Paper No. 1399. (quote when citing this article)
Eleventh World Conference on Earthquake Engineering
ISBN: 0 08 042822 3

INVESTIGATION OF A BRICK MASONRY BUILDING COMPLETED IN 1895

S.TOMISIMA and K.UDAGAWA
Taisei corporation
1-25-1 Nishi Shinjuku Sinjuku-ku
Tokyo,163-06
JAPAN
T. SHIMIZU and A.SATO
Ministry of Construction, Japan

ABSTRACT

Recent severe damage to old brick masonry building in Japan as a result of even moderate earthquakes has emphasized the need to retrofit such buildings to enhance their seismic performance.This paper describes a collaborative project to investigate the seismic performance and appropriate retrofit measures for a famous 3-story brick masonry building,130m long,45m wide and 15m high and containing many interior walls. The building was completed in 1895 and is the only remaining masonry structure in a Tokyo business district where it is part of a government offices complex. This paper consists of two parts. Part 1 provides detailed information on the strength of the brick walls. Part 2 examines the seismic performance of the building by using earthquake response spectrum methods and the strength results.

KEYWORDS

Retrofit; masonry building; brick wall tests; dynamic analysis; earthquake response; seismic performance

PART 1. FLEXURAL,SHEAR AND COMPRESSIVE STRENGTHS OF BRICK WALLS

1.Test Program

To evaluate the seismic resistance of the masonry,flexural ,shear and compressive monotonic loading tests were conducted on the walls. The walls were constructed of bricks 240*115*65mm with 10 mm thickbed joints. And tests,as follows,were conducted on the actual brick walls and on 380 mm cubic specimens cut from the walls:

a. In-plans flexural and shear tests of the walls (Fig.1 and 2);
b. Double shear tests on cubes loaded to the horizontal joints (Fig.3);
c. Compressive tests on cubes loaded perpendicular to the horizontal joints; and
d. Compressive,spliting and modulus of rupture tests on individual bricks

2.Test Results

The setup for the two in－plane shear tests is shown in Fig.2. The loading jack was inserted in a 800*1100 mm hole cut in the wall and the area between that jack and a 2400*100 mm slot also cut in the wall was preloaded to a mean vertical stress of 0.4 Mpa. Horizontal load,Q,versus horizontal center displacement, δ , relationships obtaned from this test are shown in Fig.4. The slope of the curves,which represents the stiffiness of the walls,gradually decreased as the loading increased. The load carrying capacities degraded asymptotically once severe diagonal shear cracks initiated alogn the joints. For specimen B-1 the maximum load was 230 KN at a forizontal displacement of 9.9 mm.

In the double shear tests,displacements were measured parallel (slip) and perpendicular(separation)to the horizontal joints. Fig.5 are the resultant load－displacement relationships for specimen C－1. Slips and

separations were not observed until immediately after the maximum load of 98 KN was reached. Then large plastic displancements occurred without significant reduction in load carrying capacity. The other three specimens exhibited similar behaveior but their capacities and stiffnesses were slightly smaller than for C－1. The mean shear strength at the joints derived from this test was 0.28 Mpa.

Shown in Fig.6 is a comparison of the stress－strain relationships from the compressive tests. The vertical strains at the peak load for each specimen differ due to differences in the maximum strengths of the fragile joint mortar. The initial stiffness for the walls,defined as the secant modules at one third of the maximum strength,averaged 2800 Mpa.

