Non-linear response of combined system, 3D wall panels and bending steel frame subjected to seismic loading

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Abstract

3D wall panels are used in the construction of exterior and interior bearing and non-load bearing walls and floors of building of all types of construction. This system consists of a welded wire space frame integrated with a polystyrene insulation core and two layers of concrete on both sides. In this paper, attention is focused on the experimental measurements of the seismic response of 3D wall panels surrounded by a steel bending frame. The approach of quasi-static cyclic loading is employed using horizontal actuators to the combined system. The vertical, lateral and horizontal displacements are measured by LVDT equipment. The failure mechanism of 3D wall panels is described in detail. The evaluation of strength and stiffness degradation of the whole system is presented based on the envelope force-displacement curve of actual specimens under cyclic loads. The results of the current study are shown in the form of ductility factors, hysteresis loops and load-displacement envelope curves. The comparison between the ductility of sole steel frames, 3D shear walls and the combined system as the main theme of the current research is presented. Finally, this work clarifies the benefits of using 3D wall panels as a strengthening method for existing steel frame buildings and confirms the feasibility resistance of such combined systems.
Keywords: 3D-panels, combined system, cyclic loading.

1 Introduction

3D wall panels are used in construction of exterior and interior bearing and non-load bearing walls and floors of building of all types of construction. This system

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consists of a welded wire space frame integrated with a polystyrene insulation core. The wall panel is placed in position and wythes of concrete are applied to both sides. Wall panel receives its strength and rigidity by the diagonal cross wires welded to the welded-wire fabric on each side. This combination produces a truss behavior, which provides rigidity and shear terms for full composite behavior. Figure 1 shows schematically the 3D panel.

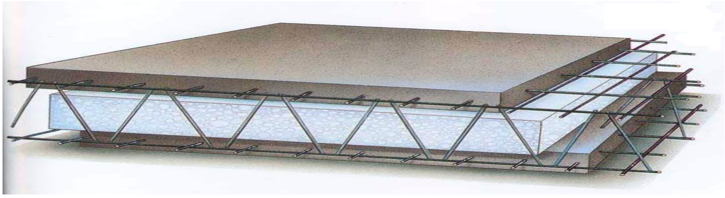


Figure 1: 3D Sandwich panel.

Salmon and Einea [1] presents the results of full-scale test of prototype sandwich panel under transverse loading in a vertical position. Nijhawan [2] measured experimentally the interface shear force and designed the shear connectors. Eiena et al. [3] used the plastic composite diagonal elements to implement in sandwich panel as shear connector for increasing the thermal insulation of this system.

Through this study the behavior of 3D panels in combination to steel moment frame was investigated, the fracture mechanism of concrete wythes and the adequacy of steel bars designed based on ACI 318-95 and procedure of PCI design handbook [4].

2 Theoretical study

2.1 Ductility capacity

The term "ductility" in seismic design is used to mean the ability of structure to undergo large amplitude cyclic deformation in the inelastic range without a substantial reduction in strength. The ductility is calculated by various types, which reflects structural behavior. The displacement ductility capacity μ is defined as:

$$\mu = \frac{\Delta_{\max}}{\Delta_y}$$

where max $\Delta_{\max}$ is the maximum displacement and Δ_y is the displacement at yield

3 Experimental program

Four 3D walls are provided to be combined with portal steel frames. These specimens are considered to represent the critical and structural elements with a rectangular cross section and are experimentally subjected to low cycle horizontal loading regimes. The type of failure and behavior of such structural elements are investigated in details.

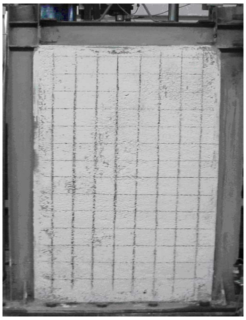


Figure 2: 3D panel with steel frame.

3.1 Wall details

The wall specimens have 1200 mm height, 640 mm width, 140 mm constant thickness including 40mm shotcrete in each side and 60 mm of expanded polystyrene core. The welded wire fabric is consisted of a cold rolling of steel bar with final outside diameter of 3.5 mm in accordance with ASTM A82 and automatic welding process with accordance of ASTM A185. The yield and ultimate strength of drawn and annealed wires are 4000 and 5200 MPa, respectively. The shotcrete used for all specimens is used from Portland cement (II), river sand with maximum 8 mm diameter, drinkable water. The (W/C) is about 0.5 and the mix is made of 400 kg cement, 1750 kg sand and 180 kg water for a unit cubic meter shotcrete. Compression tests are carried out on (150*150*150 mm) standard cubes and provided cores from shotcrete. Tables 1 and 2 are shown results of the compression strength tests.

