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# EVALUATION OF SEISMIC PERFORMANCE AND DEVELOPMENT OF RETROFITTING SCHEME FOR BRICK/BLOCK MASONRY BUILDINGS IN A MODERETE SEISMIC ZONE

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## SUMMARY

Seismic resisting capacity of existing brick/block masonry buildings in Shanghai, China is evaluated. Four apartment buildings are examined in this study. The seismic capacity is evaluated by an extended application of the seismic screening method proposed in Japan for reinforced concrete buildings. The evaluated seismic capacities are examined by a dynamic response analysis. Evaluating interstory deflection responses, seismic performance of these masonry buildings is examined during the small-scale, moderate-scale and large-scale ground motions, respectively, employed in the seismic design in Shanghai. Judged are these buildings seriously damaged during an intense ground motion. In this study herein, using one of traditional technologies, a retrofitting scheme is proposed for the buildings strengthened by placing new reinforced concrete shear walls along existing walls. The proposed retrofitting scheme is examined and verified by a dynamic response analysis.

## INTRODUCTION

During recent strong earthquakes such as those of the 1995 Kobe, Japan, 1999 Kocaeli, Turkey and Chi-Chi, Taiwan earthquakes, the 2000 Gujarat, India earthquake and the 2003 Bam, Iran earthquake, extensive damage is observed for existing buildings. The damage observed for existing buildings are significant, in particular, for those designed in accordance with the obsolete versions of seismic design codes and/or specifications, in which the updated information nor newly developed knowledge and aspects concerning with either seismology or earthquake engineering have not been taken into the procedures.

Since we have a large stock of existing buildings, most of which, in general cases, have inferior seismic capacities, it will be an important issue for us to improve the seismic capacity of those existing buildings not generating severe structural damage against a prospective seismic action. The countermeasures will lead to the disaster mitigation of building structures, and consequently to the reduction of human losses produced during a severe earthquake.

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Within the study presented herein, we discuss the items in the following:

(1) Evaluation of seismic capacities of existing buildings;
(2) Verification of the evaluated seismic capacities through a dynamic response analysis using a waveform of the design seismic action; and
(3) Development of seismic retrofitting and/or rehabilitation scheme for the buildings of which seismic performance capacities are judged less than required.

Seismic resisting capacities of the brick/block masonry buildings constructed in Shanghai, China are discussed. Four buildings are examined. They are the apartment buildings in the Tongji Village prepared for the university staff. Lateral load resisting systems of these buildings are either brick-wall or hollow concrete block-wall. Note that the seismic zone of Shanghai is considered moderate in the Chinese seismic design code.

Within the study presented herein: (1) first the seismic performance capacities of these buildings are evaluated; (2) secondly the evaluated performance is examined and verified through a dynamic response analysis using the ground motions for the high-rise building design in Shanghai; (3) thirdly the seismic performance of these buildings are examined whether they will be safe or unsafe during the seismic action in Shanghai; and (4) finally seismic retrofitting and/or rehabilitation schemes are proposed for the buildings that are judged unsafe through the dynamic response analysis carried out in the previous steps in the study.

## BUILDINGS EXAMINED WITHIN THE STUDY

Five buildings summarized in Table 1 in the following are examined. Four buildings are the apartment houses for the university staff, and the other is the classroom building within the campus of Tongji University, Shanghai. Photos 1 and 2 show the general view of the buildings examined herein of Apartment Building Type 79-02 with six stories height and Type 91-37 of seven stories height.

**Table 1. Summary of the Buildings Examined within the Study**

| Building Name/Code | Design or Construction Year | Number of Story | Features of Structural System |
|---|---|---|---|
| Apartment Type 79-01 | 1979 | 5 | (1) Hollow concrete block construction |
| Apartment Type 79-02 | 1979 | 6 | (1) Hollow concrete block construction for the 1st story and brick construction for the 2nd to 6th stories |
| Apartment Type 90-14 | 1990 | 6 | (1) Hollow concrete block construction for the 2nd to 4th stories, and brick construction for the 1st and those above the 4th story stories<br>(2) R/C columns are placed at ends of wall. |
| Apartment Type 91-37 | 1991 | 7 | (1) Brick wall system infilled within R/C column frame<br>(2) New construction building for the seven-story building. |

**Photo 1. Apartment Building: Type 79-02, Six-storied Building, Tongji Village, Tongji University Campus, Shanghai, China.**

**Photo 2. Apartment Building: Type 91-37, Seven-storied Building, Tongji Village, Tongji University Campus, Shanghai, China.**

## EVALUATION OF SEISMIC CAPACITY OF BUILDING

**Lateral strength of brick and hollow concrete block walls**

Based on both the drawing documents and the on-site observation upon the buildings, seismic capacities are evaluated. The seismic capacity is evaluated by employing the so-defined "first level screening" method utilized in practice for reinforced concrete buildings in Japan [1]. The assumptions in the following are made:

(1) Lateral strength of columns within the reinforced concrete frame, and that of walls of either brick or hollow concrete block are yielded by the cross-sectional area of members.
(2) Ductility index employed in the evaluation is determined as unity for both columns and walls.

Lateral strength of reinforced concrete columns and that of brick or hollow concrete block masonry walls are simply given by:

(1) Lateral strength of reinforced concrete columns is given with assumption that the strength of $0.98 N/mm^2$ can be taken by the unit cross-sectional area in $mm^2$.
(2) Lateral strength of brick walls $\tau_u$ is given by assuming the unit strength as in the following [2]:

$$\tau_u = f_v \gamma_f + 0.18 \sigma_o$$

where $\tau_u$ : lateral strength in stress of masonry wall;
$f_v$ : shear stress for design;
$\gamma_f$ : coefficient converting the shear stress for design into the standard shear stress; and
$\sigma_o$ : axial stress acting on the wall.

