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THE SUB-STRUCTURAL PSEUDO DYNAMIC TESTS OF A FULL-SCALE TWO-STORY STEEL PLATE SHEAR WALL

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ABSTRACT

This paper describes a 2-story steel plate shear wall frame (SPSWF) specimen tested recently by using sub-structural pseudo dynamic testing procedures in National Center for Research on Earthquake Engineering. The paper focuses on the design procedures, experimental setup and phase I test results. The thickness of SS400 grade steel plate for first story wall is 3mm; and for the second story is 2mm. All the boundary beam and column elements are A572 GR 50 steel. In phase I test, the SPSW in each story has horizontal tube restrainers on both sides to minimize the out-of-plane displacements and the buckling sounds. The specimen was tested under pseudo-dynamic loads using three ground accelerations, which were recorded in the 1999 Chi-Chi earthquake and scaled up to represent seismic hazards of 2%, 10%, and 50% probabilities of exceedance in 50 years. Test results show that 1) the SPSWF specimen sustained three earthquakes without any significant wall fracture or overall strength degradation, 2) the horizontal restrainers are very effective in improving the serviceability of SPSWs, 3) the responses of the SPSWF can be accurately predicted using the strip model and the tension-only material property implemented in PISA3D computer program.

Keywords: steel shear wall, tension field, strip model, hybrid test

INTRODUCTION

A typical SPSW frame structure is given in Fig. 1. Because of the high stiffness and strength of SPSW frame system, thin steel plates are often used. The thin plate is very easy to buckle in shear. After the infill plate was buckled in shear, diagonal tension field action can be developed as shown in Fig. 2. The SPSW system can then dissipate energy through the yield of tension field. In recent years, several researchers have confirmed that the steel plate shear wall (SPSW) can be a viable seismic force resisting system for building structures (Berman 2002, Berman and Bruneau 2003, Driver et al. 1998, Lubell 2000). Although the SPSW can cost-effectively satisfy the lateral stiffness, strength and ductility requirements for seismic buildings, experimental research on large-scale SPSW structures is rather limited. Considering the small-scale structure tests could not obtain precise seismic behavior close to the performance of real buildings, a full scale 2-story SPSW specimen was constructed and

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tested recently in Taiwan. This study was a collaborative research (Tsai et al. 2006) among National Taiwan University (NTU), National Center for Research on Earthquake Engineering (NCREE), University at Buffalo (UB) and Multidisciplinary Center for Earthquake Engineering Research (MCEER). The specimen measures eight meters tall and four meters wide (Photo 1). This paper describes the associated design procedures, test results and analytical studies.

ANALYTICAL MODEL FOR SPSWS

The strip model (Fig. 3) first proposed by Kulak (1983) is often used for the analysis of SPSW frame. In his model, a series of inclined, pin-ended, tension members are used to represent the tension field action in the steel plate. Since the thin steel plate in the SPSW can be buckled easily before tension field actions developed, these tension members should possess very little compressive capacity. In this research, a tension-only material model was implemented (Tsai et al. 2006) for the PISA3D computer program (Lin and Tsai, 2003). The stress versus strain relationships for the tension-only material are shown in Fig. 4. This material property could represent the responses of a thin steel plate subjected to cyclic strains. Figure 5 shows the elevation of a SPSW test specimen (Lin and Tsai, 2004) and the analytical tension-only strip model. In Fig. 6, it can be found that the analytical results well agree with the test. In addition, the analytical results can be conveniently used to study the deformation demands imposed in the center and corner of the steel shear wall. It can be found that the tension field action is much more severe in the center (Strip+8) than that in the corner (Strip+1) of a SPSW.

EXPERIMENTAL PROGRAM

Design of the steel plate and boundary elements

Figures 7 through 9 indicate the floor framing plane, elevation of the SPSW, and 3D perspective of the prototype structure. It is assumed that the 2-story prototype building has a perimeter steel moment resisting frame (MRF) and two SPSWs in the transverse direction. It also assumes that: (1) building is located in East District in Chiayi City of Taiwan, (2) floor weight: 700kg/m^2. The fundamental vibration periods are 0.52 and 0.72 seconds in the transverse and longitudinal directions, respectively. According to the latest seismic force requirements for new buildings in Taiwan (ABRI 2002), the design base shear for both directions is 22% weight of the structure. First assuming that two SPSW frames (steel plate and boundary frame) resist 75% of the total lateral force, but the boundary columns resist 30% of the SPSW frame lateral force. The SS400 grade steel plate is chosen for the steel shear wall. All the boundary beam and column elements are A572 GR. 50 steel. The plate thickness and boundary frame member sizes were decided based on recommendations provided by Berman and Bruneau (2003). The thickness of steel plate for the first story wall is 3mm and for the second story is 2mm. The actual yield strength for the steel plate of each story are 335MPa (1F) and 338MPa (2F). Detail specimen member sizes are shown in Fig. 8.

