DESIGN AND RESEARCH ON COMPOSITE STEEL AND CONCRETE FRAME-CORE WALL STRUCTURE

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ABSTRACT :

This paper presents the design, research and related joints' details of a 31-story composite frame-core wall structure, which is located in Beijing City, a region of seismic fortification of 8 degree. In order to improve the ductility, bearing capacity of the core walls and to ensure inelastic deformation capacity of the longitudinal coupling beams carried steel trusses, proper steel frames were embedded within the longitudinal core walls. The results of elasto-plastic time-history analysis under the action of rarely occurring earthquake are very close to the data of shaking table test. Experimental results show that there are no obvious cracks in the core walls, spalling of concrete and local buckling of reinforcement at the bottom of the core wall's boundary elements and the composite columns at the perimeter have not been observed, even the elasto-plastic story drift angle has reached 1/101, the whole structure has better seismic performance. However, higher strain measurement of the floor beam end during the rarely occurring earthquake shows partial restraining moment should be considered at the connection.

KEYWORDS: hybrid wall system, embedded steel frames, analysis and tests, anti-seismic for intensity 8.

1. PREFACE

The composite steel and concrete frame-core wall structure is developed rapidly over the last years. Compared with the reinforced concrete structure, this structural system has the advantages such as low structure self weight, large occupied area, short construction period, and lower cost. Owing to lack of experimental basis and less of earthquake disaster knowledge from the high seismic regions, the composite steel and concrete frame-core wall structure is often used in the low seismic regions and non-seismic design project. Does this structural system available for higher seismic regions? What is its seismic performance? This paper presents the design and research of the composite steel and concrete frame-core wall structure through LG Beijing Building.

LG Beijing Building is a office and commercial complex, which is located at the Chang An Avenue, a main street of Beijing City, covering an area of 151,345 ㎡. This building is composed of 4-level basements with the embedded depth of 24.6m, two 31-story towers with the height of 141m which are 56m wide apart, and a 5-story annex among them, see Figure 1.1 for the plane of first floor. The 6th and its above typical floor of the tower is in the shape close to ellipse, which is 44.2m along the long axis and 41.5m along the short axis, the width-to-height ratio of the tower is 3.4. The largest size of the column space is 9m and the largest span from the core wall to the column at the perimeter is 14.75m, see Figure 1.2 for sixth to its above typical floors. According to the sunlight requirements, the tower would gradually drawback from the 24th floor forming a large slope in the north side, see Figure 1.3 for the cross section of the structure. The seismic fortification intensity of this project is 8 degree, the shear-wave velocity of soil layers is 260m/s and the site category is Ⅱ. The plane dimension of the core wall at the low region of the tower is 12m×23.6m, taking up 16% area of the typical floor. The thickness of the core shear-wall is 450mm and 600mm along the X and Y direction respectively which is mainly controlled by story drift angle. The ductility moment-resistant space frame at the perimeter consists of circular composite columns and wide-flange steel beams. For the circular composite column, its diameter from 1st to 14th floor, from 15th to 25th, and from 26th to its above are 1500mm, 1350mm, 1200mm respectively. The steel shape of H416×406×30×48 is embedded within the column with the rate of steel of

2.76～4.3%. For the steel spandrel beam at the perimeter, its size from 1st to 24th floor, and from 25th to its above are H903×304×15×20 and H835×292×14×18 respectively. The concrete strength grade is from 1st to 15th floor, C60; from 16th to 25th floor, C50 and from 26th to its above floor, C40.