3.Summary and Conclussions

The test results showed that the strength of the joint mortar had a singnificant effect on the behavior of the brick walls. That strength was reduced by long term deterioration effects. The following conclusions were drawn as to mechanical properties of the brick walls:

1. Flexural strength	σ t =0.15 Mpa;
2. Shear strength	τ u=0.38 Mpa;
3. Compressive strength	σ u=6.4 Mpa; and
4. M odulus of elasticity	E w=2800 Mpa

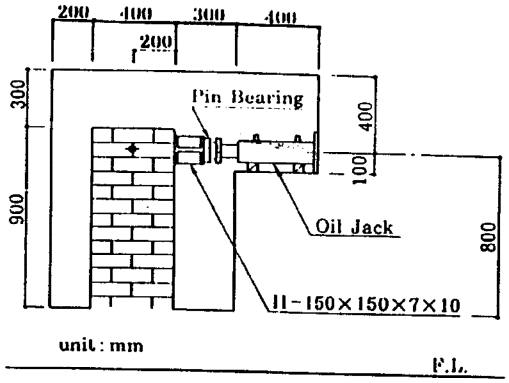

Fig.1 In–plane flexural test

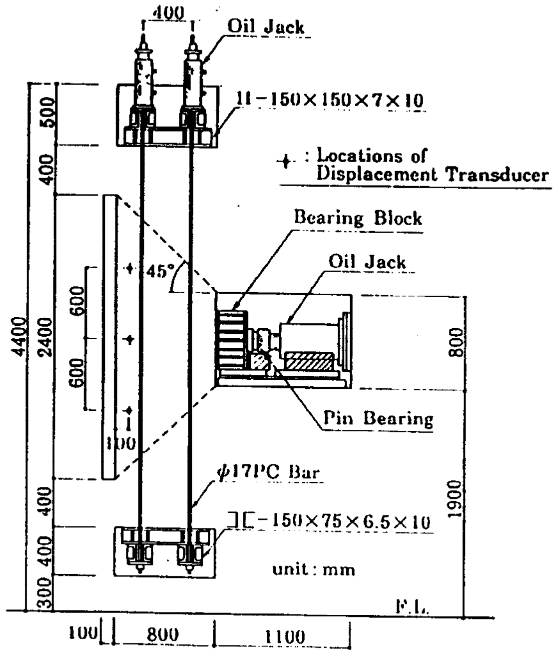

Fig.2 In–plane shear test

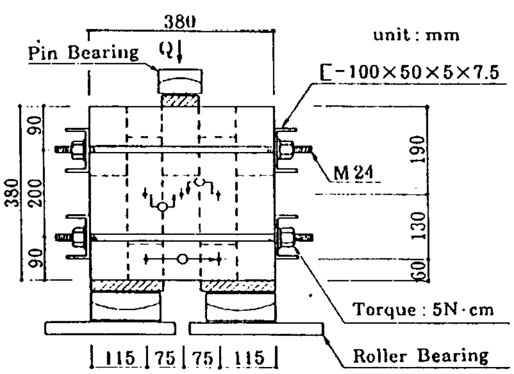

Fig.3 Double shear test

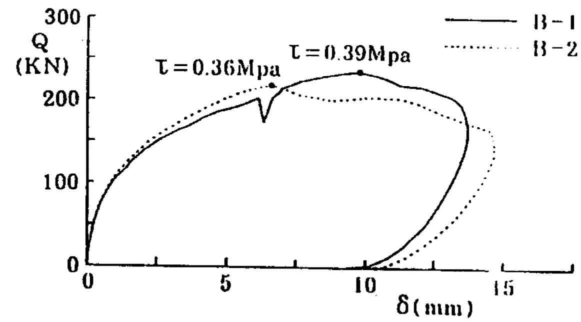

Fig.4 Load–displacement relationships for In–plane shear test

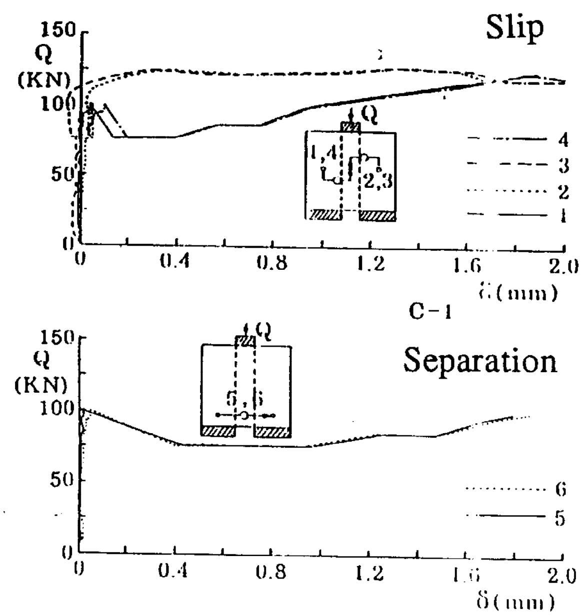

Fig.5 Load–displacement relationships at the joints

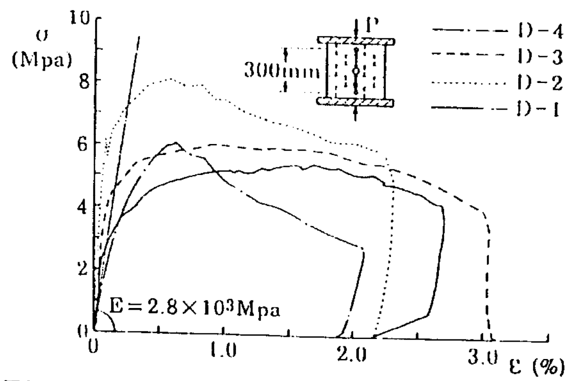

Fig.6 Stress – strain relationships from compressive tests

PART 2. SEISMIC SAFETY EVALUATION

Introduction

This papar describes the seismic appraisal of exisiting masonry building and the measures to ensure the structural meets modern Tokyo seismic requirements.

1. Response Analysis of the building