3.2 Frame details

Two IPE120 as columns and one IPE120 as beam are used for constructing of flexural steel frame. The beam to column and column to base plate connections are rigid based on Iran steel structure design code.

Table 1: Geometry and compressive strength of standard specimens.

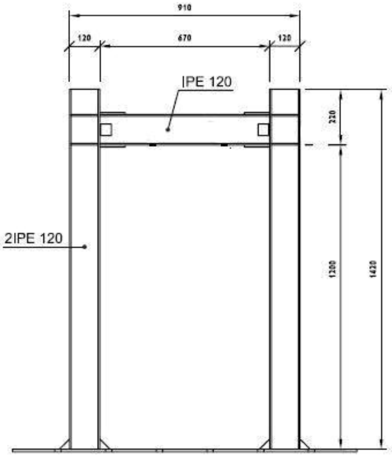
Specimen No.	Wall Dimensions (cm)	Specific Gravity (kg/m^3)	Max. app lied force (tone)	Cube compressive strength (bars)
035	120*64	2330	87	387
036	120*64	2310	91	404
037	120*64	2290	82	364
038	120*64	2280	85	378

Table 2: Geometry and compressive strength of cylindrical cores.

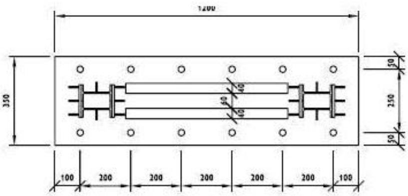
Core No.	Core Dimensions (Diameter*length)	Specific gravity (kg/m^3)	Max. applied force (kg)	Core Compressive strength (kg/cm^2)
1	107.6*54.4	2150	5350	230.2
2	100.4*54.4	2170	5100	219.4
3	99.5*54.4	2230	5250	225.9
4	108.3*54.4	2100	5400	232.3
5	103.3*54.4	2250	5200	223.7

Table 3: Tensile strength.

Specimen No.	Specimen Dimensions	Specific gravity (kg/m^3)	Max. applied force (kg)	Slump (cm)	Cylinder tensile strength (kg/cm^2)
1	15*30	2300	3700	8	52
2	15*30	2320	3500	8	50



(a) side view



(b) top view

Figure 3: Details of steel frame.

3.3 Loading history and test procedure

To simulate loading sequences that might be expected to occur during earthquake, simplified types of horizontal cyclic load history are adopted. Since no standard cyclic test procedures has been introduced for testing of such system, the horizontal load is applied at a quasi-static rate in displacement controlled cycles with different patterns which corresponded to three major states, namely cracking state, yielding state and ultimate state. The applied displacement is started from 0.5 mm to 5 mm in 10 cycles. In the second phase of loading, the increment of displacement is increased to 1 mm and after 19 cycles in displacement of 14 mm the displacement increased to 2 mm and after 23 cycle the increment increased to 4 mm and in 30 cycle system was failed.

Linear transducer of types LVDT is used to measure and monitor the horizontal displacements at top, mid height and bottom of specimens. The measured values of applied load and displacement are recorded by a computer data logger capable of measurement to sensitivity ranges of 0.1 N, 0.001 mm, respectively.

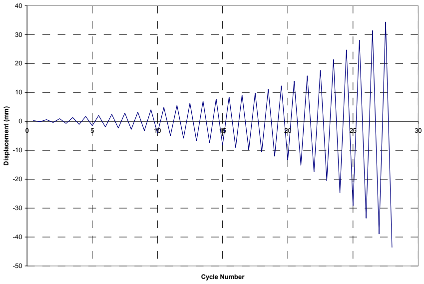


Figure 4: Applied successive cyclic displacement.