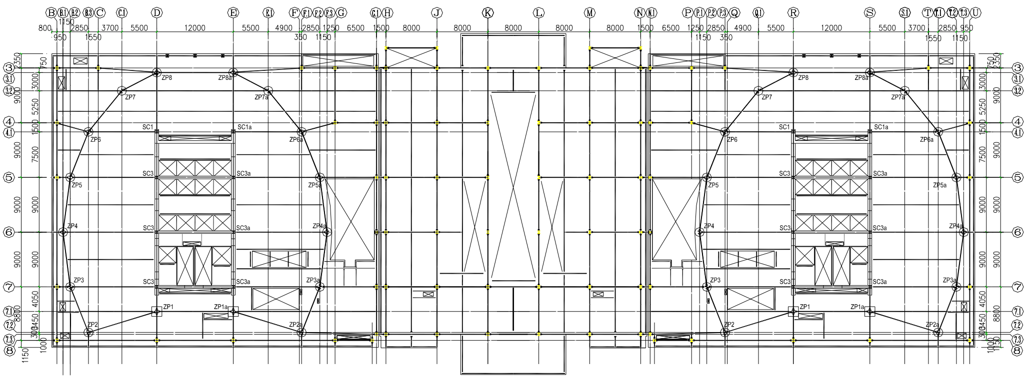


Fig. 1.1 1F Reflection-Framing Plan

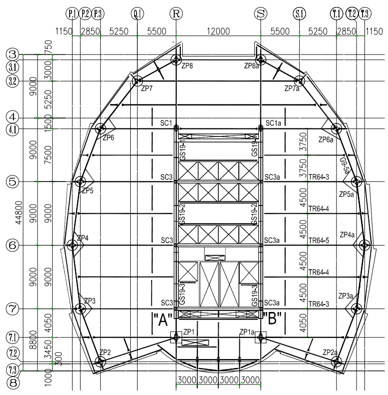


Fig. 1.2 6-12F Reflection-Framing Plan-West Tower

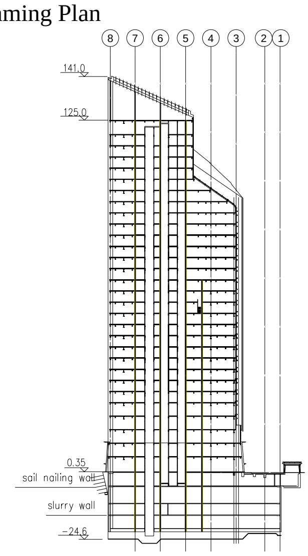


Fig. 1.3 Structural Cross Section

2. THE OVERALL SEISMIC PERFORMANCE OF THE STRUCTURE

For dual system of frame-core wall structure, core wall is the key lateral load resistant member bearing more than 80% earthquake shear force. In order to improve the ductility of the core shear walls steel columns of HM340×250×9×14 were embedded at the four corners of the core wall and the intersections of longitudinal and transversal core shear-walls which would also be considered as the second proof to the shear wall in avoidance of potential consecutive collapse owing to the degradation of its vertical bearing capacity from the serious cracking of the concrete under rarely occurring earthquake. On the other hand the embedded steel column also enhance the anchorage of the floor truss/beam. While as, in order to strengthen the inelastic deformation capacity of the longitudinal coupling beams carried steel trusses and ensure the integrity character of the core wall, steel shapes were encased inside the coupling beams, which were connected with embedded steel columns forming the embedded steel frames within the longitudinal core shear-walls.

In this project the analysis of overall structure is processed in two stages; the first stage is the elastic analysis under the action of frequently occurring earthquake, in which the response spectrum method for modal analysis was used to calculate all the members' bearing capacity and the elastic story drifts. The second stage is elasto-plastic time-history analysis under the action of rarely occurring earthquake, which is used to calculate and check the elasto-plastic story drifts.

2.1. Results of the Elastic Analysis under the Action of Frequently Occurring Earthquake

Table 2.1 shows the results of several analysis program applied in structural design and the results of impulse test on-site. During the analysis, the program of SATWE and SAP84 took the equivalent floor beam model instead of the floor truss, while the program of MIDAS/Gen established the actual truss model, see Figure 2.1. It is assumed that the floor beam (truss)-to-wall connections are hinged. It could be found that the analysis results are close to the results of impulse test, and the period for the impulse test is somewhat short due to its smaller excitation energy.