Design of restrainers

The SPSW specimen is restrained by three horizontal restrainers in each side of infill panel. This type of restrainers can reduce the annoying buckling sounds and minimize the out-of-plane displacement of the steel panels (Lin and Tsai, 2004). The restrainer is designed by considering a uniformly distributed out-of-plane tributary load equal to 3% of the SPSW maximum shear. The sizes of the tube restrainers are: Tube-125x75x4 mm for the first story and Tube-125x75x2.3 mm for the second.

Design of beam-to-column connections

In order to avoid the fracture occurs in the column face, all the beam-to-column connections use the reduced beam section (Engelhardt et al. 1998) details near each beam end (Fig. 5). Due to the depth of the column section, deep column effects (Chi and Uang 2002) were considered by adding additional later supporting beams under the concrete slab (Photo 1).

Analytical predictions and hybrid simulations

Before the actual testing, analytical predictions were performed on the complete 2-story PISA3D structure model including the parameter MRF and the SPSW. For the SPSW frames, two series of strips with inclined angles of ±41 degrees were constructed (as that shown in Fig. 9). For the parameter MRF, all the beam and column members in MRF and boundary frame of SPSW adopt the bi-linear beam-column element. The tension coupon strengths (Table 1) of the steel plates, beam and column sections were incorporated into the analytical model. Based to the analytical results, it was decided to use three 100-ton actuators in each floor. During the actual hybrid experiments, the mechanical properties and the seismic responses of the entire perimeter MRF was analytically simulated while the two SPSWs were assumed identical and experimentally tested. After the Phase I tests, the steel panels were removed and a new set of steel panels were installed for the Phase II tests in order to study the seismic performance of the SPSWs without the steel tube restrainers. For details of the test results have been documented (Tsai and Lin 2006).

TEST PROCEDURES AND RESULTS

Test Procedures

In phase I tests, it was planed to test the specimen using pseudo-dynamic test procedures and a Chi-Chi earthquake record scaled up to represent seismic hazards of 2%, 10%, and 50% probabilities of exceedance in 50 years. The original ground acceleration record is TCU082EW as shown in Fig. 10. Test schedule and excitation information is shown in Table 2. However, some unexpected situations occurred in the very first test. In the Test 1, due to the concrete slab crack in the first story, the test had to be stopped at the time step of 9.5 sec. Then four H300mm floor beam were added below the concrete slab to assist the force transfer from actuators into the SPSW frame. Then, Test 1 was restarted after strengthening, failure occurred at south column base at a time step of 24 sec. In this case, it was found that two anchor bolts in the column base plate fractured. The test was stop again, and welds were added to attach the column base plate to the strong floor tie-down plate. Test 1 resumed and hybrid test was successfully completed. It was found in the specimen that significant buckling and a number of small cracks had occurred in the steel plate in both floors as exampled in photo 2. It was also found evident yielding of various boundary members (photo 3 to photo 7). After the Test 1, Test 2 and 3 were successfully completed in the Phase I study. After the phase I tests, steel plate had seriously buckled and many cracks could be observed. However, no fracture was found in the boundary frame. During the tests, all the key analytical predictions and experimental responses were broadcasted from a website (http://exp.ncree.org/spsw).

Key test result

Fig. 11 and 12 present the roof experimental displacement and base shear time histories in both the 2/50 and 10/50 events. The peak story drift for 2/50 and 10/50 events are 0.025 and 0.02 radians, respectively. It is evident that the peak roof displacement and the base shear responses can be satisfactorily predicted by PISA3D as evidenced in Fig. 11 and 12. Figure 13 shows the inter-story drift verses story shear relationships. It appears in Fig. 13 that the energy dissipation ability of the SPSW in 2/50 is satisfactory. But in the 10/50 event, the energy dissipation of SPSW is less pronounced as that found in the 2/50 event. During the Phase I tests, it appears that the specimen's strength or stiffness degrading is not significant.