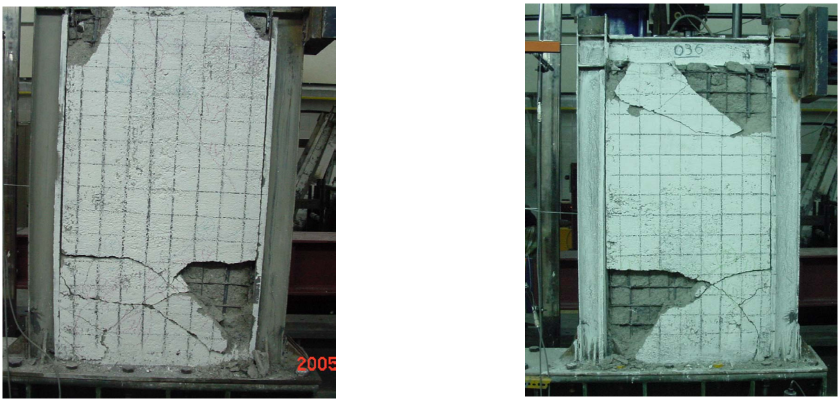


Figure 5: The crack patterns and failure mechanism in specimen at late stages of cyclic loading.

3.4 Experimental results

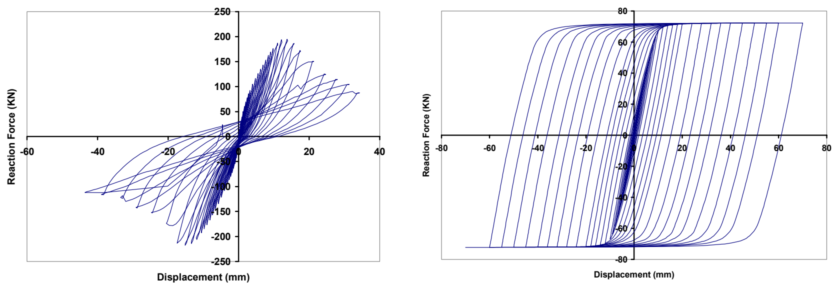
Table 4 shows the results of applied displacement in type 1 specimen during cyclic loading. Up to 0.4 mm displacement, the panel behaves in elastic zone and the first crack is occurred at the location of 200 mm below the crest, where the connector reinforcements are discontinued. In early stages of loading, the cracks are minor and their direction is mostly horizontal. By increasing applied displacement level, the cracks propagate in both sides of panel. Figure 5 represents the shear cracks pattern at near final stages. It is seen that, the cracks are more visualized and many of them are opened.

The direction and propagation of cracks in panels are shown in figure 5. The first crack is located above the base anchor bars when the applied displacement reaches to 3 mm. The main cracks in these types of wall panels are at the bottom area and it continues up to one third of panel height. It is seen that the direction of crack at the area near the panel edges are mostly horizontal and it obliged to 45° in the middle of panel.

Table 4: Experimental results in cyclic successive loading for combined system.

Cycle No.	Horizontal Displacement in both sides (mm)		Reaction Forces in Both sides (kN)		Panel Stiffness in both sides (kN/m)	
	back	forth	back	forth	back	forth
1	0.270	0.123	15.144	16.808	55884	136762
2	0.616	0.347	27.891	29.113	45237	83804
3	0.966	0.672	39.463	42.745	40847	63561
4	1.323	1.061	50.134	53.966	37871	50860
5	1.703	1.470	60.301	63.828	35397	43414
6	2.075	1.878	69.308	72.346	33387	38511
7	2.450	2.312	77.705	79.857	31713	34540
8	2.848	2.733	85.582	86.559	30042	31671
9	3.323	3.171	92.177	93.353	28468	29432
10	4.015	4.029	104.830	108.191	26106	26847
11	4.797	4.907	116.267	121.580	24235	247.76
12	5.576	5.771	126.511	134.373	22688	23284
13	6.274	6.653	135.015	146.174	21520	21971
14	6.986	7.415	143.335	156.906	20516	21158
15	7.723	8.140	153.12	166.95	19825	20509
16	8.455	8.972	162.463	176.630	19214	19686
17	9.103	9.787	169.196	184.905	18586	18892
18	9.793	10.563	176.249	192.553	17996	18228
19	11.087	12.019	187.332	206.064	16895	17144
20	12.299	13.404	194.126	214.369	15783	15992
21	13.914	15.174	194.889	217.773	14006	14351
22	15.787	17.439	186.294	213.666	11800	12252
23	17.624	20.469	172.539	174.111	9790	8505
24	21.316	24.748	150.83	152.937	7075	6179
25	24.728	29.096	125.381	143.06	5070	4916
26	28.025	33.483	114.359	123.793	4080	3697
27	31.395	38.939	104.619	116.588	3332	2994
28	34.315	43.566	87.384	112.512	2546	2582

Based on ASTM standard method, the applied displacement and their corresponding reaction forces are plotted in a load deflection hystersis curves. Their values are listed in table 4.