The coefficient $\gamma_f$ is taken as 1.5 in the evaluation, with which the shear stress will fall in the value not less than the standard shear stress with confidential level of 95 percent [2].

(3) The shear stress for design $f_v$ is given by the mortar class:
   1. For brick walls:
      $f_v$ = 0.18, 0.12 and 0.09 for mortar of M10, M5 and M2.5, respectively.
   2. For hollow concrete block walls:
      $f_v$ = 0.10, 0.07 and 0.05 for mortar of M10, M5 and M2.5, respectively.

The hollow concrete block wall (CB), the shear strength of which is less than that of brick wall (Brick), is applied for wall components in the lower story levels, i.e., the 1st and/or 2nd levels, since the compressive strength is higher than that of brick wall.

**Seismic resisting capacity of building with brick/block shear walls**

The seismic capacities of the buildings examined herein are summarized in Tables 2 and 3 for the longitudinal and transverse directions of building, respectively. The figures in the second columns are seismic performance index, representing the seismic capacity of the building. The capacities evaluated along the transverse direction of building are higher than those along the longitudinal direction of building, tendency of which is commonly observed for apartment buildings. Discussion hereinafter in this paper is performed on the seismic performance of the apartment buildings along their longitudinal direction.

**Table 2. Seismic Performance Index Eo: Longitudinal Direction**

| Story Number | Apartment Bldg. Type 79-01 | | Apartment Bldg. Type 79-02 | | Apartment Bldg. Type 90-14 | | Apartment Bldg. Type 91-37 | |
|---|---|---|---|---|---|---|---|---|
| 7 | - | - | - | - | - | - | Brick M5 | 0.61 |
| 6 | - | - | Brick M5 | 0.35 | Brick M10 | 0.57 | Brick M5 | 0.39 |
| 5 | CB M5 | 0.15 | Brick M5 | 0.22 | Brick M10 | 0.34 | Brick M5 | 0.30 |
| 4 | CB M5 | 0.10 | Brick M5 | 0.18 | CB M5 | 0.17 | Brick M5 | 0.26 |
| 3 | CB M5 | 0.09 | Brick M5 | 0.16 | CB M5 | 0.15 | Brick M5 | 0.24 |
| 2 | CB M5 | 0.09 | Brick M5 | 0.16 | CB M5 | 0.15 | Brick M5 | 0.23 |
| 1 | CB M5 | 0.09 | CB M5 | 0.12 | Brick M10 | 0.22 | Brick M10 | 0.26 |

CB: Hollow concrete block
M10, M5 and M2.5: Mortar quality class

**Table 3. Seismic Performance Index Eo: Transverse Direction**

| Story Number | Apartment Bldg. Type 79-01 | | Apartment Bldg. Type 79-02 | | Apartment Bldg. Type 90-14 | | Apartment Bldg. Type 91-37 | |
|---|---|---|---|---|---|---|---|---|
| 7 | - | - | - | - | - | - | Brick M5 | 0.73 |
| 6 | - | - | Brick M5 | 0.54 | Brick M10 | 1.06 | Brick M5 | 0.50 |
| 5 | CB M5 | 0.38 | Brick M5 | 0.33 | Brick M10 | 0.64 | Brick M5 | 0.38 |
| 4 | CB M5 | 0.26 | Brick M5 | 0.27 | CB M5 | 0.29 | Brick M5 | 0.33 |
| 3 | CB M5 | 0.23 | Brick M5 | 0.25 | CB M5 | 0.27 | Brick M5 | 0.31 |
| 2 | CB M5 | 0.23 | Brick M5 | 0.24 | CB M5 | 0.26 | Brick M5 | 0.30 |
| 1 | CB M5 | 0.24 | CB M5 | 0.19 | Brick M10 | 0.42 | Brick M10 | 0.42 |

CB: Hollow concrete block
M10, M5 and M2.5: Mortar quality class

## EVALUATION OF STIFFNESS OF BUILDING

Microtremor measurement has been carried out to find out the stiffness of the buildings examined herein. The Fourier ratios of motion observed at the top floor level compared to that at the 1st floor level are examined. The fundamental periods of the building and the amplification factor between the lowest and highest floor levels are summarized as in Table 4 for the building examined in this study.

**Table 4. Fundamental Period and Amplification Factor**

| Building Code | Number of Story | Const. Type | Fundamental Period (s) Trans. Direction (X) | Fundamental Period (s) Long. Direction (Y) | Amplification Factor Trans. Direction (X) | Amplification Factor Long. Direction (Y) |
|---|---|---|---|---|---|---|
| Apartment 79-01 | 5 | CB | 0.29 | 0.21 | 5.8 | 5.9 |
| Apartment 79-02 | 6 | CB+Brick | 0.33 | 0.34 | 7.7 | 11.7 |
| Apartment 90-14 | 6 | Brick+CB | 0.31 | 0.27 | 9.8 | 7.4 |
| Apartment 91-37 | 7 | Brick | 0.34 | 0.21 | 7.4 | 5.2 |
| North Building | 4 | Brick | 0.25 | 0.21 | 3.5 | 4.3 |
| Civil Engng Building | 3 | RC | 0.23 | 0.24 | 4.9 | 5.3 |

CB: Hollow concrete block

Figure 1 illustrates the correlation between the observed fundamental period of building obtained from the microtremor measurement and that speculated from the weight and estimated stiffness of the building. Note that the axis x designates the square root of the mass $m$ in kN of the building divided by the total cross-sectional area of brick and/or hollow concrete block walls $Aw$ in m$^2$.

