AN INVESTIGATION FOR THE DESIGN OF FRAMED STRUCTURES WITH INFILL WALLS

by

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SUMMARY

Results of six tests on approximately one-third scale steel frames with infill concrete shear walls are briefly summarized. The findings indicate that (1) infilled shear walls provide frames with adequate stiffness and with improved load-carrying capacity, (2) steel frames enhance the post-ultimate load-carrying characteristics of composite frames, and (3) locally buckled steel frames can be fully re-strengthened to their shear strengths with newly infill walls.

INTRODUCTION

The composite structures of steel skeleton clad with reinforced concrete (so called the steel reinforced concrete structures) have been used for more than fifty years in Japan. It is well known that two buildings of this type designed by Tachu Naito(1886-1970) behaved satisfactorily during the Great Kanto Earthquake of 1923 while a few of other buildings were badly damaged. The most probable conclusion is the combination effect of rigid frames and proper diagonal bracings and/or seismic walls. Since 1923 the majority of intermediate height buildings in Japan are of this type which are modified in steel members from solid web to built-up open web.[1] The steel frames with infill shear walls are scarcely used because design practice is not established yet; although, steel frames clad with shear resisting precast concrete units are gradually introduced in modular building systems such as telecommunication stations.

Low-rise steel structures, which are now common for small buildings in Japan, are frequently designed too flexible for the drift excursion under earthquake shocks. A typical inelastic displacement was observed on a well fabricated four-story steel building in Sendai after the 1978 Miyagiken-oki Earthquake. The inelastic drift of over 10 cm concentrated on the third floor. The steel structure infilled with shear walls is one of the practical and economical solutions for the prevention of such excessive displacement.

OBJECTIVE AND SCOPE

In-situ concrete walls are considered to be convenient components for small steel buildings which are being constructed in various places, in various forms and for various purposes. The combination of in-situ concrete wall and steel building could be available to re-strengthen steel buildings damaged during a severe earthquake.

To establish a design provision, six tests on approximately one-third scale model were investigated. There were two types of portal steel frames

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(two specimens of each type) of wide flange sections in which the columns were oriented for strong and weak axes bending respectively. Prior to casting the walls into the frames, one of the frames of each type was loaded to a post-ultimate state to investigate the load-carrying characteristics of the steel frames alone.

Headed studs are employed mainly to prevent the falling off of the wall from the frame even during an earthquake shock. The test specimens are designated in Table 1.

EXPERIMENTAL INVESTIGATION

Test Specimen- The test specimens to investigate the composite action are shown in Fig. 1. The pretested steel frames were adjusted to minimize the residual horizontal displacement. Local bucklings developed in the columns (as shown in Fig. 2) were left without making any adjustments. In-situ concrete was cast vertically to simulate the actual conditions of forms fitted within steel frames through delivery mouths which were provided under the beams. Excessive concrete in the mouths was removed and that area was made even immediately after the concrete set. There was no visible seperation between the beam and the wall. Wall reinforcements are about 5 % in both directions. The amount is about twice the minimum reinforcement of walls specified in AIJ building standard.[2] Extra bars, which were provided vertically in the wall to seperate it into three blocks, were intended for distributing cracks evenly under shear without the appearance of major diagonal cracks. The measured properties of materials used in the specimens are summarized in Table 2.

Method of Loading- The stiff foundation beam of the specimens was tightly fastened to a rigid test floor or to a heavy loading frame. All specimens always sustained a constant vertical load at the top of columns during the tests. The load corresponded to 30 % of the yield strength of the columns. A reversed and repeated horizontal load were then applied to a post-ultimate state at the top of a column. The ultimate deflections were about 4 cm. However, specimens B1 and B2 were loaded in one direction after several reversals of loading for lack of the capacity of a double acting jack.

