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Safety Assessment and Strengthening of Existing Steel Frames Containing Semi-Rigid Joints

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ABSTRACT: Bund 18 is a historical building at the Bund of Shanghai. The structural system of the building consists of cast in-situ RC slabs, steel beams and columns, which is commonly used in other historical buildings in Shanghai. Through inspection and calculation for Bund 18, the safety state of the building is assessed. Based on the results of safety inspection and assessment, the strengthening and rehabilitation plan is proposed. By renovating some semi-rigid beam-column joints into rigid ones, adding some steel brace struts between the neighbouring columns and strengthening the masonry walls with single or double steel mesh reinforcement and cement mortar, the structural safety are substantially improved. The above strengthening plan is also applicable to other historical buildings which have the similar structural system.

1 INTRODUCTION

The collection of historic buildings along the Bund in Shanghai is the very thing that has attracted the luxury brand names. These buildings were built during the period from 1920 to 1939, featuring such architectural styles as Gothic, Baroque, Romanesque, Classic, Neo-classic and Renaissance. A walk down the Bund is like entering an exhibition of world architecture. As a kind of classic structural system, steel frames containing semi-rigid joints were extensively used in historic buildings during 1920-40s in Shanghai, such as Bund 18, Bund 23(Jiang *et.al*. 2005) and No. 60 on Jiujiang Road.

Bund 18 (Fig. 1) is located in the centre of the historic waterfront promenade of the Bund, which was built as the China headquarters for the Chartered Bank of India, Australia and China in 1923, designed by the British architectural firm Palmer & Turner Architects and Surveyors. In Bund 18, most of the surrounding marble walls need to be repaired due to the corrosive effects of humidity, but the intricate ceiling of the atrium still remains intact after 80-plus years. After two years of planning, inspection, safety assessment, strengthening, and restoration, Bund 18 was re-opened with a world-class dining, retail, flagships and entertainment in 2004.

During the inspection, assessment and strengthening of Bund 18, there exists many difficulties to get accurate and sufficient information about the building. Hence, it is hard to set up a calculation model which is totally consistent with the actual situation. As a historic building, how to protect it from the strengthening and make full use of it at the same time is also a big problem.

Based on the results of the safety inspection and assessment for Bund 18, this paper gives some advices for safety assessment and strengthening of steel frames containing semi-rigid joints which are commonly used in historical buildings constructed during the period from 1920s to 1940s in Shanghai.

2 INSPECTION OF THE BULIDING

2.1 Structural system of the building

The drawings of the building are preserved well in Shanghai Urban Construction Archives, but information about the steel columns and beam-column joints are insufficient. It is necessary to survey all of the building drawings. The surveyed drawings for the east elevation, the structural plane of the building, beam-column joints, steel beams and the typical column are shown in Fig. 2 to Fig. 5 respectively. From the drawings, it can be seen that cast in-situ RC slabs, steel columns and beams form the structural system of the building.

Beams and columns of the structure are surrounded with concrete and bricks respectively. The raft foundation of the building consists of RC slabs and wood piles which are all 7.62m long. Although used more than 80 years, the building still works well.