SUMMARY AND CONCLUSIONS

Base on the test results and analytic study, conclusions and recommendations are made as follows:

- The responses of the SPSWF can be accurately predicted using the strip model and the tension-only material property implemented in PISA3D computer program.

- The SPSWF specimen sustained three earthquake excitations without significant steel plate fracture or overall strength degradation. It appears that the member sizes of the boundary frame is adequate to sustain the tension field actions developed in the steel plate.
- After phase I tests, the horizontal restrainers did not show any damage. It appears that the 3% of the in-plane force assumption is appropriate for sizing the restrainers.
- The strip model can be conveniently used to study the deformation demands imposed in the center and corner of the steel shear wall. It can be found that the tension field action is much more severe in the center than that in the corner of a SPSW.

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Table 1 Material coupon test result

	coupon positions		Thickness(mm)	fy(MPa)	fu(MPa)
Steel	Panel (SS400)	1F	3	338	482
		2F	2	335	412
	Beam (A572)	Base Beam(Web)	19	285	480
		Base Beam(Flange)	28	355	487
		Middle Beam(Web)	12	505	626
		Middle Beam(Flange)	19	476	581
		Top Beam(Web)	13	305	460
		Top Beam(Flange)	22	354	517
	Column (A572)	web	25	377	505
		Flange	40	363	544
Concrete		fc'=27.5 MPa			

Table 2 Test schedule

Phase Ⅰ Test: Restrained Steel Plate Shear Wall		
	Excitation	Hazard Level
Test 1	Chi-Chi(TCU082EW)	2% in 50 Years (PGA=0.67g)
Test 2	Chi-Chi(TCU082EW)	10% in 50 Years (PGA=0.53g)
Test3	Chi-Chi(TCU082EW)	50% in 50 Years (PGA=0.22g)

