16th World Conference on Earthquake Engineering, 16WCEE 2017

Santiago Chile, January 9th to 13th 2017

Paper N° 2977

Registration Code: S-617792

PRELIMINARY SEISMIC ASSESSMENT OF LOW-RISE REINFORCED CONCRETE BUILDINGS IN TAIWAN

T.C. Chiou[(1)], S.J. Hwang[(2)], L.L. Chung[(3)], Y.S. Tu[(4)], W.C. Shen[(5)], P.W. Weng[(6)]

[(1)] *Associate Researcher, National Center for Research on Earthquake Engineering, Taipei, Taiwan, tcchiou@ncree.narl.org.tw*
[(2)] *Professor, National Taiwan University, Department of Civil Engineering, Taipei, Taiwan, sjhwang@ntu.edu.tw*
[(3)] *Division Head, National Center for Research on Earthquake Engineering, Taipei, Taiwan, chung@ncree.narl.org.tw*
[(4)] *Associate Professor, De Lin Institute of Technology, Department of Construction Engineering, New Taipei, Taiwan, tuysmail@gmail.com*
[(5)] *Assistant Researcher, National Center for Research on Earthquake Engineering, Taipei, Taiwan, wcshen@ncree.narl.org.tw*
[(6)] *Assistant Researcher, National Center for Research on Earthquake Engineering, Taipei, Taiwan, pwweng@ncree.narl.org.tw*

Abstract

An effective preliminary seismic assessment can provide a pre-screening of seismic capacity for a huge number of low-rise reinforced concrete (RC) buildings (*e.g.*, school buildings and street houses) in Taiwan. The buildings that are found to have insufficient seismic capacity must then be subjected to detailed evaluation to conform to the required seismic capacity. The National Center for Research on Earthquake Engineering has developed a preliminary seismic assessment method using a detailed evaluation of a database of school buildings in Taiwan. From statistical regression results, the performance ground acceleration of each school building is found to be related to the total floor area, column section area, and wall section area of the school building. In other words, the performance ground acceleration of a low-rise RC building can be rapidly evaluated using the ratio of column section area to floor area of the building. The preliminary seismic assessment method is verified using another database of earthquake reconnaissance. The buildings with moderate or severe seismic damage were found to have concerns regarding their seismic capacity. Meanwhile, for buildings with slight seismic damage, the evaluation reveals only minor concerns over their seismic capacity. Consequently, the preliminary assessment method is found to be conservative and consistent with the trend of damage levels of existing low-rise RC buildings.

Keywords: preliminary seismic assessment; low-rise building; street house; reinforced concrete

1. Introduction

Seismic reconnaissance shows that, among the existing reinforced concrete (RC) buildings in Taiwan, street houses and school buildings suffer the most severe damages (Figures 1 and 2). For example, in the 921 Chi-Chi earthquake, nearly half of the street houses in Central Taiwan were either severely damaged or had collapsed. To avoid such large-scale damages in future, it is essential to conduct a comprehensive health check on existing RC buildings in Taiwan and to retrofit buildings with inadequate seismic capacity. To efficiently assess the seismic capacity of a large number of street houses, the National Center for Research on Earthquake Engineering (NCREE) utilized a database of school buildings and developed a preliminary seismic assessment method for low-rise RC structures.

Street houses in Taiwan typically have the ground floor as shop fronts or living rooms, featuring many doors and windows (Figure 3), while the second and upper stories are divided into smaller spaces, such as bedrooms, study rooms, or showers, often with many partition walls (Figure 4). A comparison of Figures 3 and 4 indicates that the ground floor has fewer walls but bears higher loads and hence is more susceptible to damage. Figure 3 also shows that most typical street houses are terraced residential buildings (called townhouses) with multiple houses connected to one another by 20-cm- or 30-cm-thick brick walls in a direction perpendicular to the corridor. Therefore, the higher number of walls in the direction perpendicular to the corridor than along it renders the houses weak under seismic loading in that direction. Street houses commonly collapse in the ground floor along the corridor, as can be seen in Figure 1.