Figure 1 : Building of Bund 18

Figure 2 : East elevation view of Bund 18

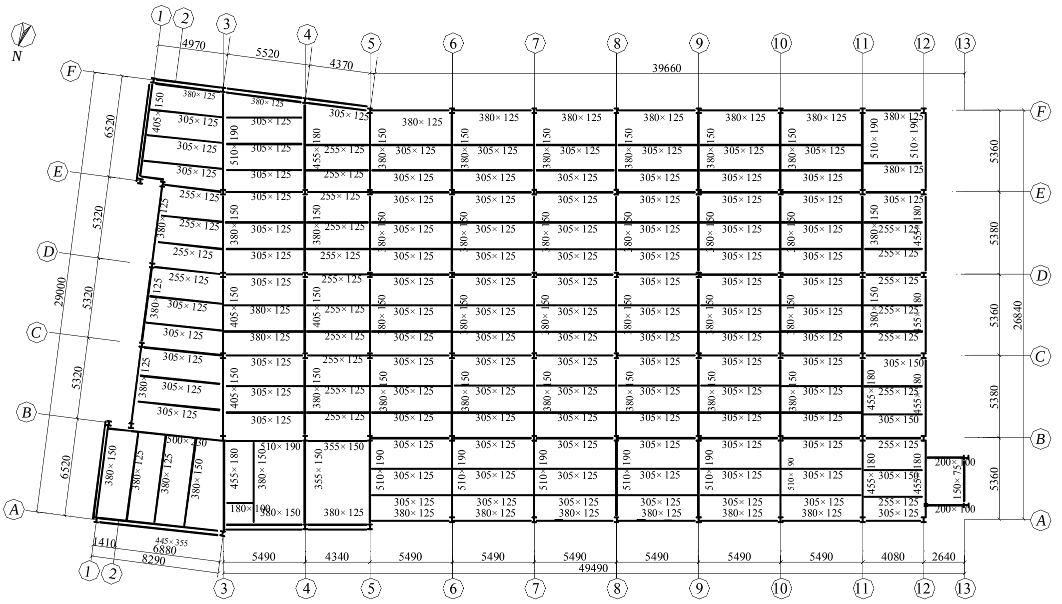


Figure 3 : Structural plane of the standard storey of the building

In Fig. 4, steel beams are connected with the steel column only by angle bars. So in this way the beam-column joints can be taken as hinged ones and the structure could be considered as a hinged frame for the analysis.

Provided that the steel beams have perfect connectors with the RC slab, the beam-column joints can also be taken as rigid ones, in such way, the structure could be considered as a rigid frame for the analysis. In fact, there are no connectors between the steel beams and the slab, but the concrete surrounding the steel beams and that of the slab are cast as a whole. As a result, the interaction between RC slab and steel beams make the structure a semi-rigid frame. Taking the above two extreme situations into consideration, the structural analysis could be carried out according to the rigid frame model and the hinged frame model, while the real situation of the structure may belong the one between the two extreme situations.

Slabs of the building are all cast in-situ, which are approximately 100mm thick. Loads on the floor are transferred from the beam to the girder, and then to the columns, so the vertical bearing system of the structure mainly consists of the steel columns.

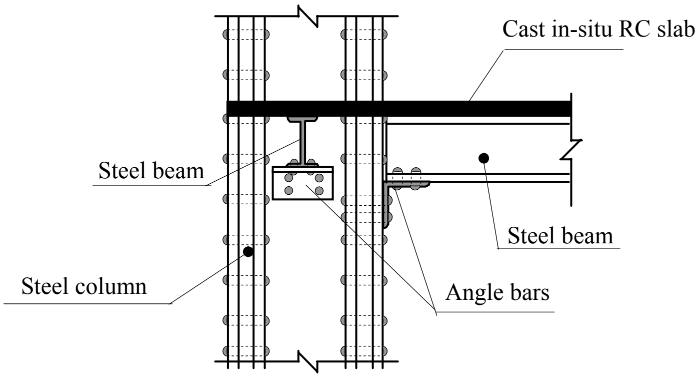


Figure 4 : Beam-column joints

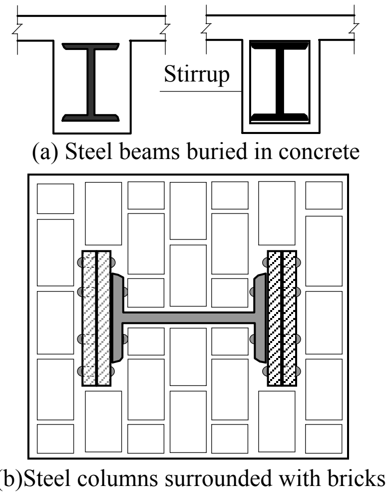


Figure 5 : Steel beams and the typical column

Steel beams are buried in concrete, similar to the composite steel–reinforced concrete beams (Fig. 5a). During the inspection, it has been found that the formed steel sections in the beam are all *I* shape, and the sizes of sections in beams are between 125×305mm~125×380mm, sections in girders are all 150×380mm. As shown in Fig. 5a, some formed steel sections are hooped with stirrups while some are not.