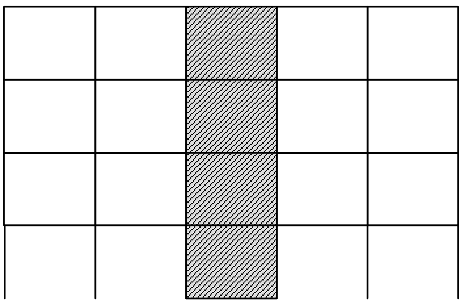

Fig. 1 SPSW frame system

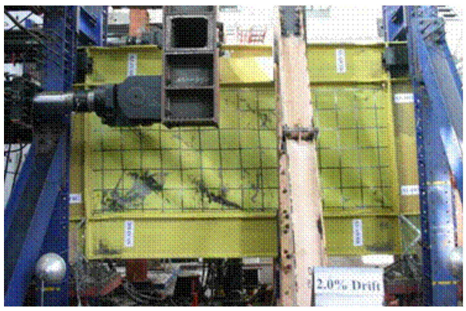

Fig. 2 Tension field action

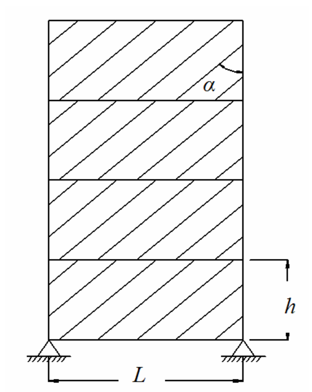

Fig. 3 Strip model

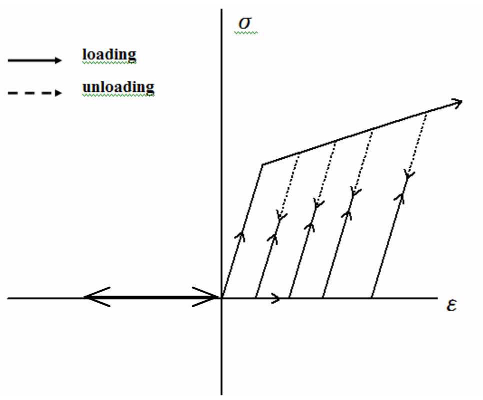

Fig. 4 Tension-only material

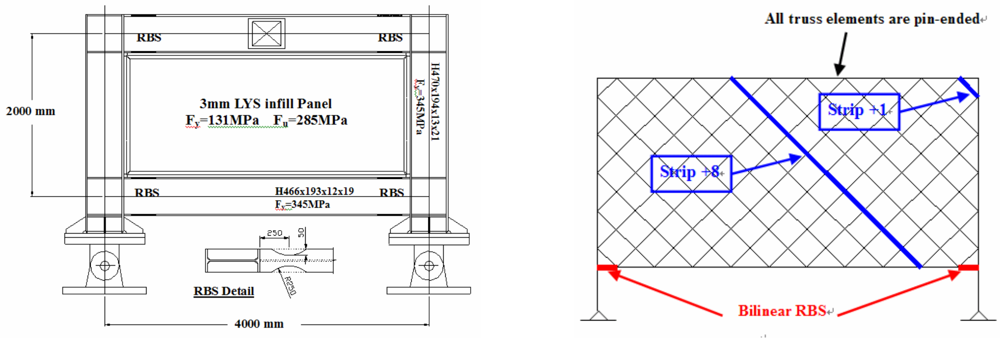

Fig. 5 SPSW specimen and PISA3D analytical model (Lin and Tsai, 2004)