Fig.7 shows the first plan of this building. As the structural charachteristic in the plan,X and Y directions are differrent,separate models were created for each direction (see Fig.8). Each floor was assumed to consists of 5 lumped masses, connected by assumed stiffness for floor slab derived from the test – recorded stiffness value for the wall. Thus vertically,the masses are connected by the brick wall stiffness value based on the shear modulus, and horizontaly the masses connected by the floor slab stiffness having both shear and axial components.

The calculation models are shown Fig.9. By Comparing with the buildings dynamic charactristics,the input seismic waves adopted for analysis were ELCENTRO(1940NS),HACHINOHE (1968 NS),TAFT (1952 EW) and TOKYO (1956 NS). The fundamental natural frequency of the structure was calculated as 5Hz(approx.) and the peak value of input acceleration normalized to 200 cm／s ^{2}.The base of the structure's foundation was assumed as fixed against rotation in consideration of the restraint provided by the soil.

From the analysis,the maximum response acceleration in the X direction was 561 cm／s(TAFT),representing an amplification factor of 2.81,and in the Y direction, was 610 cm／s ^{2} (HACHINOHE),an amplification of 3.05 (Table 1.)

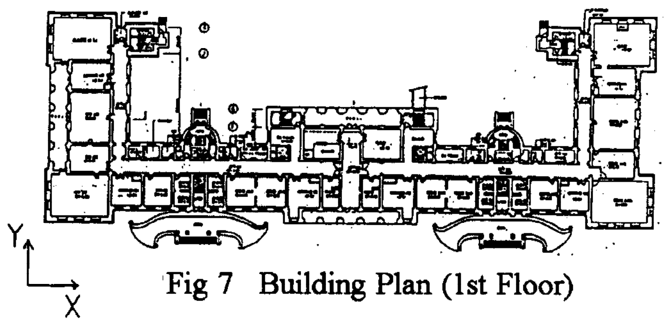

Fig 7 Building Plan (1st Floor)