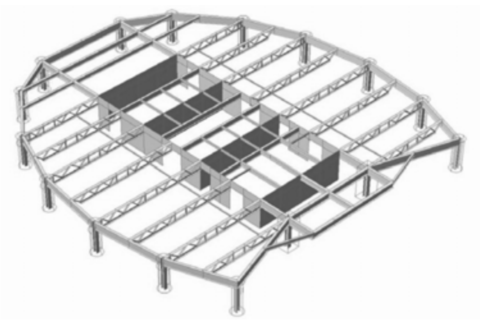


Fig.2.1 Typical Floor Plan with Steel Truss

Table 2.1 Results of Different Structural Analysis Program

Serial No.	Name of the Program and Site Test	First Period (s)	Second Period (s)	Third Period (s)	Maximum Story Drift Angle				Instruction to Model
					X Direction		Y Direction		
					Value	Position	Value	Position	
1	SATWE	2.51	1.70	1.29	1/1026	26F	1/2123	24F	Equivalent Steel Beam instead of Steel Truss
2	SAP84	2.45	2.05	1.87					
3	MIDAS/Gen	2.615	2.166	1.738	1/909	21F	1/1429	21F	Actual Truss
4	IMPULSE TEST	2.041	1.690	1.204					Site Measurement

2.2. Response of the Top Steel Frame under the Action of Frequently Occurring Earthquake

Owing to the top steel frame on the top of structure is flexible and abrupt change of story stiffness, the whipping effect is clear and the torsion influence is very serious, diagonal braces are set between the columns (Figure 2.2.1). Figure 2.2.2 and Figure 2.2.3 show the elastic time-history displacement curve of the roof and the top of the steel frame with and without diagonal braces respectively along X direction. During the calculation, the EL Centro earthquake wave based on 0.07g was used. It could be found the maximum displacement of the top of steel frame with and without diagonal braces are 125mm and 147.5mm (occur at the 4th second) respectively, while at the same time, the maximum displacements of the roof are 100mm and 97.5mm respectively. The displacement of the top of steel frame are 125% and 150% of the maximum displacement of the roof. It is evident that the stiffness of the top steel frame is strengthened, the whipping effect and the torsion influence are decreased after diagonal braces are added.

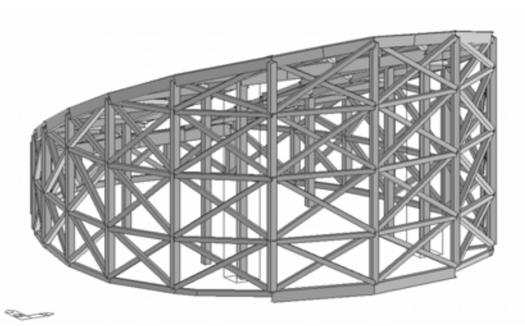


Fig.2.2.1 Top Steel Frame with Diagonal Braces

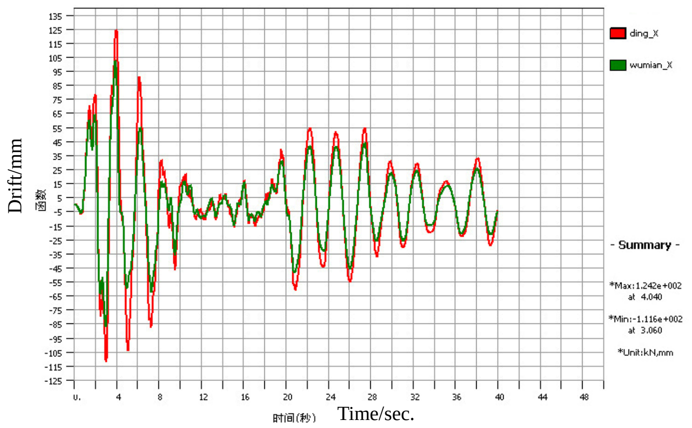


Fig.2.2.2 Time-History–Displacement-Curve of the Roof and Top Steel Frame With Diagonal Braces

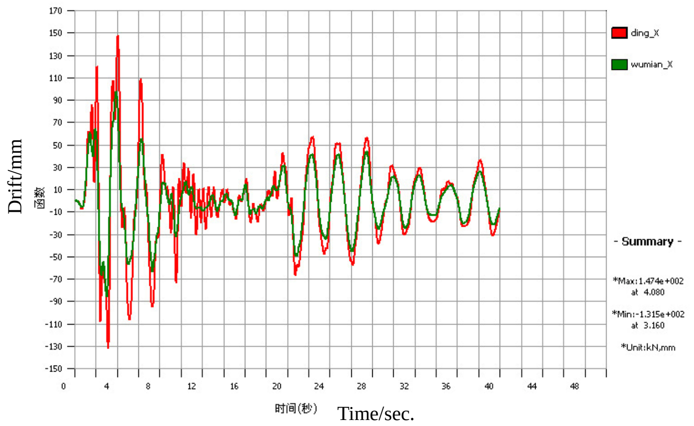


Fig.2.2.3 Time-History–Displacement-Curve of the Roof and Top Steel Frame Without Diagonal Braces