Steel columns are covered with bricks (Fig. 5b). It consists of *I* shape formed steel sections and steel plates, which are clinched together. The thickness of the steel plate is 12.7mm.

The concrete and bricks surrounding the beams and columns are preserved well and contribute to the structural safety. And the masonry walls filling the space bounded by the structural framing members, although considered nonstructural elements, tend to interact with the surrounding frame and improve the structural rigidity.

2.2 State of the building

During the inspection, it has been found that some of steel bars and formed steel sections in the building corroded. Because of the loss of protected cover, bars at the bottom of slabs and formed steel sections in the base room, where is humid, corroded severely(Fig. 6a). Due to the same reason, steel bars of the roof in the southeast corner corroded as well as the steel bars inside the stair beams and balcony slabs in the west side of the building(Fig. 6b).

The base room between 1 and 8 axis is pervaded with water. After pumping out water, it can be seen that there is no cracks on the slab and beams of the foundation, and water comes from the wall of the basement and accumulates in the base room.

Cracks have been found in the area of 5-11-*C-D* on the slab of every storey, where is the aisle of the building. The distribution of cracks in other places could not be observed because of the ceiling and wood floor. Most cracks are in south-north direction and distribute evenly, which may suggest that the crack is caused by the shrinkage of concrete.

The uneven settlements of the building which were obtained using the SOKKIA C40 high precision level are shown in Fig. 7. The results show that the lowest point is located in the southeast of the building.

2.3 Material properties of the building

Based on the characteristics of the building and the inspection conditions, 17ϕ100 cylinder samples were taken out from the concrete slabs, while 10ϕ100 cylinder samples from the concrete beams. The test results show that the maximum carbonization depth of the concrete is bigger than the radius of the samples, which means that the concrete are totally carbonized.

The strength of the concrete is evaluated by testing the cylinder samples, which shows that the cubic compressive strength of the concrete of the slab is between 15.3 ~ 35.1MPa, and the cubic concrete compressive strength of the beams and columns in the base room is between 29.0MPa ~ 43.0MPa.

3 samples of the formed steel sections and 2 samples of the steel bars are taken out from the base room and tested to get the strength of the steel, which shows that the ultimate strength is 287MPa, and the yield strength is approximately 201MPa.

(a) Corroded formed steel sections in the base room (b) Corroded steel bars in the roof

Figure 6 : Corroded formed steel sections and steel bars

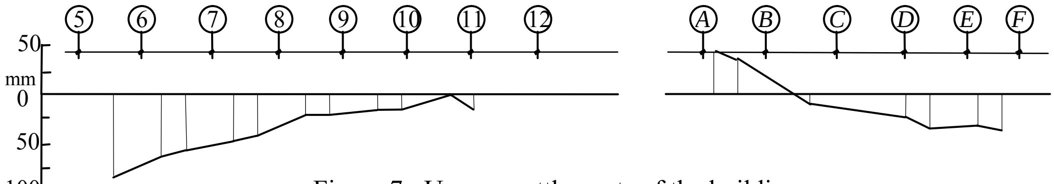


Figure 7 : Uneven settlements of the building

3 BEHAVIOR OF THE STRUCTURE UNDER VERTICAL LOADS

3.1 Calculation for RC slabs

Through inspection, it has been found that RC slabs of the building are cast in-situ. Calculation model for the slab is adopted as the continuous beam model, which takes into account the worst arrangement of live loads. The calculation results are shown in Table 1.

