_{RBS} \tag{3.8}$$

$$M_{RBSL} = \beta_{RBSL} R_y f_y Z_{RBS} \tag{3.9}$$

where β_{RBSR} and β_{RBSL} can be determined by following the procedure proposed by Qu and Bruneau (2008b), R_y is the ratio of expected to nominal yield stress, and f_y is the yield strength of intermediate HBE.

Using the above method to account for the actual location and strength of plastic hinge, the free body diagram shown in Figure 4 and the corresponding equations remain valid. Noted that the moment demands predicted from Eqns. 3.4 and 3.6 should compare with the available strength at the right and left VBE faces.

4. EXAMINATION OF INTERMEDIATE HBE OF MCEER/NCREE SPECIMEN

Using the recommendations proposed in the prior section for checking the adequacy of flexural strength at VBE face, the intermediate HBE of MCEER/NCREE specimen was redesigned. Assuming the material has a yield strength of 346 MPa, the new intermediate HBE was determined to be a W24x76 member. The cross-section properties and flange reduction geometries of the redesigned and original members are summarized in Table 1.

Table 4.1 Summary of cross-section properties and flange reduction geometries

HBE	d (mm)	b_f (mm)	t_f (mm)	t_w (mm)	a^{-*} (mm)	b^{-*} (mm)	c^{-*} (mm)
Original	350	252	19	11	135	230	48
Redesigned	607	228	17.3	11.2	160	486	57

$^{-*}$ flange reduction geometry parameters described in FEMA 350

A preliminary assessment was made by comparing the design moment demands and available flexural strengths at the VBE faces. For comparison purpose, results of both the redesigned and original members are provided in Table 4.2.

Table 4.2 Design demands and available strengths at VBE faces

HBE	Left VBE face		Right VBE face	
	Demand (*kN.m*)	Strength (*kN.m*)	Demand (*kN.m*)	Strength (*kN.m*)
Original	660	774	748	571
Redesigned	809	951	876	897

As shown in the above table, the flexural strength of the original HBE at the right VBE face is smaller than the demand. This would explain the unexpected failure (i.e. fractures at the HBE ends) observed during MCEER/NCREE tests as shown in Figure 3. By comparison, the strengths of the redesigned HBE are greater than the demands, which indicate the SPSW designed per the recommendation proposed here would not have likely suffered from the observed premature failure.

5. CONCLUSIONS

Based on the observation of the yielding pattern and failure mode of the intermediate HBE in MCEER/NCREE specimen, recommendations to estimate the moment demand at the end of the intermediate HBE having RBS connections in SPSW have been proposed. A design procedure based on these recommendations uses simple free body diagrams and is able to prevent the observed premature failure of the HBE.

6. ACKNOWLEDGEMENTS

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