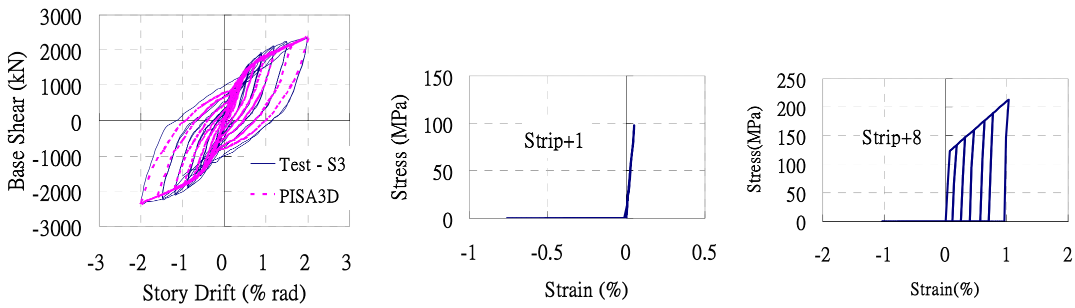

Fig. 6 PISA3D simulation result

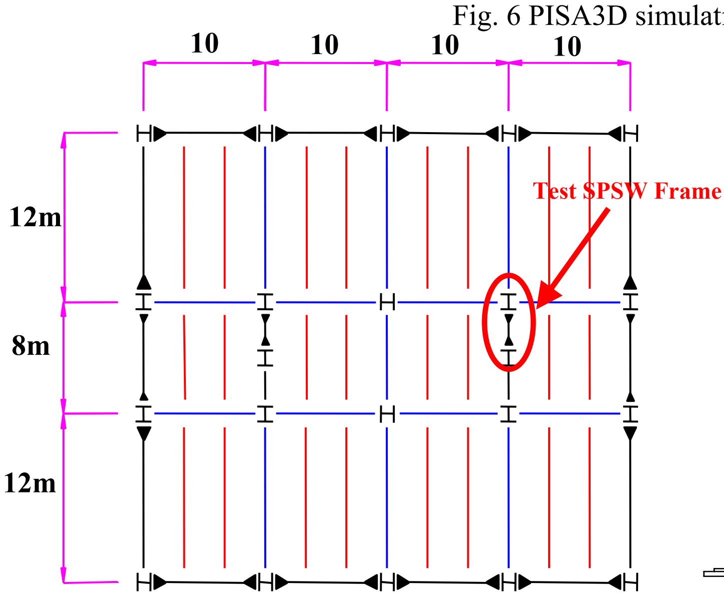

Fig. 7 floor framing plan

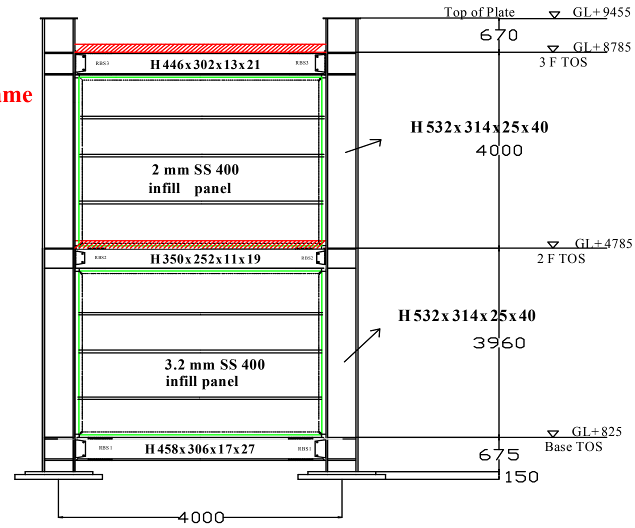

Fig. 8 Specimen elevation

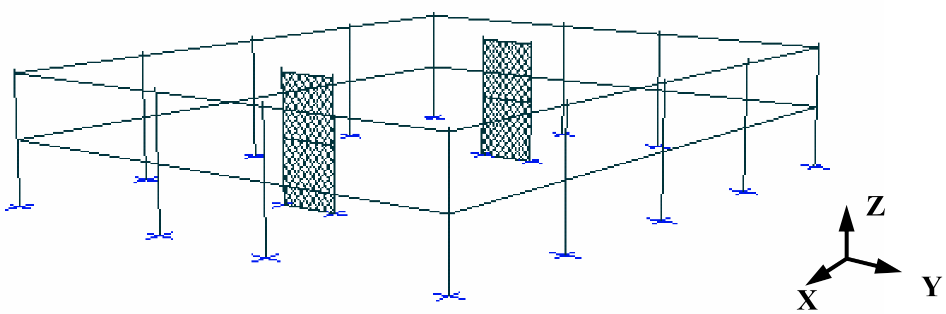

Fig. 9 PISA3D analytical model

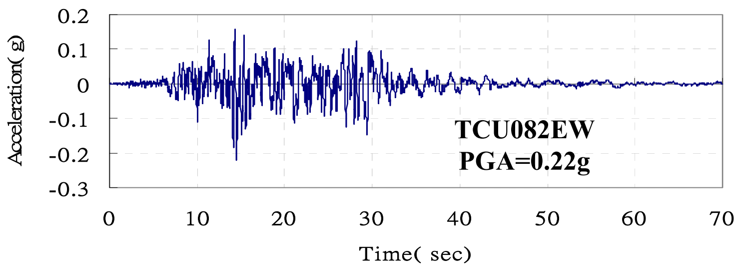

Fig. 10 Original Ground Acceleration Time History

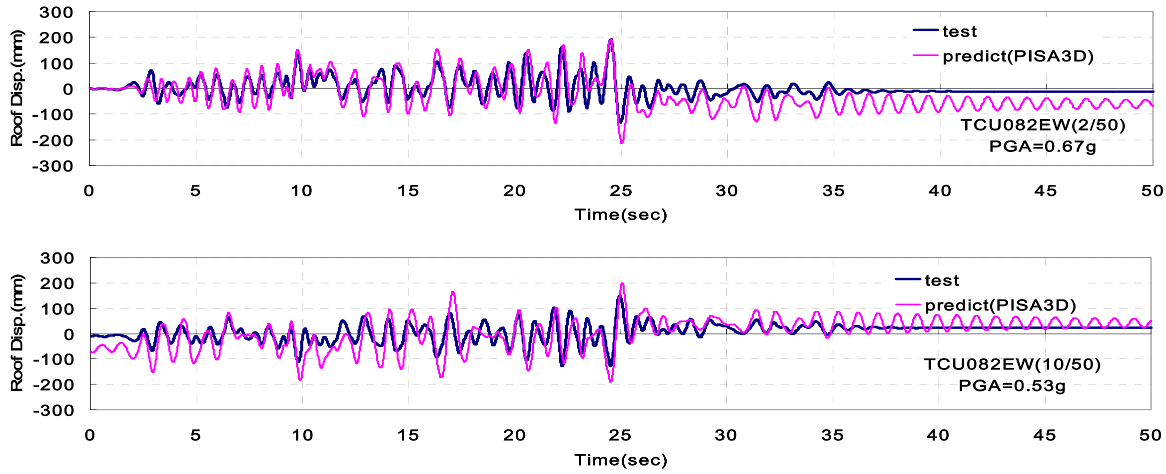

Fig. 11 Roof displacement time history (2/50&10/50)