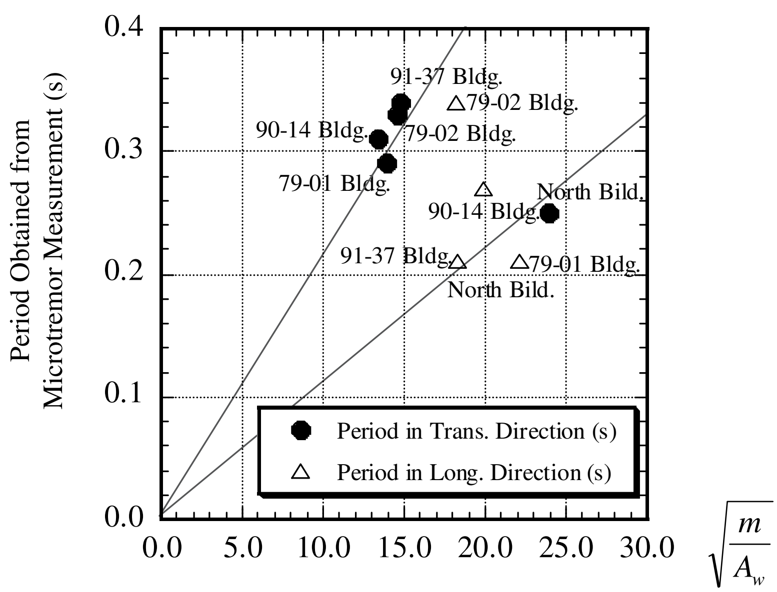


**Figure 1. Correlation of the observed fundamental period of building from the microtremor measurement with the mass and stiffness of building obtained from the cross-sectional area of wall components placed within the building. The notations $m$ and $A_w$ represent average mass of story in $kN$ average cross-sectional area in $m^2$ of wall components placed within the building taken across the story levels, respectively.**

A good correlation is found between the observed period and the estimated fundamental circular frequency angle $\omega o$ determined from the cross-sectional area of wall components, provided that the evaluation is carried our along with the transverse direction and longitudinal direction separately.

Further analyses will be needed, while we will reach to the conclusive remark that the fundamental period of the building can be estimated from the mass $m$ of the building and the stiffness of the building obtained from the cross-sectional area of wall components $A_w$ positioned within the building along the transverse or longitudinal direction.

## EVALUATION OF SEISMIC PERFORMANCE OF BUILDING

### Modeling of the buildings

The brick masonry buildings examined herein are represented by a MDOF oscillating system with shear mode shape [3]. The major structural properties with which the primary curves are prescribed are given as follows:

- (a) For masonry wall elements:
  - (1) Shear strength is determined from the lateral strength of masonry walls;
  - (2) Shear failure deflection of 1/300 is specified from the empirical research works on masonry wall components; and
  - (3) Hysteresis rule for the elements is represented by the so-called Origin-oriented model revealing inferior energy dissipation characteristics of masonry elements.
- (a) For reinforced concrete column elements:
  - (1) Ultimate strength is given by the lateral strength of columns;
  - (2) Ultimate failure deflection of 1/150 for flexural yielding columns; and
  - (3) Hysteresis rule is given by the so-named Takeda model representing a certain amount of energy dissipation.

Figure 2 shows the load-deflection characteristics determined for the masonry buildings with reinforced concrete column placed within the frame. Note that the shear failure deflection angle of 1/300 within this study is determined from experimental researches carried at the Tongji University, Shanghai, China.

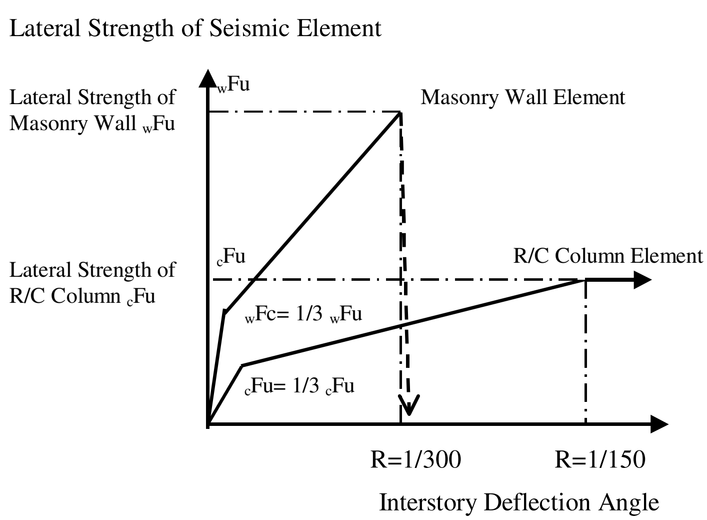


**Figure 2. Load-deflection characteristics determined for masonry wall and reinforced concrete.**

**Earthquake ground motions utilized in the analysis**

According to the seismic hazard evaluation in China, the City of Shanghai is located within the area where the seismic action of MM7 (Modified Mericall Intensity) is expected. The three levels of seismic action shall be in principle considered in China as follows.

(1) Small-scale ground motion: It corresponds to the seismic action against which the building can be subjected to a few times within the period of 50 years. During the specified ground motion, the building shall remain in an elastic range showing no structural damages.

(2) Moderate-scale ground motion: It corresponds to the ground motion, the probability of occurrence of which is 10% within the 50-year interval. For the moderate-scale ground motion, minor damages that can be repairable after the action can be generated during the seismic action.

(3) Large-scale ground motion: The motion is regarded as the maximum possible ground motion, the probability of occurrence of which is 2 to 3 % within the 50-year interval. Against the large-scale ground motion, structural damages can be generated, while the damage should not be generated which can produce human losses.