From the table, it can be seen that some of moments at the end are bigger than the cracking moment, which means that some cracks would occur at the top of slabs, as found in the inspection.

Table 1 : Comparison of internal force and bearing capacity of RC slabs under vertical loads (kN·m)

Floor	Section at the mid span				Section at the end		
	Moment for the side span	Moment for the middle span	Cracking moment	Bearing capacity	Negative moment	Cracking moment	Bearing capacity
1	3.81	2.73	5.22	9.10	4.90	4.70	8.05
2a	3.06	2.08	5.22	9.10	3.99	4.70	8.05
2b	3.56	2.51	5.22	9.10	4.60	4.70	8.05
3a	2.97	2.10	5.22	9.10	3.84	4.70	8.05
3b	3.58	2.62	5.22	9.10	4.57	4.70	8.05
4	3.06	2.30	5.22	9.10	3.99	4.70	8.05
5a	3.31	2.02	4.56	8.95	4.29	4.26	7.97
5b	4.01	2.99	4.56	8.95	5.09	4.26	7.97
Roof	3.62	2.41	5.22	9.10	4.74	4.70	8.05

3.2 Calculation for beams

Since the beams and columns are connected by angle bars, the calculation model for beams could be considered as a single-span simply supported beam, which is the worst case for a symmetrical steel *I*-beam. The calculation results for the beams of No.8 axis are shown in Table 2. The contribution of the surrounded concrete is not taken into account during the calculation of bearing capacity for steel beams.

From the table, it can be seen that some beams on the roof can not meet the requirements of the bearing capacity, due to the additional service loads.

Table 2 : Comparison of internal force and bearing capacity of beams under vertical loads

Floor	Beam				Girder			
	Bending moment *M* and bearing capacity [*M*] at mid span (kN.m)		Shear force *V* and bearing capacity [*V*] at the end (kN)		Bending moment *M* and bearing capacity [*M*] at mid span (kN.m)		Shear force *V*and bearing capacity [*V*] at the end (kN)	
	M	[*M*]	*V*	[*V*]	*M*	[*M*]	*V*	[*V*]
1a	110	170	81	309	246	476	141	376
1b	140	170	102	309	325	476	184	376
2a	95	170	70	309	207	306	119	249
2b	106	170	77	309	233	306	133	249
3a	92	170	67	309	198	306	113	249
3b	104	170	76	309	229	306	131	249
4	101	170	73	309	220	306	126	249
5a	101	170	73	309	220	306	126	249
5b	113	170	82	309	252	306	144	249
Roof a	110	98	80	157	244	287	139	212
Roof b	115	98	84	157	257	287	147	212

3.3 Calculation for columns

The calculation results of columns are shown in Table 3. From the table, it can be seen that the axial forces *N* of all columns are smaller than the corresponding bearing capacity [*N*].That is to say, all columns can meet the requirements of the bearing capacity under vertical loads, as found in the inspection.

Table 3 : Comparison of internal force *N* and bearing capacity [*N*] of columns under vertical loads (kN)

Column	1*A*		1*B*		2*B*		2*C*		3*A*		3*B*		3*C*	
	N	[*N*]	*N*	[*N*]	*N*	[*N*]	*N*	[*N*]	*N*	[*N*]	*N*	[*N*]	*N*	[*N*]
Floor 5	-	-	-	-	379	3560	454	2560	521	4380	538	4308	471	2200
Floor 4	383	4308	458	4308	712	3560	838	2560	905	7080	897	4308	830	2200
Floor 3	662	4308	788	3560	1043	4308	1212	3560	1286	7080	1254	5760	1187	3560
Floor 2	925	5760	1154	3560	1386	5760	1615	3560	1682	7080	1609	7080	1542	3560
Floor 1	1362	7080	1623	4308	1839	5760	2100	4308	2167	7080	1944	7080	1927	4308