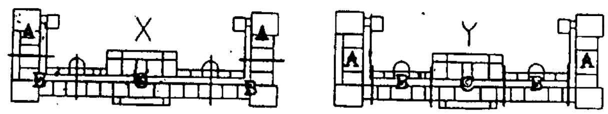

Fig 8 Building Sub-division for Modeling

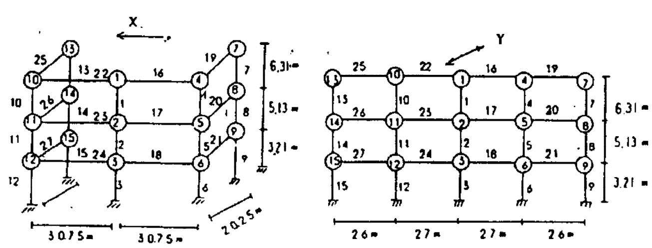

Fig 9 Building Model for Calculation

Table 1 Maximum Response of Mass Points

Max. Response		X- direction TAFT 1952 EW 200cm/s²						Y- direction HACHINOHE 1968 NS 200cm/s²					
Block	No.	DISP.	(TIME)	VEL.	(TIME)	ACC.	(TIME)	DISP.	(TIME)	VEL.	(TIME)	ACC.	(TIME)
C	1	0.33	(4.69)	11.1	(4.64)	406.	(4.60)	0.64	(4.16)	14.2	(4.10)	580.	(4.15)
	2	0.20	(4.69)	6.6	(4.64)	272.	(4.69)	0.46	(4.16)	10.3	(4.10)	462.	(4.16)
	3	0.08	(4.69)	2.5	(4.64)	202.	(3.71)	0.15	(4.16)	3.1	(4.10)	278.	(4.16)
B	4	0.37	(4.69)	12.4	(4.65)	451.	(4.69)	0.67	(4.16)	14.9	(4.10)	610.	(4.15)
	5	0.25	(4.69)	8.0	(4.65)	319.	(4.69)	0.43	(4.16)	9.6	(4.10)	438.	(4.15)
	6	0.09	(4.69)	2.9	(4.65)	205.	(6.55)	0.18	(4.16)	4.0	(4.10)	295.	(4.16)
A	7	0.47	(4.70)	15.7	(4.65)	561.	(4.69)	0.50	(4.16)	11.1	(4.10)	490.	(4.15)
	8	0.31	(4.70)	10.1	(4.65)	386.	(4.69)	0.34	(4.16)	7.5	(4.10)	389.	(4.15)
	9	0.10	(4.70)	3.3	(4.65)	214.	(6.56)	0.13	(4.16)	2.7	(4.10)	265.	(4.15)
B'	10	0.40	(4.69)	13.4	(4.65)	484.	(4.69)	0.63	(4.16)	14.1	(4.10)	587.	(4.15)
	11	0.26	(4.69)	8.6	(4.65)	339.	(4.69)	0.40	(4.16)	8.9	(4.10)	417.	(4.15)
	12	0.10	(4.69)	3.1	(4.65)	210.	(6.55)	0.17	(4.16)	3.7	(4.10)	286.	(4.16)
A'	13	0.47	(4.70)	15.7	(4.65)	558.	(4.69)	0.47	(4.16)	10.4	(4.10)	472.	(4.15)
	14	0.31	(4.70)	10.1	(4.65)	387.	(4.69)	0.29	(4.16)	6.4	(4.10)	355.	(4.15)
	15	0.11	(4.69)	3.3	(4.65)	216.	(6.55)	0.11	(4.16)	2.3	(4.09)	253.	(4.15)

2. Structual Assessment from Results of Response Analysis

Masonry allowable stresses are obtained directly from testing and divided by a safety factor of 1.5 for short term(seismic) conditions.(Table 2) Maximum responses shear forces and average shear stresses,based on the 200 cm／s ^{2} acceleration,are shown in Table 3. Areas exceeding the allowable stress are also indicated(mark *). The stresses from the maximum response forces in the slab are in all cases less than allowble stresses. From the results discussed,it was decided to reinforce those walls which were shown to be over stressed,by constructing reinforced concrete strengthning walls connected by shear stud bolts to the existing walls. The maximum shear stress in the upgraded wall,which in all cases, are less than the allowable stresses. Regarding out-of-plane direction(perpendicular to masonry walls),shear forces based on the maximum response acceleration of inplane direction are adopted as the external forces to check the wall bending bearing capacity (Fig.10). By means of this calculation,at thin walls such as 380 mm thk and 510 mm thk., steel plates(3.2 mm thk.) are installed at bothsides of the wall surface to strenghthen flexural capacity.