Principal Test Results- Test results are summarized in Table 3 and horizontal load-carrying characteristics are illustrated in Fig. 3. Since the studs alone were inadequate to tightly connect the shear wall to the steel frame, the elastic deflections of the frames with infill walls were about twice as large as those expected by the calculations in the fully composed structures. After reaching ultimate the composite frames showed large ductilities as good as those in the steel frames. It is evident that this fact is mainly due to slips between the steel members and concrete walls. These slips might be enhanced by a small number of the studs and shrinkage of concrete. The cracks which appeared in the walls at the ultimate load were well distributed in an orthogonal pattern. The destructive diagonal cracks did not appear because the walls were constrained to deform smoothly by stiff columns; however, the effect of extra bars was insignificant. Near the last stage of loading, crashing and spalling developed at the corners of the walls. The steel members were deformed outwardly due to the wall and their flanges were distorted accordingly. However, the vertical loads maintained satisfactorily. The final stages of specimens A2 and B2 are illustrated in Fig. 4. The repaired specimens A1 and B1 showed higher maximum loads but smaller ductilities than those in standard specimens A2 and B2 respectively. Cyclic straining at plastificated parts of the steel frames could provide not only more strength but also fatigue cracks which reduced the ductilities.

FINDINGS

Locally buckled frames are able to fully recover their load-carrying capacities to the earthquake shock with newly infilled shear walls. The maximum horizontal load may be calculated with the summation of the ultimate load in the steel frame and the ultimate strength of the wall as a compressive bracing. It is concluded that from these experiments the effective width of the bracing in the wall is 5.4 times the thickness of the wall. This width is in agreement with the results obtained in reinforced concrete shear walls under shear failure.[3)]

RECOMMENDATIONS

The infill walls are recommended to provide proper stiffness and strength to steel frames which are likely to be designed too flexible. In these cases, the steel frames should sustain their gravity loads even under a severe earthquake shock. To repair the damaged structure may be by no means an easy task. The infill wall will be considered as a compressive bracing of a rectangular cross section composed of an effective width and the thickness of the wall. The effective width is supposed to be 5.4 times the thickness.

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Table 1. Designation of specimens

Name	Bending Axis in Columns	Remarks
Steel Frame A0	Strong Axis	No Infill Wall
Repaired Structure A1	〃	Second Use of A0
Standard Structure A2	〃	
Steel Frame B0	Weak Axis	No infill Wall
Repaired Structure B1	〃	Second Use of B0
Standard Structure B2	〃	

Table 2. Measured properties of materials

a) Steel

	Yield Strength (t/cm^2)	Ultimate Strength (t/cm^2)	Elongation (%)
Beam and Column flange	3.23	4.76	22.6
Beam and Column web	3.29	4.87	19.3
Deformed bar D6	4.25	6.61	21.1
Stud 13φ-100	3.82	4.93	16.4

b) Concrete

Specimens	Compressive Strength at Test (kg/cm^2)	Split-Cylinder Test (kg/cm^2)	Young's Modulus (kg/cm^2)
A1	131	15.0	1.5×10^5
A2	135	13.1	1.7×10^5
B1	176	20.8	2.3×10^5
B2	183	13.7	2.3×10^5

Table 3. Test results

Specimens	Q_i (ton)	R_i ($\times 10^{-3}$)	Q_u (ton)	R_u ($\times 10^{-3}$)
A0				
A1	10.6	1.34	44.4	13
A2	12.4	1.03	34.9	13
B0				
B1	17.8	2.10	33.0	12
B2	15.0	1.60	31.4	30

Q_i= initial crack load
R_u= ultimate load
R_i= lateral displacement to height ratio at initial crack load
R_u= lateral displacement to height ratio at ultimate load