In general, the higher the number of stories of a building, the larger the load acting on it and the greater the seismic capacity required. The seismic capacity of a building is derived from its columns and walls; larger areas of columns and walls impart greater seismic capacity. According to data on the seismic damage of street houses and school buildings (Figures 1 and 2), the ground floors of such low-rise RC buildings often experience severe damage or collapse, while the higher floors suffer only minor damages. Therefore, the lateral strength of members in the ground level primarily controls the seismic capacity of these structures. In other words, a structure's seismic capacity can be estimated from the strength of the members in its ground floor. Given this, the principle of preliminary evaluation of the seismic capacity of low-rise RC street houses is to estimate the seismic capacity based on the number of columns and walls on the ground floor of such buildings. NCREE has used a database obtained from detailed seismic evaluations conducted on school buildings to derive a relationship between seismic capacity and column and wall areas on the ground floors of low-rise RC buildings. On the basis of this relationship, a quick assessment method for the seismic capacity of low-rise RC buildings has been developed.

This study applies the Taiwan Earthquake Assessment for Structures by Pushover Analysis (TEASPA) for detailed assessment of selected school buildings. This preliminary assessment method applies only to RC or confined masonry buildings with less than six levels (inclusive) and having rigid floor panels. In this paper, the underlying principle of the proposed method is first introduced, then validated using a seismic database of RC buildings from past reconnaissance reports of NCREE. The proposed method is demonstrated to have a screening function. Finally, the proposed method is applied to the NCREE database of typical street-house buildings in Taiwan and the percentage required for a detailed evaluation is estimated for low-rise RC buildings.