Three types of seismic motions are utilized in the dynamic response analysis in the following; i.e., a synthetic motion, SHW-1, the real motion recorded at the Hollywood during the 1952 Kern County Earthquake, SHW-2, and the real motion recorded at the El Centro during the 1996 Imperial Valley Earthquake, SHW-3, respectively.

**Response during the design ground motions**

***Response during the small-scaled design ground motions***

Table 5 summarizes the maximum interstory deflection angles of the buildings obtained when subjected to the small-scaled design ground motions SHW-1, SHW-2 and SHW-3. The maximum response of interstory deflection angles in the range of 1/1,400 – 1/2,100 can produce cracking within the wall elements. Since the maximum interstory deflection responses are one fifth to one seventh as large as the ultimate deflection angle, the buildings are considered not severely damaged during the small-scaled ground motions.

**Table 5. Maximum Interstory Responses in Deflection Angle during Small-scale Design Ground Motions**

| Story Number | Apartment Bldg. Type 79-01 | | Apartment Bldg. Type 79-02 | | Apartment Bldg. Type 90-14 | | Apartment Bldg. Type 91-37 | |
|---|---|---|---|---|---|---|---|---|
| | Long. | Trans. | Long. | Trans. | Long. | Trans. | Long. | Trans. |
| 7 | - | - | - | - | - | - | 1/27,000 | 1/11,000 |
| 6 | - | - | 1/19,000 | 1/19,000 | 1/38,000 | 1/17,000 | 1/7,200 | 1/6,900 |
| 5 | 1/2,900 | 1/11,000 | 1/6,500 | 1/10,000 | 1/21,000 | 1/9,800 | 1/4,100 | 1/3,200 |
| 4 | 1/1,700 | 1/4,000 | 1/2,500 | 1/6,900 | 1/2,000 | 1/2,500 | 1/3,000 | 1/2,400 |
| 3 | 1/1,500 | 1/3,300 | 1/1,900 | 1/5,100 | 1/1,800 | 1/2,200 | 1/2,700 | 1/2,100 |
| 2 | 1/1,600 | 1/3,700 | 1/1,800 | 1/4,900 | 1/1,800 | 1/2,300 | 1/2,700 | 1/2,100 |
| 1 | 1/2,000 | 1/4,000 | 1/1,400 | 1/3,000 | 1/6,200 | 1/4,400 | 1/3,000 | 1/4,800 |

***Response during the moderate-scaled design ground motions***

Table 6 tabulated the maximum interstory deflection angles of the buildings obtained when subjected to the moderate-scaled design ground motions. The maximum responses are generally given by the SHW-2 excitation. For the buildings 79-01, 90-14 and 91-37, the maximum responses fall in the range not greater than the ultimate interstory deflection of 1/300. For the building 79-02, however, the maximum interstory deflection response of 1/95 is produced at the first story level, indicating the evidence that the building approaches the collapse with critical damages for the masonry wall elements.

**Table 6. Maximum Interstory Responses in Deflection Angle during Moderate-scale Design Ground Motions**

| Story Number | Apartment Bldg. Type 79-01 | | Apartment Bldg. Type 79-02 | | Apartment Bldg. Type 90-14 | | Apartment Bldg. Type 91-37 | |
|---|---|---|---|---|---|---|---|---|
| | Long. | Trans. | Long. | Trans. | Long. | Trans. | Long. | Trans. |
| 7 | - | - | - | - | - | - | 1/2,900 | 1/2,400 |
| 6 | - | - | 1/800 | 1/2,200 | 1/2,100 | 1/10,000 | 1/1,400 | 1/1,300 |
| 5 | 1/670 | 1/640 | 1/410 | 1/780 | 1/840 | 1/2,400 | 1/890 | 1/730 |
| 4 | 1/440 | 1/500 | 1/340 | 1/600 | 1/390 | 1/650 | 1/670 | 1/600 |
| 3 | 1/370 | 1/510 | 1/320 | 1/560 | 1/340 | 1/610 | 1/580 | 1/560 |
| 2 | 1/350 | 1/570 | 1/330 | 1/570 | 1/320 | 1/630 | 1/540 | 1/570 |
| 1 | 1/350 | 1/690 | 1/95 | 1/480 | 1/430 | 1/1,100 | 1/550 | 1/970 |

***Response during the large-scaled design ground motions***

Table 7 summarizes the interstory deflection responses that are obtained when subjected to the large-scaled design ground motions. The deflection responses lie in the range greater than the critical response of 1/300, revealing the evidence that the buildings reach the serious damage of collapse. The maximum responses for the buildings are 1/17 for the 79-01 building at the first story level, 1/20 for the 79-02 building at the first story level, 1/70 for the 90-14 building at the second story level, and 1/59 for the 91-37 building at the second story level, respectively. Note that the response deflection angles greater than 1/300 do not essentially indicate the realistic deflection angles, but reveal the evidence that the building reaches the heavily damaged condition leading collapse.