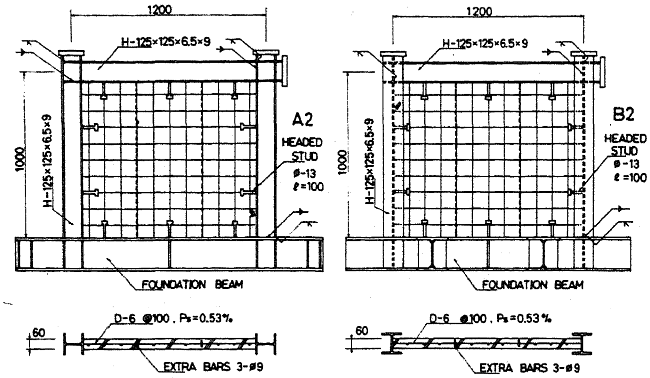

Fig. 1 Test specimens A2 and B2

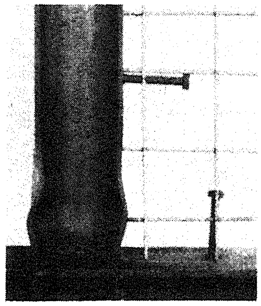

Fig. 2 Local buckling before concreting

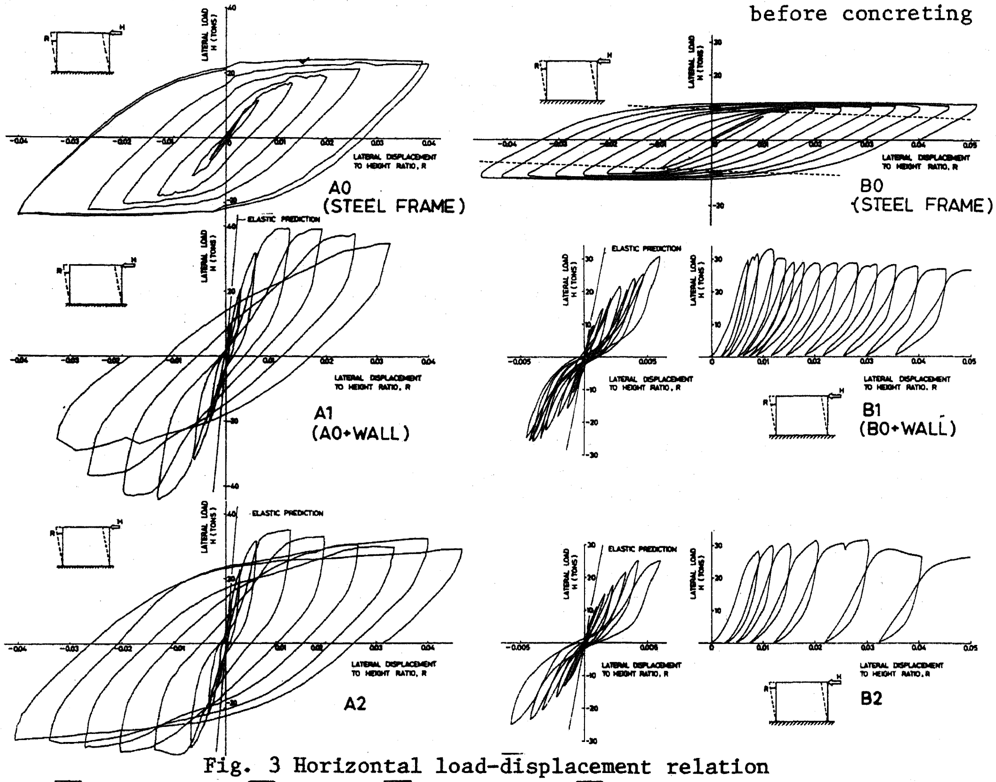

Fig. 3 Horizontal load-displacement relation

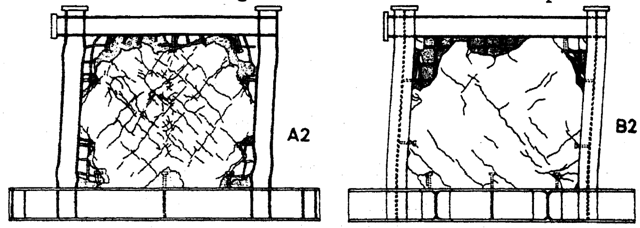

Fig. 4 Final stage of specimens A2 and B2