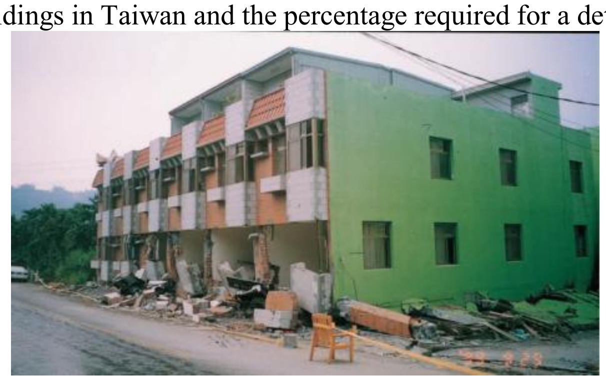


Fig. 1 – Collapse of the ground floor of a typical townhouse-type street house

Fig. 2 – Collapse of the ground floor of a school building

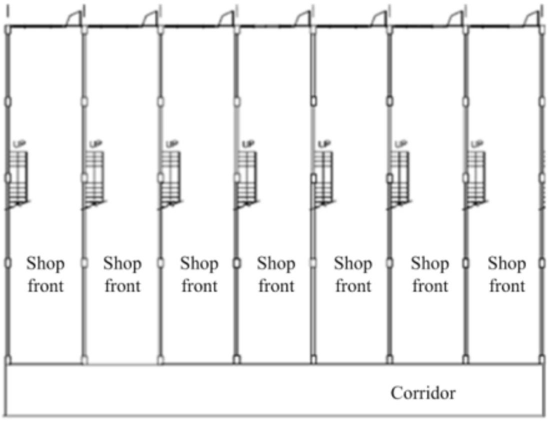


Fig. 3 – Ground floor plan of a typical townhouse building

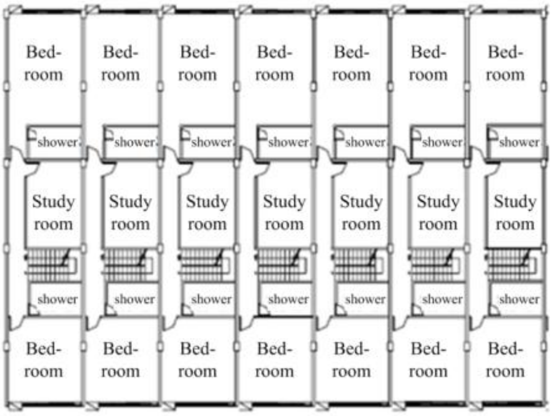


Fig. 4 – Second floor plan of a typical townhouse building

2. Relationship between column and wall areas and seismic capacity

NCREE has long cooperated with the Ministry of Education, Taiwan, to promote a project for seismic upgrading of school buildings. NCREE also set up an information website for seismic assessment and retrofitting of school buildings, which contains detailed information including the total floor area, column area, wall area, and peak ground acceleration of the performance point for each assessed building. From the database up to April 5, 2011, Song *et al*. [1] selected data of 3,504 buildings assessed by TEASPA [2]. Among these, 2,922 were typical school buildings. After a series of screening processes, 2,774 school buildings were finally adopted for a statistical analysis. Structures assessed include bare frames, brick infilled frames, RC infilled frames, and buildings with both brick and RC infilled frames. In this section, a statistical analysis of the results of the detailed seismic assessment of 1,187 bare frame structures is conducted in order to derive the relationship between column area and seismic capacity. The strength of infill walls is converted to equivalent area of the column members. The seismic capacity of typical street houses is then estimated using the ratio of equivalent column area to total floor area.

2.1 Seismic capacity of bare frame structures estimated by the column-to-floor area ratio

To perform a statistical analysis, Song *et al*. [1] chose 1,187 school buildings with bare frames in the longitudinal direction from the NCREE school building seismic database. The database contains information on the total floor area, column area, and performance ground acceleration A_p derived from pushover analysis. A relationship between the column-to-floor area ratio and the associated seismic performance, A_p, can be derived through regression analysis, as shown below:

$$\begin{cases} A_p = \dfrac{100CFR - 0.4 + 0.05N_f}{1.62 - 0.24N_f}, CFR \geq (0.4 - 0.05N_f)\% \\ A_p = 0, CFR < (0.4 - 0.05N_f)\% \end{cases}, \tag{1}$$

where N_f is the number of levels in the building (for a five- or six-level building, N_f takes a value of 4 to produce conservative assessment results) Here, CFR is the column-to-floor area ratio, calculated as follows:

$$CFR = \frac{\sum A_C}{\sum A_f}, \tag{2}$$

where $\sum A_C$ is the total column area on the ground floor and $\sum A_f$ is the total floor area on the second and higher floor. If there is an additional penthouse on the top floor of the RC structure, then the entire additional floor area is accounted for in the calculation of the total floor area; if this penthouse is made of lightweight material such as steel or wood, then half of its area is included in the calculation of the total floor area.

The base shear for each bare frame can be obtained from the pushover analysis. The total column areas of the bottom floors of the buildings can also be taken from the database. Therefore, the relation between the base shear and the column area of the ground floor is as follows:

$$V = \tau_c \sum A_c \ , \tag{3}$$

where V is the base shear of the building, τ_c is the average ultimate shear strength of the RC columns (kgf/cm^2), and ΣA_c is the sum of cross-sectional areas of all RC columns on the ground floor. Song *et al*. [1] summarized the assessment results for the bare frames of school buildings and plotted the total column area ΣA_c against the base shear V_p at the performance point in a scatter plot (Figure 5). Through linear regression, the average ultimate shear strength of the columns is calculated to be 7.95 kgf/cm^2, with a correlation coefficient, r, of 0.79. This outcome is in good agreement with the results of other past studies, as shown in Table 1.