**Table 7. Maximum Interstory Responses in Deflection Angle during Large-scale Design Ground Motions**

| Story Number | Apartment Bldg. Type 79-01 | | Apartment Bldg. Type 79-02 | | Apartment Bldg. Type 90-14 | | Apartment Bldg. Type 91-37 | |
|---|---|---|---|---|---|---|---|---|
| | Long. | Trans. | Long. | Trans. | Long. | Trans. | Long. | Trans. |
| 7 | - | - | - | - | - | - | 1/1,500 | 1/1,300 |
| 6 | - | - | 1/790 | 1/850 | 1/1,700 | 1/2,600 | 1/700 | 1/590 |
| 5 | 1/490 | 1/390 | 1/430 | 1/440 | 1/730 | 1/880 | 1/480 | 1/400 |
| 4 | 1/340 | 1/300 | 1/350 | 1/360 | 1/360 | 1/340 | 1/380 | 1/340 |
| 3 | 1/37 | 1/68 | 1/320 | 1/340 | 1/160 | 1/310 | 1/330 | 1/320 |
| 2 | 1/35 | 1/35 | 1/310 | 1/340 | 1/91 | 1/70 | 1/84 | 1/59 |
| 1 | 1/23 | 1/17 | 1/22 | 1/20 | 1/110 | 1/400 | 1/86 | 1/390 |

The peak intensity of ground motion is evaluated when the responses of the buildings equal the critical responses. The critical responses taken greater than 1/300 in deflection angle for the masonry wall elements reveal shear failure. The peak intensity of ground motion is expressed in terms of peak acceleration. The critical ground motion intensity evaluated for the conditions are summarized in Table 8 for the buildings 79-01, 79-02, 90-14 and 91-37 during the design ground motions SHW-1, –2 and -3. In the case when the figures in Table 8 lie greater than $220 cm/s^2$, the building remains beyond the critical response when subjected to the large-scale design ground motion. In the other cases, buildings are judged to suffer serious damages during the motion. On an average, the intensity of ground motions that can produce the critical responses for the buildings is almost half of that of the large-scaled design ground motions.

**Table 8. Intensity of Design Ground Motion Producing the Critical Response**

| Earthq. Ground Motion | Apartment Bldg. Type 79-01 | | Apartment Bldg. Type 79-02 | | Apartment Bldg. Type 90-14 | | Apartment Bldg. Type 91-37 | |
|---|---|---|---|---|---|---|---|---|
| | Long. | Trans. | Long. | Trans. | Long. | Trans. | Long. | Trans. |
| SHW-1 | 110<br>2F:1/303 | 163<br>3F:1/303 | 111<br>1F:1/302 | 164<br>1F:1/305 | 125<br>2F:1/302 | 191<br>3F:1/303 | 174<br>1f:1/301 | 187<br>3F:1/303 |
| SHW-2 | 115<br>2F:1/303 | 147<br>3F:1/306 | 92<br>1F:1/301 | 143<br>1F:1/303 | 106<br>2F:1/301 | 158<br>3F:1/302 | 149<br>2F:1/303 | 163<br>3F:1/303 |
| SHW-3 | 118<br>2F:1/303 | 218<br>3F:1/304 | 151<br>1F:1/304 | 211<br>1F:1/301 | 178<br>2F:1/302 | 232<br>3F:1/302 | 232<br>1F:1/301 | 236<br>3F:1/302 |

(in $cm/s^2$)

***Response during the large moderate-scaled/moderately large-scale design ground motions***

Herein the study, we introduce the fourth level of design ground motions named hereinafter “the large moderate-scale” or “moderately large-scale” design ground motion. The medium large-scale design ground motion is defined so as the peak ground acceleration to be $160 cm/s^2$, which is the medium figure between the moderate-scaled motion of $100 cm/s^2$ and the large-scale motion of $220 cm/s^2$. The maximum response of buildings evaluated when subjected to the large moderate-scale motions are summarized in Table 9.

**Table 9. Maximum Interstory Responses in Deflection Angle during Large Moderate-scale Design Ground Motions**

| Story Number | Apartment Bldg. Type 79-01 | | Apartment Bldg. Type 79-02 | | Apartment Bldg. Type 90-14 | | Apartment Bldg. Type 91-37 | |
|---|---|---|---|---|---|---|---|---|
| | Long. | Trans. | Long. | Trans. | Long. | Trans. | Long. | Trans. |
| 7 | - | - | - | - | - | - | 1/1,400 | 1/1,100 |
| 6 | - | - | 1/940 | 1/960 | 1/1,700 | 1/2,000 | 1/640 | 1/540 |
| 5 | 1/600 | 1/410 | 1/470 | 1/460 | 1/760 | 1/770 | 1/440 | 1/370 |
| 4 | 1/380 | 1/310 | 1/380 | 1/370 | 1/350 | 1/320 | 1/360 | 1/320 |
| 3 | 1/320 | 1/87 | 1/350 | 1/350 | 1/310 | 1/140 | 1/320 | 1/120 |
| 2 | 1/36 | 1/300 | 1/330 | 1/350 | 1/110 | 1/300 | 1/130 | 1/310 |
| 1 | 1/31 | 1/330 | 1/26 | 1/66 | 1/350 | 1/460 | 1/310 | 1/470 |

## RETROFITTING SCHEME FOR THE EXISTING MASONRY BUIDLINGS

**Retrofitting (Strengthening/Rehabilitation) scheme: Evaluation conditions**

We examine a retrofitting scheme for the buildings that are judged seriously damaged during the prospective seismic action. For our discussion within the study, we examine the apartment building of the Type 79-01 along its longitudinal direction. Analysis and discussion identical to that herein can be extended to that along the transverse direction, and to other buildings.

Fundamental assumptions for the development of retrofitting scheme are as follows:

(1) The buildings are strengthened by placing reinforced concrete wall elements, jacketing the existing masonry walls.

(2) The fundamental properties of these newly placed reinforced concrete walls are:
  (a) The load-deflection curve is represented by a tri-linear curve as illustrated in Figure 3, indicating that the deflection angle of maximum strength is specified as 1/200;
  (b) Strength of wall is calculated by assuming the unit shear strength per to be 2.0N/mm$^2$;
  (c) Weight of wall is estimated with unit weight of 24 kN/m$^3$; and
  (d) Thickness of wall is uniformly prescribed as 20cm to evaluate the wall length necessary for wall length $L_w$ in meter for upgrading structural performance in Tables 8 through 10 in the following.

