# Seismic Evaluation of a Brick Masonry Building of 1895

## Comportement aux séismes d'un bâtiment en maçonnerie de 1895

## Erbebenverhalten eines Backsteingebäudes von 1895

**Kazuhiro KANADA**
Taisei Corporation
Japan

**Kimio UDAGAWA**
Taisei Corporation
Japan

**Toyakazu SHIMIZU**
Ministry of Construction
Japan

**Akiyoshi SATO**
Ministry of Construction
Japan

### 1. Introduction

This papar describes the seismic appraisal of exisiting masonry building and the measures needed to ensure the structural meets modern Tokyo seismic requirements. Fig. 1 shows the first plan.

### 2. Response Analysis of the building

As the structural characteristic in the plan, X and Y directions are different, separate models were created for each direction (see Fig. 2). 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 (see Part 1) . Thus vertically, the masses are connected by the brick wall stiffness value based on the shear modulus, and horizontally the masses connected by the floor slab stiffness having both shear and axial components.
The calculation models are shown Fig. 3.

By Comparing the buildings dynamic characteristics, the input seismic waves adopted for analysis were EL CENTRO (1940 NS), HACHINOHE (1968 NS), TAFT (1952 EW) and TOKYO (1956 NS).

The fundamental natural frequency of the structure was calculated as 5 Hz (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 and the soil's damping factor ratio assumed as 7%.

From the analysis, the maximum response anaylsis in the X direction was 561 cm/s$^2$ (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)

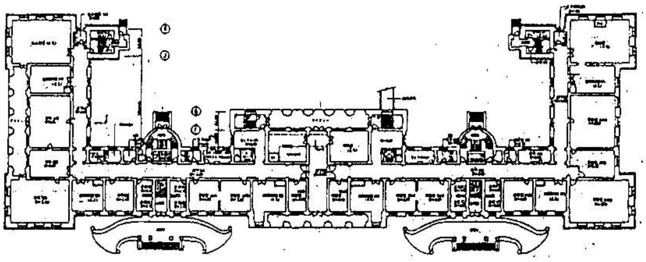

Fig 1 Building Plan (1st Floor)

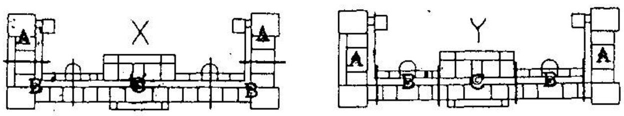

Fig 2 Building Sub-division for Modeling

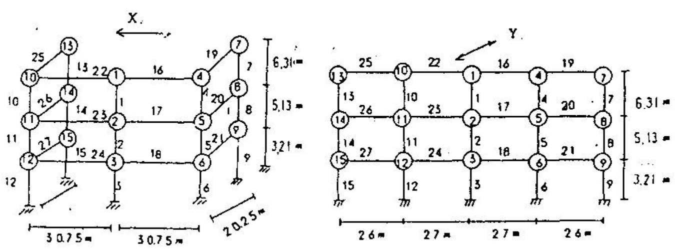

Fig 3 Building Model for Calculation

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) |

## 3. Structual Assessmemt 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 200cm/s² input 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 allowable stresses.

From the results discussed, it was desided 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.4). By means of this calculation, at thin walls such as 380mm THK.,510mm THK. steel plates (3.2mm THK.) are installed at both sides 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 | 5650 | 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 | * |

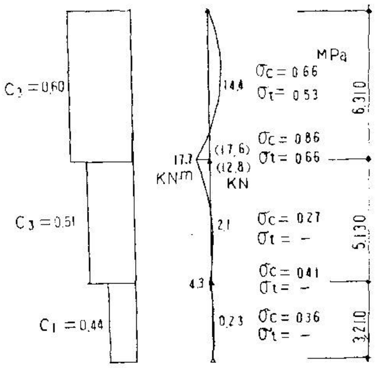


Fig 4 Bending Diagram Perpendicular to Wall

## 4. Conclusion

From the response analysis, it was shown the the natural period of the structure is 0.2 seconds as compared to 0.33 seconds for the surrounding soil. This large difference would appear to partly explain why the building didn't suffer any severe damage when struck by the Kanto earthquake.

Thus, structural stability is maintained for an input level up to 200 cm/s² 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 300-400cm/s².

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 how masonry (or indeed other materials) can be engineered to create seicmic resistant structures.