2.2 Equivalent column area of RC walls

The dimensions of a typical RC wall confined on four sides with RC frame are determined through statistical analysis of the database of typical street house buildings in Taiwan [3]. In typical street house buildings, the lengths of RC walls confined on four sides in the direction perpendicular to the corridor vary in the range 253–417 cm. The average height of the ground floor panel is approximately 350 cm. After deducting 50 cm for the depth of beam below the panel, the net height of the wall is approximately 300 cm. Therefore, in this study, it is assumed that the length of an RC wall confined on four sides is 3 m. Given that the confined RC walls have RC columns on both sides, there is sufficient flexural strength, and, as a result, these walls are often controlled by shear failure. This study employs the TEASPA formula [2] to calculate the ultimate shear strength of RC walls in order to evaluate the in-plane lateral strength of a typical RC wall confined on four sides. The average concrete strength is assumed to be $f_c' = 175$ kgf/cm^2. As school buildings are low-rise buildings, the axial force resulting from the gravity of the walls can be conservatively taken as $N = 0.1 f_c' A_w$. The ultimate shear strength of the RC walls according to TEASPA [2] is:

$$V_n = (K_h + K_v - 1)\zeta f_c' A_{str} \cos\theta \ , \tag{4}$$

where ζ is the softening coefficient of concrete, which is 0.52 for $f_c' = 175$ kgf/cm^2. For a conservative evaluation of the shear strength of RC walls, the strength contribution of both horizontal and vertical reinforcements is ignored (*i.e.*, $K_h = K_v = 1$). The angle of the diagonal strut, θ, for members with a height-to-length ratio less than 1 is conservatively taken as 45°. The area of the diagonal strut, A_{str} is calculated as:

$$A_{str} = (0.25 + 0.85 \frac{N}{f_c' A_w}) \ell_w \times t_w \ , \tag{5}$$

where A_w is the cross-sectional area of the wall ($\ell_w \times t_w$), ℓ_w is the wall length, and t_w is the wall thickness. Substituting Equation (5) into (4) and simplifying gives the shear strength per unit area of a typical RC wall:

$$\frac{V_n}{\ell_w t_w} = (1 + 1 - 1) \times 0.52 \times 175 \times (0.25 + 0.85 \times 0.1) \times \cos 45° = 21.6 \text{ kgf/cm}^2. \tag{6}$$

Hence, a shear strength value of 21 kgf/cm^2 is adopted to be conservative.

Table 1 lists existing results for the lateral strength of RC walls confined on three sides. Among them, Hsu *et al*. [4], who developed a preliminary evaluation method for school buildings, suggested a value of 24 kgf/cm^2. Su [5] analyzed typical school building models and found that when an RC wall confined on three sides suffers shear failure, the average shear force per unit area is 18.7 kgf/cm^2, which is close to a corresponding value of 20 kgf/cm^2 suggested by JBDPA [6] and Kuo [7]. However, when RC walls are considered to be governed by flexural failure, their lateral strength is 12.9 kgf/cm^2 as calculated by Su [5]. Therefore, Su recommended using

12 kgf/cm^2 in their preliminary evaluation method [5]. In this study, it is considered that RC walls of street house buildings confined on three sides resemble those of school buildings. To be conservative, a value of 12 kgf/cm^2 is adopted for the lateral strengths of RC walls confined on three sides.

2.3 Equivalent column area of brick walls

From Sheu *et al.* [8], TEASPA [2] proposes that for a brick wall confined on four sides, if $h_b/\ell_b \leq \tan\theta_c$, then its lateral strength can be calculated using:

$$V_{bw4} = \tau_f(\ell_b \times t_b) + 0.45 f_{mt}(h_b \times t_b) \tag{7}$$

while if $h_b/\ell_b > \tan\theta_c$, then it can be calculated using:

$$V_{bw4} = \tau_f(\ell_b \times t_b) + 0.45 f_{mt}(\ell_b \tan\theta_c \times t_b) + 0.45\left(\frac{f_{mt}+f_{bt}}{2}\right)(h'_b - \ell_b \tan\theta_c) \times t_b \, , \tag{8}$$