(3) The amount of strengthening walls upgrading structural performance at lower story levels should not be less than that at higher story levels.

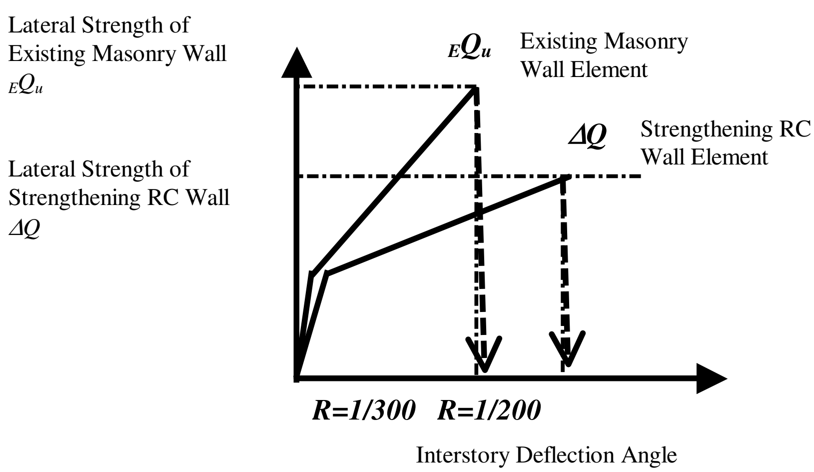


**Figure 3. Load-deflection characteristics for existing masonry walls and strengthening reinforced concrete walls.**

**Retrofitting scheme: Evaluation procedures**
We evaluate the amount of reinforced concrete walls required for the building responses during the design ground motion not to exceed the critical responses of 1/300 in deflection angle, at which the brick/block masonry walls reach the ultimate condition for the fatal collapse of building. The evaluation herein is carried essentially based on try-and-error basis as follows:

(1) We assume the amount of strengthening wall elements at each story level $\Delta Q$, and obtain the responses of the strengthened building.
(2) We carry out dynamic response analysis, and obtain the interstory responses $R$.
(3) Provided that the responses at each story level $R$ do not exceed the critical responses, the proposed retrofitting scheme is one of appropriate and possible schemes.
(4) Provided, however, the obtained responses $R$ are remarkable less than the critical responses, we reexamine the amount of reinforced concrete wall, since it indicates that the amount of strengthening wall would be unnecessary greater than that required.
(5) Provided that the responses $R$ are greater than the critical responses, we review the retrofitting scheme, since the scheme has not been appropriate.

Based on try-and-error basis, we examine and review various schemes of retrofitting proposal, and reach the conclusion that an appropriate and possible retrofitting scheme can be found among the proposed schemes, with which the responses of building fulfill the prescribed condition with less amount of strengthening wall elements.

**Retrofitting scheme: Evaluation results**
***Retrofitting scheme for the small-scale ground motions***
The maximum interstory deflection is 1/1,400, indicating that minor cracks can be generated, while no serious damages that can lead to fatal collapse of building.

***Retrofitting scheme for the moderate-scale ground motions***
For the Type 79-02 building, when subjected to the moderate-scale of SHW-2 design ground motion, we obtain the maximum interstory deflection of 1/95 at the first story. For other buildings, the maximum responses fall in the range less than 1/300, indicating that those buildings do not produce serious damages that can lead to collapse of building. It is concluded that we need to fix up a retrofitting scheme for the building not to produce critical responses for the prescribed ground motion.

Table 10 tabulates the results obtained by dynamic response analysis for the Type 79-01 building, obtained when subjected to the moderate-scale design ground motion of SHW-2.

**Table 10. Seismic Retrofitting Schemes for an Existing Masonry Building: Apartment Bldg. 79-02; Longitudinal Direction; Moderate-Scale Ground Motion of SHW-2**

| Story Level | | 1 | 2 | 3 | 4 | 5 | 6 |
|---|---|---|---|---|---|---|---|
| W (kN) | | 1,430 | 1,360 | 1,360 | 1,360 | 1,360 | 1,430 |
| ΣW (kN) | | 8,310 | 6,890 | 5,520 | 4,160 | 2,800 | 1,430 |
| Scheme-1 | ΔQ (kN) | 210 | 210 | 210 | 210 | 210 | 210 |
| | ΔC | 0.03 | 0.03 | 0.04 | 0.05 | 0.08 | 0.15 |
| | Lw (m) | 0.5 | 0.5 | 0.5 | 0.5 | 0.5 | 0.5 |
| To = 0.326 s | R | 1/305 | 1/350 | 1/360 | 1/420 | 1/590 | 1/1990 |
| Scheme-2 | ΔQ (kN) | 160 | 160 | 160 | 160 | 0 | 0 |
| | ΔC | 0.02 | 0.02 | 0.03 | 0.04 | 0.00 | 0.00 |
| | Lw (m) | 0.4 | 0.4 | 0.4 | 0.4 | 0.0 | 0.0 |
| To = 0.330 s | R | 1/300 | 1/350 | 1/350 | 1/390 | 1/360 | 1/640 |
| Scheme-3 | ΔQ (kN) | 130 | 130 | 50 | 50 | 0 | 0 |
| | ΔC | 0.02 | 0.02 | 0.01 | 0.01 | 0.00 | 0.00 |
| | Lw (m) | 0.3 | 0.3 | 0.1 | 0.1 | 0.0 | 0.0 |
| To = 0.334 s | R | 1/300 | 1/330 | 1/300 | 1/320 | 1/360 | 1/670 |
| Scheme-4 | ΔQ (kN) | 170 | 170 | 170 | 0 | 0 | 0 |
| | ΔC | 0.02 | 0.02 | 0.03 | 0.00 | 0.00 | 0.00 |
| | Lw | 0.4 | 0.4 | 0.4 | 0.0 | 0.0 | 0.0 |
| To = 0.332 s | R | 1/310 | 1/360 | 1/360 | 1/300 | 1/360 | 1/670 |

***Retrofitting scheme for the large-scale ground motions***

When subjected to the large-scale design ground motions, buildings examined herein the study produce the maximum deflection responses greater than the critical angle of 1/300, indicating the evidence that those apartment houses are unsafe suffering serious damages that can yield collapse of building during the prospective ground motions.