2.3. Results of the Elasto-plastic Time-history Analysis under the Action of Rarely Occurring Earthquake

As the structural plane is complex, the three-dimensional dynamic program EPDA was adopted to conduct the elasto-plastic time-history analysis to the structure. The Taft seismic wave in south to north direction was selected as one of the actual wave, while based on the situation that although the site category is Ⅱ, but the shear wave velocity is close to that of site category Ⅲ, then the EL Centro seismic wave in south to north direction was selected as the another actual wave. The synthetic wave from site EQ2 was provided by Beijing Geotechnical Design & Research Institute. After the elastic time-history analysis under the action of frequently occurring earthquake, these three waves could satisfy the requirements of the national standard of the People's Republic of China, *Code for Seismic Design of Building* GB50011-2001. The acceleration peak value of the time-history analysis under the action of rarely occurring earthquake should be 0.4g.

Table 2.3.1 and 2.3.2 summarize up the story drift angles along X and Y directions during the elasto-plastic time-history analysis under the action of rarely occurring earthquake. According to the *Code for Seismic Design of Building*, the elasto-plastic time-history analysis should take the average value of the maximum story drift angles of the above three seismic waves, therefore the X direction should be 1/135, the Y direction should be 1/149. It could be found the average values are smaller than 1/100 of the specified allowable value. Table 2.3.3 shows the comparison of first inter-story's seismic shear force, it could be found that the story seismic shear force under the action of rarely occurring earthquake should be at least 3.16 times greater than that of the frequently occurring earthquake, which could be referred that the structure has a better seismic bearing capacity.

Table 2.3.1 X Direction Story Drift Angles of Elasto-Plastic Time-History Analysis

Floor	EL Centro (NS)	Taft (NS)	Synthetic Wave EQⅡ	Average Value	Remarks
	Amax=0.4g	Amax=0.4g	Amax=0.4g		
Top Steel Frame	1/1108	1/1076	1/801	1/974	
Top Steel Frame	1/316	1/338	1/277	1/324	Weak Location
31	1/397	1/409	1/330	1/375	
30	1/386	1/391	1/316	1/361	
29	1/384	1/374	1/303	1/350	
28	1/342	1/343	1/278	1/318	
27	1/269	1/272	1/229	1/255	
26	1/182	1/197	1/151	1/175	
25	1/166	1/182	1/139	1/160	
24	1/157	1/171	1/130	1/151	
23	1/152	1/164	1/125	1/145	
22	1/150	1/158	1/121	1/141	
21	1/150	1/152	1/117	1/138	
20	1/151	1/148	1/114	1/135	Weak Location
19	1/155	1/145	1/112	1/135	
18	1/161	1/144	1/111	1/135	
17	1/170	1/144	1/111	1/137	
16	1/182	1/146	1/112	1/141	
15	1/195	1/149	1/113	1/145	
14	1/212	1/158	1/116	1/153	
13	1/210	1/202	1/116	1/164	
12	1/210	1/247	1/112	1/169	
11	1/211	1/242	1/110	1/167	
10	1/214	1/241	1/109	1/167	
9	1/220	1/245	1/108	1/168	
8	1/227	1/254	1/107	1/170	
7	1/228	1/267	1/107	1/172	
6	1/232	1/280	1/108	1/175	
5	1/240	1/296	1/109	1/179	
4	1/252	1/323	1/111	1/187	
3	1/279	1/374	1/113	1/199	
2	1/477	1/595	1/114	1/239	
1	1/1543	1/1398	1/117	1/303	

Table 2.3.2 Y Direction Story Drift Angles of Elasto-Plastic Time-History Analysis