Column	3*F*		4*A*		4*B*		5*A*		5*B*		6*A*		6*B*	
	N	[*N*]	*N*	[*N*]	*N*	[*N*]	*N*	[*N*]	*N*	[*N*]	*N*	[*N*]	*N*	[*N*]
Floor 5	-	-	415	3560	422	3560	420	4308	402	3560	443	2200	449	2200
Floor 4	384	3560	781	3560	771	3560	799	4308	752	3560	843	2200	839	2200
Floor 3	765	4308	1145	5760	1120	4308	1165	5760	1101	5760	1240	2560	1230	3560
Floor 2	1161	5760	1497	5760	1418	5760	1543	5760	1399	5760	1625	2560	1562	3560

Floor 1	1645	5760	1944	5760	1763	5760	2008	7080	1743	7080	2110	3560	1947	4308
Column	6*C*		10*A*		10*B*		11*A*		11*B*		12*A*		12*B*	
	N	[*N*]	*N*	[*N*]	*N*	[*N*]	*N*	[*N*]	*N*	[*N*]	*N*	[*N*]	*N*	[*N*]
Floor 5	449	2200	443	2200	449	200	408	2560	413	2560	364	2560	425	4308
Floor 4	885	2200	843	2200	885	2200	796	2560	812	2560	707	2560	829	4308
Floor 3	1275	3560	1285	2560	1320	3560	1181	3560	1212	4308	1049	4308	1231	5760
Floor 2	1645	3560	1689	2560	1690	3560	1551	3560	1552	4308	1389	4308	1620	5760
Floor 1	2061	4308	2212	3560	2152	4308	2040	4308	1988	7080	1854	4308	2130	5760

4 SEISMIC BEHAVIOR OF THE STRUCTURE

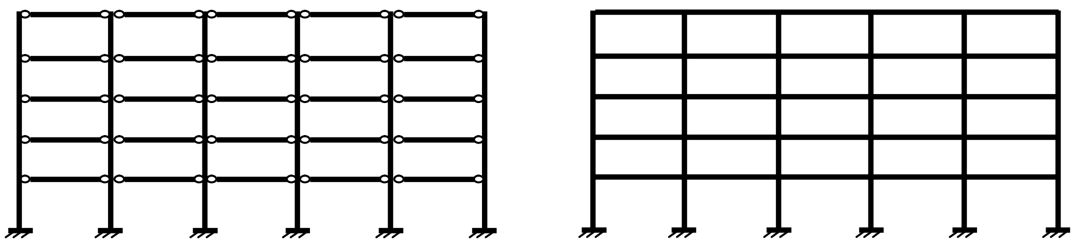
Under the guideline of the Chinese code, the seismic performance of the building is also calculated in two extreme situations (Fig. 8). It is assumed that steel columns would not buckle due to the well surrounded bricks. Evidently, the internal forces of the steel columns calculated by the hinged frame model (Fig. 8a) are bigger than those calculated by the rigid frame model (Fig. 8a). The internal forces of the steel columns at the base floor by the rigid frame model are shown in Table 4.

Table 4 : Internal forces of the steel columns at base floor under earthquake

Floor	Axis *A* (*F*)			Axis *B* (*E*)			Axis *C* (*D*)		
	M/kN.m	*N*/kN	σ/MPa	*M*/kN.m	*N*/kN	σ/MPa	*M*/kN.m	*N*/kN	σ/MPa
1	448	1241	254.6	1031	1424	378.5	1031	1425	378.6

From the table, it can be seen that the stress of the columns are all bigger than the strength of the steel (215MPa), which can not meet the needs of the bearing capacities.

The drifts of the building are also calculated as Table 5 shown. The calculation takes into account the contribution of surrounded bricks, while the drifts in the parenthesis are calculated by not considering the contribution. According to Chinese code, the limit value of elastic angular drift between adjacent floors is 1/550. From the table, taken as the hinged frame model, the drifts are much bigger than the limit value. However, taken as the rigid frame model, without the contribution of surrounded bricks considered, the drifts are also bigger than the limit value, while with the contribution of surrounded bricks considered, the drifts are smaller than the limit value.