For the Type 79-02 building, when subjected to the moderate-scale of SHW-2 design ground motion, we obtain the maximum interstory deflection of 1/95 at the first story. For other buildings, the maximum responses fall in the range less than 1/300, indicating that those buildings do not produce serious damages that can reach collapse of building. It is concluded that we need to fix up a retrofitting scheme for the building not to produce critical responses for the prescribed ground motion.

Table 11 tabulates the results obtained by dynamic response analysis for the buildings Type 79-01, obtained when subjected to the large-scale design ground motion SHW-2 for an exemplary examination and discussion.

**Table 11. Seismic Retrofitting Schemes for an Existing Masonry Building: Apartment Bldg. 79-02; Longitudinal Direction; Large-Scale Ground Motion of SHW-2**

| Story Level | | 1 | 2 | 3 | 4 | 5 | 6 |
|---|---|---|---|---|---|---|---|
| W (kN) | | 1,430 | 1,360 | 1,360 | 1,360 | 1,360 | 1,430 |
| ΣW (kN) | | 8,310 | 6,890 | 5,520 | 4,160 | 2,800 | 1,430 |
| Scheme-1 | ΔQ (kN) | 2,220 | 2,220 | 2,220 | 2,220 | 2,220 | 2,220 |
| | ΔC | 0.27 | 0.32 | 0.40 | 0.53 | 0.79 | 1.55 |
| | Lw (m) | 5.7 | 5.7 | 5.7 | 5.7 | 5.7 | 5.7 |
| To = 0.251 s | R | 1/300 | 1/350 | 1/400 | 1/460 | 1/520 | 1/810 |
| Scheme-2 | ΔQ (kN) | 1,900 | 1,900 | 1,440 | 1,440 | 750 | 750 |
| | ΔC | 0.23 | 0.28 | 0.26 | 0.35 | 0.27 | 0.52 |
| | Lw (m) | 4.9 | 4.9 | 3.7 | 3.7 | 1.9 | 1.9 |
| To = 0.267 s | R | 1/300 | 1/360 | 1/300 | 1/380 | 1/300 | 1/880 |
| Scheme-3 | ΔQ (kN) | 1,900 | 1,900 | 1,440 | 1,440 | 750 | 380 |
| | ΔC | 0.23 | 0.28 | 0.26 | 0.35 | 0.27 | 0.27 |
| | Lw (m) | 4.9 | 1.9 | 3.7 | 3.7 | 1.9 | 1.0 |
| To = 0.267 s | R | 1/310 | 1/370 | 1/310 | 1/390 | 1/300 | 1/390 |

***Retrofitting scheme for the large moderate-scale ground motions***

The results for the buildings Type 79-01 obtained when subjected to the large moderate-scale design ground motion SHW-2 are summarized in Table 12.

**Table 12. Seismic Retrofitting Schemes for an Existing Masonry Building: Apartment Bldg. 79-02; Longitudinal Direction; Large Moderate-Scale Ground Motion of SHW-2**

| Story Level | | 1 | 2 | 3 | 4 | 5 | 6 |
|---|---|---|---|---|---|---|---|
| W (kN) | | 1,430 | 1,360 | 1,360 | 1,360 | 1,360 | 1,430 |
| ΣW (kN) | | 8,310 | 6,890 | 5,520 | 4,160 | 2,800 | 1,430 |
| Scheme-1 | ΔQ (kN) | 1,150 | 1,150 | 1,150 | 1,150 | 1,150 | 11,150 |
| | ΔC | 0.14 | 0.17 | 0.21 | 0.28 | 0.41 | 7.80 |
| | Lw (m) | 1.5 | 1.5 | 1.5 | 1.5 | 1.5 | 1.5 |
| To = 0.279 s | R | 1/300 | 1/350 | 1/410 | 1/560 | 1/1000 | 1/1300 |
| Scheme-2 | ΔQ (kN) | 1,170 | 1,170 | 1,170 | 585 | 585 | 585 |
| | ΔC | 0.14 | 0.17 | 0.21 | 0.14 | 0.21 | 0.41 |
| | Lw (m) | 2.9 | 2.9 | 2.9 | 1.50 | 1.50 | 1.50 |
| To = 0.285s | R | 1/330 | 1/390 | 1/440 | 1/300 | 1/430 | 1/1700 |
| Scheme-3 | ΔQ (kN) | 890 | 890 | 600 | 600 | 300 | 300 |
| | ΔC | 0.11 | 0.13 | 0.11 | 0.14 | 0.11 | 0.21 |
| | Lw (m) | 2.2 | 2.2 | 1.5 | 1.5 | 0.7 | 0.7 |
| To = 0.299 s | R | 1/320 | 1/370 | 1/300 | 1/340 | 1/320 | 1/730 |

**Retrofitting scheme: Discussion on the evaluated results**

Retrofitting schemes proposed within the study are listed in Tables 10 through 12 for the Type 79-02 building for the moderate-scale, large-scale and large moderate-scale design ground motions of SHW-2 motion, respectively, along the longitudinal direction of building. Similar evaluation has been performed for the motions SHW-2 and SHW-3. Figures *Lw* in meter within the rows in Tables give the length of strengthening walls required for seismic retrofitting. The dimension of the building is 18m by 16m.