Floor	EL Centro (NS)	Taft (NS)	Synthetic Wave EQⅡ	Average Value	Remarks
	Amax=0.4g	Amax=0.4g	Amax=0.4g		
Top Steel Frame	1/306	1/300	1/243	1/280	
Top Steel Frame	1/200	1/195	1/159	1/183	Weak Location
31	1/858	1/858	1/614	1/758	
30	1/636	1/620	1/403	1/529	
29	1/678	1/665	1/436	1/569	
28	1/614	1/605	1/405	1/522	
27	1/406	1/394	1/276	1/348	
26	1/269	1/311	1/220	1/261	
25	1/239	1/298	1/211	1/244	
24	1/224	1/287	1/203	1/233	
23	1/213	1/281	1/198	1/226	
22	1/187	1/245	1/182	1/201	
21	1/164	1/216	1/160	1/177	
20	1/149	1/197	1/142	1/159	
19	1/144	1/192	1/133	1/153	
18	1/144	1/191	1/129	1/151	Weak Location
17	1/145	1/191	1/125	1/149	
16	1/148	1/193	1/124	1/150	
15	1/155	1/198	1/124	1/153	
14	1/167	1/206	1/125	1/159	
13	1/190	1/239	1/129	1/174	
12	1/213	1/288	1/134	1/192	
11	1/217	1/308	1/137	1/198	
10	1/217	1/319	1/140	1/202	
9	1/219	1/325	1/143	1/205	
8	1/221	1/326	1/145	1/207	
7	1/226	1/326	1/148	1/211	
6	1/232	1/328	1/150	1/214	
5	1/238	1/338	1/151	1/218	
4	1/250	1/361	1/153	1/226	
3	1/281	1/412	1/154	1/240	
2	1/424	1/539	1/156	1/282	
1	1/1327	1/1251	1/160	1/384	

Table 2.3.3 Comparison of First inter-story's Seismic Shear Force

Direction	Item	El Centro (NS)	Taft (NS)	Synthetic Wave	Elastic Analysis
		Amax=0.4g	Amax=0.4g	Amax=0.4g	Amax=0.07g
X	Inter-story Shear Force	67850	79390	56780	17980
	comparison	3.77	4.42	3.16	1.00
Y	Inter-story Shear Force	73730	82720	63230	20020
	comparison	3.68	4.13	3.16	1.00

2.4 Comparison of the Elasto-plastic Dynamic Time-history Analysis and the Shaking Table Test

In order to verify the overall seismic performance of the LG Beijing Building under the action of rarely occurring earthquake after embedded steel frame within the core wall, Beijing Institute of Architectural Design cooperated with Tongji University carried a shaking table test of a 1/20 scale model of the tower of LG Beijing Building on 2002 at Shanghai. During the test, EL Centro, Taft and synthetic seismic wave EQⅡ were used. Experimental results show there were no obvious cracks in the core shear-walls, spalling of concrete and local buckling of reinforcement at the bottom of the boundary element of core shear-walls and the composite columns at the perimeter have not been observed, the interaction of core wall and its surrounding composite frame was efficient. These all represent that the structure has better deformation capability and bearing capacity.

2.4.1 *Displacement*

In the elasto-plastic time-history analysis, the average value of the maximum story drift angles along X and Y direction were 1/135 and 1/149 respectively, while the maximum value of the story drift angle of the shaking table test along X and Y direction were 1/120 and 1/101 respectively. It could be found that the result of the X direction was very close and there was a little difference along Y direction.

2.4.2 *Weak location*

The elasto-plastic time- history analysis showed the weak locations of the tower along X and Y direction were at 18th to 20th floors and 16th to 18th floors respectively, and the top steel frame. According to the shaking table test, LG Beijing Building had no evident weak story, but had the following weak locations: (1) the top steel frame; (2) levels between 17th to 23rd floor of the tower; (3) cross section of the columns close to the beam-column joints of the frame at the perimeter. It could be found that the calculated results were very close to the testing results.