(a) The hinged frame model (b) The rigid frame model

Figure 8 : Analysis models of the structure

The real situation of the structure can be taken as a semi-rigid frame between the hinged and the rigid one, as indicated above. By comparison of the calculation results in the Table 5, it can be inferred that the lateral rigidity can be improved efficiently by renovating the existing column-beam joints into rigid ones. In addition, it is feasible to reduce the drifts under action of the earthquake by making full use of the contribution of the surrounded bricks for the columns.

If the infill masonry walls can work well under action of the earthquake by reasonable strengthening, the lateral rigidity of the structure can be further improved (EI-Dakhakni et al. 2004).

Table5 : Drifts of the building under earthquakes

Floor	The hinged frame model			The rigid frame model		
	Elastic drift of floor(mm)	Elastic drift between floors (mm)	Elastic angular dirift between floors	Elastic drift of floor(mm)	Elastic drift between floors (mm)	Elastic angular drift between floors
5	322.48(890.05)	96.00(264.97)	1/55(1/20)	31.63(87.27)	3.59(9.90)	1/1480(1/536)
4	226.48(625.08)	79.00(218.05)	1/62(1/23)	28.04(77.37)	4.85(13.38)	1/1014(1/367)
3	147.48(407.03)	64.24(177.31)	1/75(1/27)	23.19(63.99)	4.46(12.31)	1/1091(1/395)
2	83.24(229.72)	52.61(145.19)	1/104(1/38)	18.73(51.68)	7.11(19.62)	1/772(1/280)
1	30.63(84.53)	30.63(84.53)	1/259(1/94)	11.62(32.06)	11.62(32.06)	1/684(1/248)

5 STRENGTHENING OF THE BUILDING

5.1 Strengthening design

Based on the results of the safety inspection and assessment, the strengthening and rehabilitation plan was made. Besides cleaning and restoration, some semi-rigid beam-column joints were renovated into rigid ones, some steel brace struts were added between the neighbouring columns, masonry walls were strengthened with single or double steel mesh reinforcement and cement mortar, bricks surrounding some columns were removed and concrete were cast around the columns. The typical strengthening plan for joints, struts, walls, beams and columns are as shown in Figs. 9-12.

5.2 Calculation of the building after strengthening

After strengthening according to the plan, the architectural style is kept unchanged, but the structural safety is substantially improved.

Table 6 shows the natural frequencies of the building. From the comparison, it can be seen that the natural frequencies of the building are increased after strengthening, which can be inferred that the rigidity and the capacities to resist lateral load improved dramatically.

Table 6 : Comparison of natural frequencies before and after strengthening

Results	Natural frequencies(f_3 is the first torsional natural frequency)			
	f_1 (Hz)	f_2 (Hz)	f_3 (Hz)	f_3/ f_1
Before strengthening	0.5286	0.5542	0.7435	1.4065
After strengthening	0.7344	0.7483	0.9957	1.3513