It will be worth to note that, in general cases, the retrofitting scheme for a building shall not be uniquely determined. We can find that one proposal will more appropriate and applicable than the other when to compare them with each other. In other words, we can make a proposal for a wide variety of retrofitting schemes, and we will take one scheme among them with full consideration of robustness and redundancy of retrofitting scheme, feasibility of planning, execution of work and other conditions.

## CONCLUDING REMARKS

Seismic capacities of existing buildings in Shanghai, China are evaluated based upon the structural documents and structural data [4-7]. Four apartment buildings in the Tongji Village located on the Tongji campus are examined. By use of the screening method proposed for Japanese practice, the seismic capacities of these buildings are evaluated. For these buildings, microtremor measurement has been carried out, through which the fundamental period, mode of oscillation and amplitude factor are obtained.

Using the results obtained in the international cooperative research program between the fist and second authors of this paper with supports provided by colleagues in each party, analytical models for the buildings have been established. Applying the ground motions used in the design practice in Shanghai, China, dynamic response analysis has been carried out. The responses in terms of interstory deflection angle have been obtained and compared with the critical deflection angle established for the masonry shear wall elements. The following remarks have been obtained through the process of seismic evaluation of buildings:

(1) For the buildings examined herein, the seismic performance is not essentially evaluated most inferior at the lowest story levels. The lateral resisting capacity for the brick/block masonry building is closely correlated with the axial force in the seismic resisting elements. Since the elements at the lower story level should carry a large amount of axial force, the lateral capacity of the elements becomes large, while the lateral force produced during the seismic action becomes large as well. The seismic capacity against the required performance within the buildings examined in this study is smallest, in general, at the second or third story level.

(2) The evaluated seismic performance of the buildings examined herein the study for the small-scaled, moderate-scale and large-scale ground motions is that of “repairable after the seismic action”, “with heavy damages with which the building can fall in the critical condition”, and “with serious damages with which the building leads to collapsed,” respectively, while the specified design conditions in Chinese practice are “without damage”, “repairable after the seismic action” and “with heavy damages allowed but no human losses accompanied with.”

(3) Dynamic response analysis indicates evidence that we need a large amount of upgrading structural elements placed for the large-scale design ground motions, and a small amount of structural elements positioned for the moderate-scale design ground motions for the specified criteria that the buildings do not suffer serious damages that can produce a collapse of building. Herein the study, another ground motion level defined as the “medium large-scale” ground motion is introduced, for which the building should not suffer serious damages that can yield loss of human lives.

A scheme for retrofitting of building is proposed for prospective earthquake ground motions to be considered in Shanghai. Herein the study, we examine retrofitting scheme for the medium large-scale design ground motion as well as the small-scale, moderate-scale and large-scale design ground motions. Note that a scheme proposed herein the study is one and possible preferable retrofitting scheme, and not the one and only one scheme for the retrofitting of the building. One can propose the other plans of retrofitting for upgrading the structural performance of building against the prospective seismic action. The scheme introduced herein the study can be utilized as one of way of thinking for the retrofitting processes, revealing the processes of structural upgrading for the buildings that are judged unsafe for the prospective seismic action.

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## REFERENCES

[1] Standards and Commentary of Seismic Screening Method for Existing Reinforced Concrete Buildings: Japan Building Disaster Prevention Association, 140 pages, December 1990. (in Japanese)

[2] Structural Design Standards for Masonry Structures GBJ 3-88: National Standards of the Peoples of Republic of China, Department of Construction, Chinese Government, September 1989. (in Chinese)

[3] Tetsuo Kubo, Matsuo Aono, Toshihiro Achiwa and Maged A. H. Etawa: Evaluation of Seismic Performance by The Seismic Screening Method and Its Verification by Dynamic Response Analysis on A Standard School Building in The Philippines, Proceedings of The 11th Japan Earthquake Engineering Symposium, Paper # 377, CD-ROM, 6 pages, November, 2002.

[4] Tetsuo Kubo: Evaluation of Seismic Resisting Capacities of Existing Buildings –Brick-Wall Buildings in Shanghai, China-, Proc. The 3rd Multi-lateral Workshop on *EqTAP*, Manila, Philippines, November 28-30, 2000, EdM Report No. 11, EdM, RIKEN, pp. 219-225, March 2001,

[5] Tetsuo Kubo: Evaluation of Seismic Resisting Capacities of Existing Buildings – Correlation of Seismic Performance Index with Dynamic Response during an Intense Ground Motion for the Brick-Masonry Buildings in Shanghai, China-, Proc. The 4th Multi-lateral Workshop on *EqTAP*, Kamakura, Japan, December 3-4, 2001, EdM Report No. 14, EdM, NIED, 6 pages (CD-ROM), January 2002.

[6] Tetsuo Kubo: A Retrofitting Scheme for Masonry Buildings Strengthened by Placing Reinforced Concrete Walls –Development of Retrofitting Technology Using R/C Walls for Brick/Block Masonry Buildings in Shanghai, China-, Proc. The 5th Multi-lateral Workshop on *EqTAP*, Bangkok, Thailand, December 2-3, 2002, EdM Report No. 16, EdM, NIED, 6 pages (CD-ROM), March 2003.

[7] Tetsuo Kubo: Seismic Performance Evaluation And Rehabilitation of Existing Masonry Apartment Buildings in Shanghai, China –A case study on apartment houses in the Tongji village, Shanghai, China-, Proc. The 6th Multi-lateral Workshop on *EqTAP*, Kashikojima, Japan, December 2-3, 2003, 12 pages. (in printing)