(a) combined system (b) steel frame

Figure 6: Load deflection Hystersis energy loops due to successive applied displacement.

4 Discussion

Due to the significant stiffness of wall panels, up to 217kN, the majority of produced energy in cyclic loading is absorbed by the action of combination of wall and frame. It was shown, [5], the bare shotcrete wall could resist only about 20 kN. The corresponding displacement of total system is about 15.2 mm.

It should be noted that all the combined systems are essentially subjected to the intended in-plane action of successive displacement where the out-of-plane movement is prevented, figure 7. Measured maximum value of the vertical displacement at the top of the wall is found to be insignificant since its value is about 0.1% of the maximum wall displacement for all specimens.

Figure 7: Lateral supports for out-of-plane movement.

4.1 Cracking

For all specimens, cracks are initially formed near the bottom part of the tensile zone of the wall when only about 10% of the final horizontal deformation capacity is applied. The significant inclined cracks initiated at the tension zone of the wall during compressive reversal displacements. These cracks continue to penetrate deeply into the centre of the wall towards the compression zone.

4.1.1 Strength reduction

Table 5 indicates horizontal load-carrying capacity of specimens 035 and 036, respectively. It is seen from these tables that the cyclic displacement regime employed in all specimens appeared to have had an insignificant effect on the ultimate strength of the walls.

4.1.2 Stiffness and deformations

Variation of lateral deflection in successive cyclic loading and also average of stiffness against horizontal reaction forces are illustrated in table 4. The displacement relating to the first occurred cracking is about 10%. The horizontal load versus top displacement is shown in figure 8, indicates a distinctly non-linear deformation response. In terms of stiffness reduction, it should be stated that the stiffness is gradually decreased for both, fig. 8.

4.1.3 Ductility

The capacity of the structural elements to deform beyond yield or elastic limit with minimum loss of strength and stiffness depends upon their ductility. The load-displacement envelope curve obtained from hystersis loops is sketched in figure 7. The maximum displacement corresponding to 80% of ultimate load is introduced as Δmax. The displacement at the first yielding in panel, Δy, is determined at the intersection of two lines. The initial tangent of envelope curve with horizontal line passes at the level of 80% of P_u, [6]. It is described in figure 8.

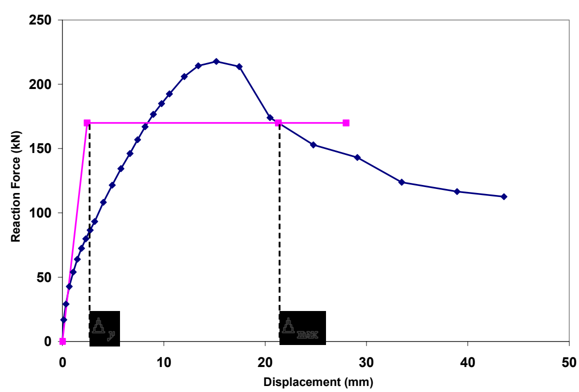


Figure 8: Load deflection envelope curve.

Table 5: The ductility values for wall specimens based on Δmax /Δy.

Wall panels		Δ_{max}	Δ_y	μ
Specimen without gap	035	21.32	2.44	8.73
	036	22.07	2.58	8.55

5 Conclusions

The current work describes the cooperation between 3D panels as infill wall and traditional steel frames. The following conclusions remarks are raised based on previous work by the first author [5]:

- For all specimens, the plastic hinge is formed at the extreme fiber of the wall section and at the vicinity at the base, above the foundation. The distribution and propagation of cracks show that 3D sandwich panels with limited height, behaves in shear performance. The observed horizontal crack at the base prior to ultimate state may be due to sliding at the vertical reinforcement. This caused considerable reduction in the strength, stiffness and energy dissipation of the specimens.
- Externally reinforcing 3D wall panels, which basically behaves in shear manner, enhances more ductility in performance design approach and increases substantially load carrying capacity of system.

Finally, this study clarifies benefits of using lightweight prefabricated panels as a strengthening method for existing of steel frame building and confirms feasibility resistance of such compound system.

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