where h_b is the height of the brick infill walls (mm), ℓ_b is the brick wall length (mm), t_b is the brick wall thickness (mm), and $h'_b = \min(h_b, \ell_b)$. The sliding shear strength of bedding joints is calculated as:

$$\tau_f = 0.0258(f_{mc})^{0.885} + (0.654 + 0.00514 f_{mc})\frac{N}{\ell_b \times t_b} \, , \tag{9}$$

where N is the axial force resulting from gravity (N) and f_{mc} is the compressive strength of mortar. Meanwhile, f_{mt} is the split strength of mortar and can be calculated as follows:

$$f_{mt} = 0.232\left(f_{mc}\right)^{0.338} . \tag{10}$$

Next, f_{bt} is the split strength of the solid-clay brick wall and is given as:

$$f_{bt} = 0.22 f_{bc} \, , \tag{11}$$

where f_{bc} is the compressive strength of the solid-clay brick wall.

Using the above formula for calculating the strength of brick walls confined on four sides, Chiou and Hwang [9] experimentally verified that the analytical result and the experimental outcome of the maximum lateral strength agree with each other. The splitting strength of vertical cross-joints of mortar and solid-clay brick walls is appropriately considered.

In this study, a single brick wall is simulated as a diagonal strut. The strut angle is determined by the center of the compressive force at both ends. Strut strength is governed by the main failure path, which, according to the crack development graph, should include the sliding cracks of bedding joints and the vertical splitting cracks of the cross-joints and solid-clay bricks. The main failure path of brick walls confined on three sides is summarized in Figure 6. The aforementioned three types of cracks are accounted for in the calculation of the strength of brick walls confined on three sides, according to the following equation:

$$\begin{aligned} V_{bw3,proposed} = \tau_f\left(\frac{2}{3}\ell_b \times t_b\right) + 0.225 f_{mt}\left(\frac{2}{3}\ell_b \tan\theta_c \times t_b\right) + \\ 0.225\left(\frac{f_{mt}+f_{bt}}{2}\right)\left(h'_b - \frac{2}{3}\ell_b \tan\theta_c\right) \times t_b \end{aligned} \, , \tag{12}$$

where h'_b represents the effective height of the force transfer path, determined by the intersection of the diagonal strut and the column. If there are two wall piers on both sides of the column (Figure 6(a)), then h'_b is given as:

$$h_b' = \min(\frac{h_b}{\frac{2}{3}\ell_b \times 2 + \ell_c} \times \frac{2}{3}\ell_b, \ell_b) . \tag{13}$$

If there is a wall pier on only one side of the column (Figure 6(b)), then h_b' is given as:

$$h_b' = \min(\frac{h_b}{\frac{2}{3}\ell_b + \ell_c - \frac{a_c}{3}} \times \frac{2}{3}\ell_b, \ell_b) . \tag{14}$$

Here, ℓ_c is the column depth and a_c is the depth of the compression zone in the column. It is calculated using the formula for the depth of the compression zone in elastic columns under bending:

$$a_c = (0.25 + 0.85\frac{N}{A_g f_c'})\ell_c . \tag{15}$$

In Equation (12), the first term represents the shear strength of sliding of bed-joints, with an associated crack length of $2/3\ell_b$; the second term represents the splitting strength of vertical joints in inclined cracks, with a crack length of $2/3\ell_b \tan\theta_c$; the third term represents the splitting strength of vertical joints and solid-clay bricks in vertical cracks, as shown in the middle of Figure 6, and its crack length is $h_b' - 2/3\ell_b \tan\theta_c$. The splitting strength coefficient in Equation (12) is taken as 0.225. As the brick walls confined on three sides are free on one side, the vertical splitting strength of cross-joints and solid-clay bricks should be less than the splitting strength of brick walls confined on all four sides. Therefore, a value of 0.225 is adopted in this study.