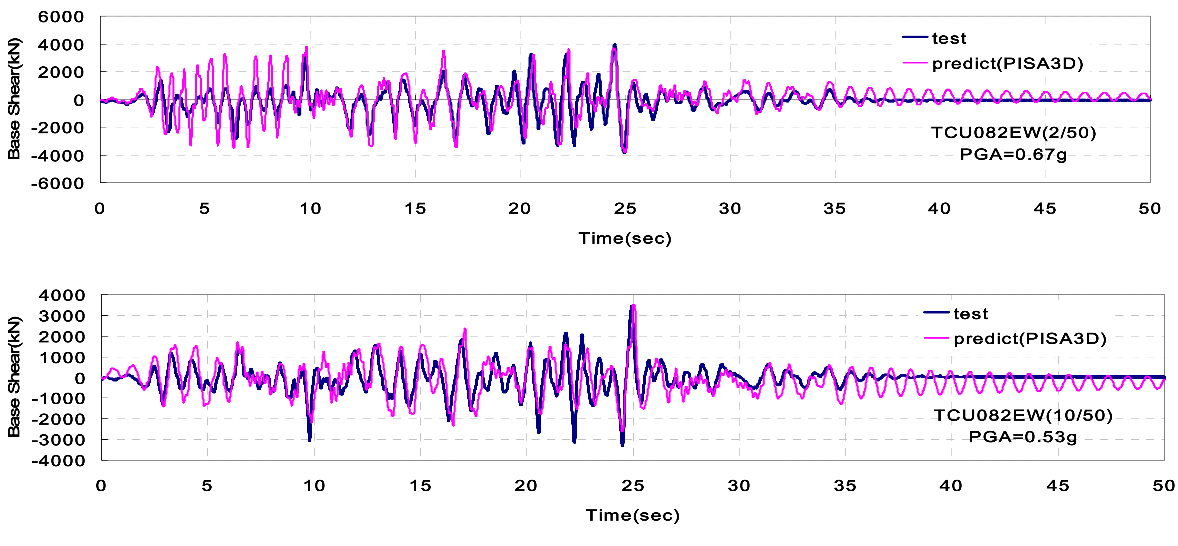

Fig. 12 Base Shear time history (2/50&10/50)

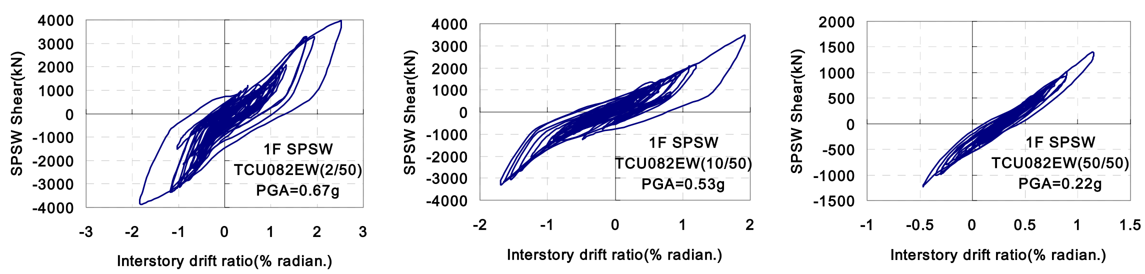

Fig. 13 Inter-story drift ratio vs. story shear (1F SPSW)

Photo 1 Two-story SPSW specimen

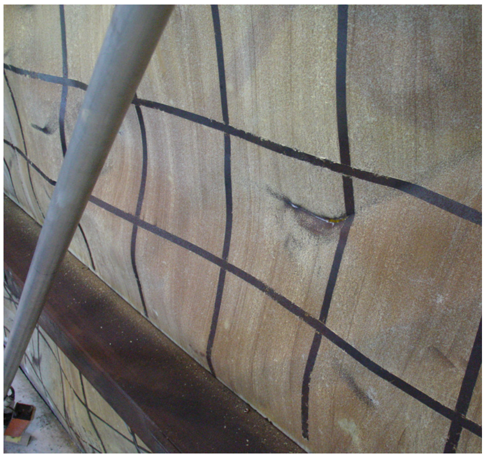

Photo 2 Steel plate buckling and cracks

Photo 3 Top Beam web yielding

Photo 4 Middle Beam web yielding

Photo 5 Base Beam web yielding

Photo 6 Column flange yielding

Photo 7 Column web yielding