2.4.3 *Stress response of the beam end connected with the core wall under the action of earthquake*

Table2.4.3 Maximum Strain of Beam End Under the Action of Earthquake

Measure Point	Floor	8 Degree Frequently		8 Degree Basically		8 Degree Rarely	
		Lower Flange of Beam	Close to the Upper Flange	Lower Flange of Beam	Close to the Upper Flange	Lower Flange of beam	Close to the Upper Flange
A	1	72	30	176	95	344	176
	15	114	55	590	291	1108	366
	30	54	7	185	16	443	29
B	1	3	13	4	17	4	41
	15	79	30	175	97	335	432
	30	37	18	101	69	227	95

During the analysis of the frame-core wall structure, it is usually assumed that the floor beam (truss) undergo the gravity load only, the steel beam (truss) is hinged with the core wall . But as the effect of the high strength bolt, the rotation-restriction still exist at the end of the beam (truss) connected with the concrete core wall. Such rotation-restriction capacity would definitely response to the end of beam (truss) under the earthquake action. Table 2.4.3 provides the maximum stress value of the beams end at the point A and point B (see Fig 1.2), during the shaking table test, based on the different fortify level under the EL Centro seismic wave. From the Table 2.4.3 it could be found the following rules:

(1) The maximum tensile stress of the cross section of the beam end occurred at the lower flange of the beam, and decreased gradually toward the upper flange, which presented that the neutral axis located at the concrete topping above the beam. The compression of the concrete topping and the tension of the steel beam forming the restraining moment at the beam end under the earthquake action, which could increased with the fortify level. It should be paid more attention on the partial restraining moment of the beam end during the design process.

(2) All the maximum stress of the beam end occurred at the 15th floor and not elsewhere under the earthquake action based on different fortify level. These positions of the maximum stress were very close to those of the maximum story drift angle, such phenomenon showed that the integrity character of the whole structure was well, the interaction between core wall and the composite space frame at the perimeter was efficient, so the deformation curve of the core wall could be changed from bending type to shear-bending type.

(3) The connection of the floor beam (truss) and core wall is one of the key locations of the frame-core wall structure. The type of anchorage of embedded steel column could enhance the anchorage ability of the floor beam (truss), and the experimental results showed that the connections kept in good condition.

3. JOINTS DESIGN

Joints design is one of the important links of the structural design. Besides the bearing capacity, energy dissipation, ductility should meet the requirements, the installation of the headed studs are very important for the steel shape and concrete to share the applied loads especially within the range of the beam-column joint, which was proved by the joints test. Additional the problem among steel shape, steel bars, and concrete casting should be well solved for the sake of convenience to construct.

3.1 Beam-column joint

The steel beam and embedded steel column could be connected by two patterns: the column through type and the beam through type. For LG Beijing Building column through type was selected, the steel beam and the embedded steel column was connected within the joint in order to restrict the connection by the concrete. During the joint design, the curved face bearing plate (FBP) with the same height of the steel beam's web was installed along the concrete surface of the composite column, its thickness was 20mm, see Figure 3.1. In order to verify the seismic behavior of such circular composite column, Beijing Institute of Architectural Design cooperated with Beijing University of Technology carried out 6 large-sized specimens of circular composite column-steel beam joints test under the cyclic reversed loading. The results of the test showed that FBP could restrict the concrete of the joint core and delay the damage of the concrete as well as restrict the shear deformation of the embedded steel column's flange within the joint.