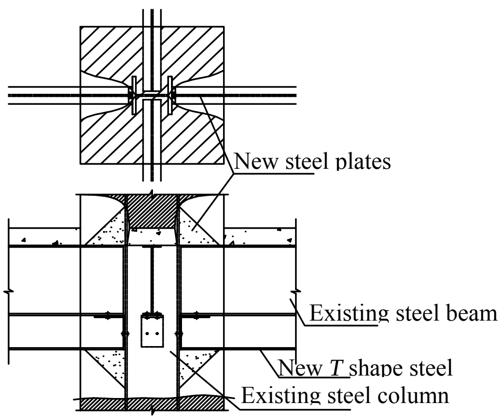


Figure 9 : Strengthening of the joint

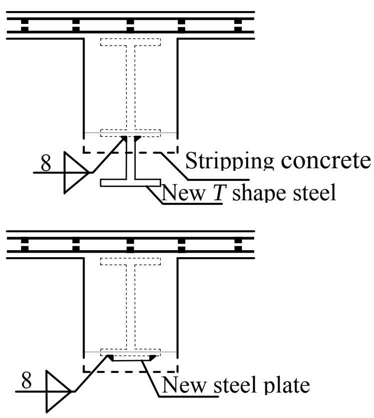


Figure 10 : Strengthening of the beam

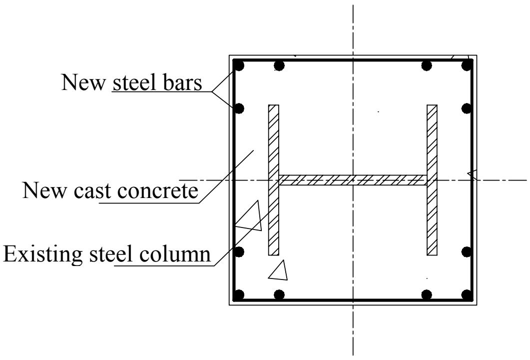


Figure 11 : Strengthening of the column

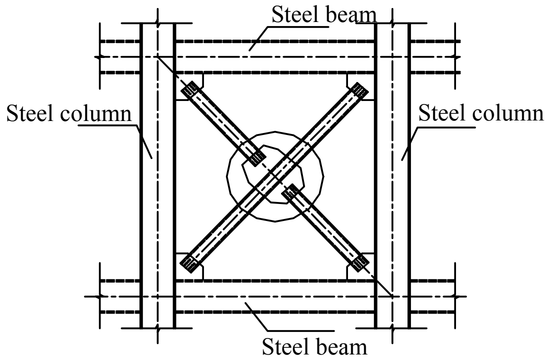


Figure 12: Steel brace struts

Table 7 shows the drifts of the building under action of the earthquake before and after strengthening when infill masonry walls and the surrounded bricks for the steel columns are still working elastically, while Table 8 shows the drifts when the walls and bricked are out of work. It is calculated by the hinged frame model before strengthening, while the strengthened joints are taken as rigid ones after strengthening. It can be seen from Table 8 that the drifts is so big that the structure may with renovated rigid joints collapse if infill masonry walls are out of work. Table 7 shows that the drifts are smaller than the limit value after strengthening, taking into account the contribution of the strengthened infill masonry walls and the surrounded bricks. It is verified by the calculation and the comparison that the proposed strengthening plan is efficient.

Table 7 : Comparison of drifts considering the contribution of infill masonry

Results	Maximum elastic angular drift between floors		Maximum elastic drift between layers/ Average elastic drift between floors	
	d_x/h	d_y/h	$d_{x\text{-max}} / d_{x\text{-ave}}$	$d_{y\text{-max}} / d_{y\text{-ave}}$
Before strengthening	1/464	1/419	1.21	1.65
After strengthening	1/749	1/581	1.26	1.64

Table 8 : Comparison of drifts not considering the contribution of infill masonry

Results	Maximum elastic angular drift between floors		Maximum elastic drift between layers/ Average elastic drift between floors	
	d_x/h	d_y/h	$d_{x\text{-max}} / d_{x\text{-ave}}$	$d_{y\text{-max}} / d_{y\text{-ave}}$
Before strengthening	1/13	1/20	1.06	1.28
After strengthening	1/27	1/182	1.04	1.14

6 CONCLUSIONS

The structural system of Bund 18 is commonly used in other historical buildings in Shanghai. By inspecting, assessing and strengthening for it, some effective solutions are proposed which is beneficial to the protection of similar historical buildings. These solutions include renovating the semi-rigid joints into rigid ones, strengthening the masonry walls, beams and columns, and adding steel brace struts. The applicability of the solutions has been verified by the use of the building.

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