Table 2 Allowable Stress (Mpa)

		Testing Value	Short Term
Compression		6.0	4.0
Bending		0.15	0.10
Tension		0.15	0.10
Shear	3rd fl.	0.30	0.20
	2nd fl.	0.35	0.23
	1st fl.	0.40	0.27

Table 3 Maximum Shear Stresses in Wall

X-Dir	FL	Mem. No.	Weight (KN)	Shear Area (m^2)	Sh. Force (KN)	Shear Stress (MPa)	
A	3	7	8480	20.7	4150	0.20	
	2	8	9650	24.1	7520	0.31	*
	1	9	8750	35.4	9500	0.27	
A'	3	13	8780	22.5	4390	0.20	
	2	14	10570	26.0	8040	0.31	*
	1	15	9720	37.6	10300	0.27	*
B	3	4	11360	34.6	5660	0.16	
	2	5	13820	41.3	10010	0.24	*
	1	6	11830	51.9	12990	0.25	
B'	3	10	11430	31.9	5610	0.18	
	2	11	14010	39.7	10040	0.25	*
	1	12	13060	49.7	13290	0.27	
C	3	1	26410	75.8	13240	0.17	
	2	2	33150	115.9	23730	0.20	
	1	3	29230	140.1	31330	0.22	

Y-Dir	FL	Mem. No.	Weight (KN)	Shear Area (m^2)	Sh. Force (KN)	Shear Stress (MPa)	
A	3	7	14660	49.6	8700	0.18	
	2	8	16750	54.1	16070	0.30	*
	1	9	14990	73.3	20630	0.28	*
A'	3	13	15020	43.2	8510	0.20	
	2	14	18490	64.5	16180	0.25	*
	1	15	17200	87.9	21180	0.24	
B	3	4	10350	17.6	4740	0.27	*
	2	5	13440	29.7	10190	0.34	*
	1	6	11180	31.6	12790	0.40	*
B'	3	10	10370	18.2	4790	0.26	*
	2	11	12180	29.7	9370	0.32	*
	1	12	11160	31.6	11890	0.38	*
C	3	1	16050	49.3	9700	0.20	
	2	2	20340	43.0	18680	0.43	*
	1	3	18060	74.6	24220	0.32	*

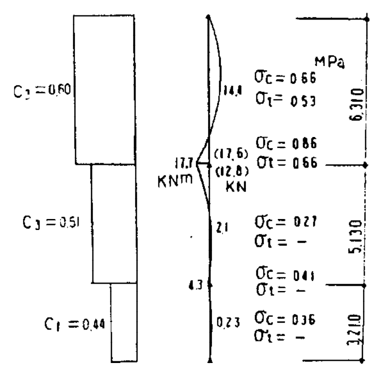

Fig 10 Building Diagram Perpendicular to Wall

3. Conclusion

From the response analysis,it was shown the natural period of the structure is 0.2 seconds as compared to 0.33 seconds of the surrounding soil. This differennce would partly explain why the building didn't suffer any severe damages when struck by the Kanto earthqake.Thus,structural stability is maintained for an input level up to 200 cm／s ^{2} at the ground surface. Further,if the ultimate strength is assumed to be equivalent to the material strength obtained from testing and some of the walls are upgraded as described above,the structure should withstand ground surface accelerations up to 300-400 cm／s ^{2}.

Despite the building's 100 years of age,it can be seen that this famous old building can remain in their masonry building for many years to come. This study also illustrates hou masonry (or indeed other materials) can be engineered to create seicmic resistant structures.