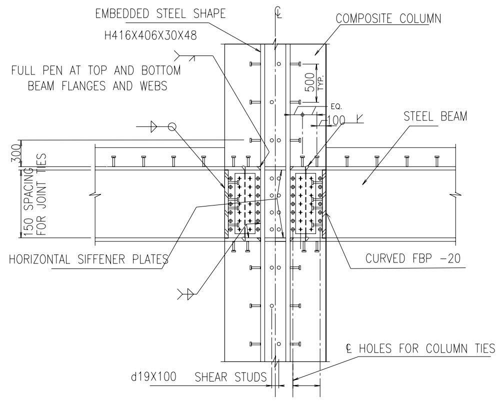


Fig 3.1 Beam-Column Connection

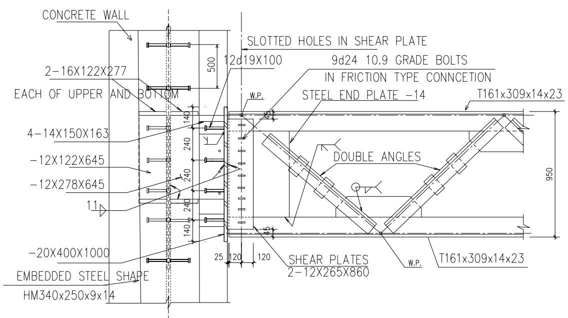


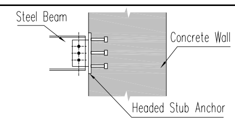
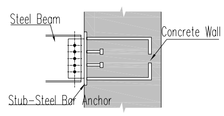
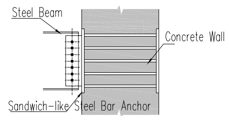
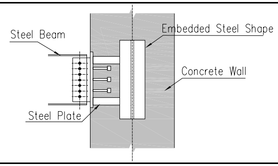
Fig 3.2 Steel Truss-Concrete Wall Connection

3.2 Steel truss-concrete wall connection

During the analysis of the whole structure, the connection between steel truss (beam) and concrete wall is usually considered as in the form of hinged, while as the effect of the high strength bolts, the restraining moment still exist at the truss (beam) end. The finite element analysis indicated that the restraining moment is related with the thickness and height of the shear plate, and the net span of the truss. The results of the analysis and test showed that the restraining moment would be larger than the calculation value in Reference [6], which should paid attention to the design. Fig. 3.2 shows the detail of steel truss-concrete wall connection.

3.3 Embedded anchors for the connection between steel beam and concrete wall

Table 3.3 Different Types of Beam-Wall Connection

Type	Types of Steel Beam-Concrete Wall Connection	Mainly Applied to
1		Light Load Secondary Member
2		Ordinary Member
3		Major Member
4		Heavy Load Major Member

The embedded anchors for the connection between steel beam and concrete wall could adopt different types according to the magnitude of load applied on the beam, see Table 3.3. The experiments show that type 1, the headed stud anchor would lead to the cracking of the concrete wall and slippage of the studs under the cyclic reversed loading, see Figure 3.3. Type 2, stud-steel bar anchor and type 3, sandwich-like steel bar anchor, as well as type 4, the embedded steel shape, their failures mainly depended on the upper or lower embedded steel bars or steel plates. While type 4 its anchorage capacity, energy dissipation and ductility are better than the others, it is suggested using to the major bearing member in high seismic fortification areas.

4. CONCLUSION

(1) For dual system of frame-core wall structure, core wall is the key lateral load resistant member, which should be strengthened during design to ensure its ductility and bearing capacity. Analysis and test results show that the composite steel and concrete frame-core structure is adequate for high seismic region after rational design and properly embedded steel shapes within the core wall, which has better seismic performance, the interaction between core wall and its surrounding ductility moment-resistant frame is efficient.

(2)Usually the floor truss (beam)-concrete wall connection is assumed as hinged, however the results of finite element analysis and cyclic loading test of specimens as well as shaking table test show the partial restraining moment at the floor truss (beam) end still exist under the action of earthquake, it should be considered during design.

(3) The embedded anchor for the connection between floor truss (beam) and concrete wall should be adopted carefully. During design It is suggested to apply type 4 in Table 3.3 to the major bearing members in high seismic regions.

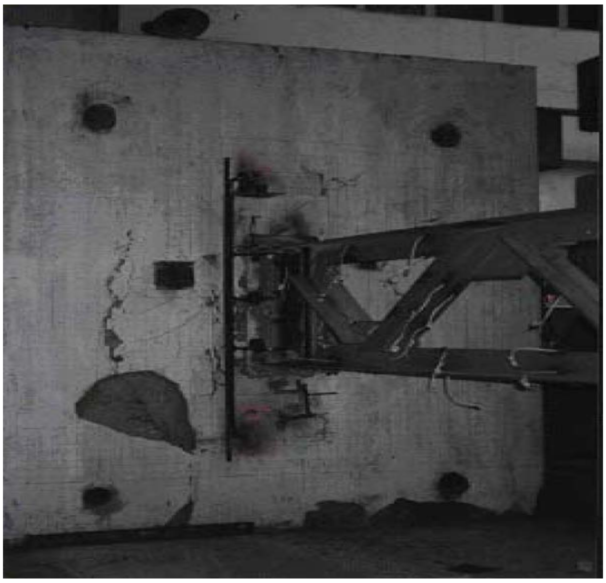


Fig 3.3 Failure of Headed Stud Anchor

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