Equation (12) is derived from observations of the main failure path characteristics in double-curvature experiments conducted on brick walls confined on three sides. When infill walls confined on three sides undergo double-curvature deformation, the top of the wall is compressed by the overlying beam and a compression zone is formed, which facilitates a diagonal force transfer mechanism at the bottom. Therefore, Equation (12) can be applied to both infill and confined masonry walls. However, in the case of infill walls, because the self-weight of the frame structure is already in equilibrium prior to the construction of the wall, the need to withstand the weight of the structure above is eliminated. When applying Equation (12) to infill walls, the axial force should be assumed to be zero in order to obtain more conservative assessment results.

Based on the results of statistical analysis of the database of typical street house buildings in Taiwan [3], the average height of ground floor panel is approximately 350 cm. Deducting 50 cm for the beam depth below the panel, the net height is approximately 300 cm. The average length of brick walls confined on four sides is approximately 338 cm, with a range of 225–451 cm within one standard deviation. The average length of brick walls confined on three sides is approximately 251 cm, with a range of 120–382 cm within one standard deviation. The walls of typical street house buildings are assumed to be brick infill walls and the axial force as zero in order to determine the distribution of the strength of the brick walls. The walls are constructed using English bond (one stretcher by one header alternatively). The tangent value of the critical failure angle ($\tan\theta_c$) is 0.6. In addition, the minimum compressive strength of mortar (f_{mc}) is assumed to be 100 kgf/cm^2 and that of brick (f_{bc}) to be 150 kgf/cm^2. The splitting strength of mortar, f_{mt}, calculated using Equation (10), is 5.12 kgf/cm^2, and the splitting strength of solid bricks, f_{bt}, calculated using Equation (11), is 33 kgf/cm^2. The statistical distribution of the length of typical street house walls confined on four sides (225–451 cm) is substituted in Equations (7) and (8) to calculate the strength. Similarly, the distribution for walls confined on three sides (120–382 cm) is substituted into Equation (12). The relationship between lateral strength per unit area and wall length is plotted as shown in Figure 7. The plot shows that the statistical average length of walls confined on four sides is 450 cm with one standard deviation, and the corresponding lateral strength per unit area is approximately 4.0 kgf/cm^2. For walls confined on three sides, the statistical average length is 380 cm with one standard deviation, and the corresponding lateral strength per unit area is approximately 3.2 kgf/cm^2.

Table 1 shows that a lateral strength of 4.0 kgf/cm^2 recommended in this study for brick walls confined on four sides is similar to results in the literature. For brick walls confined on three sides, the value of 3.2 kgf/cm^2

recommended in this study is higher than those from other studies, but is still conservative when compared with the experimental results.

3. Preliminary seismic assessment of low-rise reinforced-concrete buildings

The weak axis of columns in typical street houses is often in the direction along the corridor. The difference in the lateral strength of the column in different directions is negligible when considering the dimensions of walls. Therefore, in a rapid assessment, the calculation of the column area does not distinguish between different directions. However, the calculation of wall area does require a distinction between directions parallel and perpendicular to the street. Using the in-plane lateral strength recommended in this study for various types of walls (as shown in Table 1) and a lateral strength per unit column area of 7.95 kgf/cm^2, the equivalent conversion coefficients for different types of walls can be derived. First, for brick walls confined on three sides, conversion into the equivalent column area can be performed with:

$$\tau_c \sum A_c = \tau_{bw3} \sum A_{bw3} , \tag{16}$$

where τ_{bw3} is the average ultimate shear strength of brick walls confined on three sides and $\sum A_{bw3}$ is the sum of the cross-sectional areas of brick walls confined on three sides on the ground floor. Substituting $\tau_c = 7.95$ and $\tau_{bw3} = 3.2$ into Equation (16) gives the conversion coefficient for converting the strength of brick walls confined on three sides into equivalent column area:

$$\sum A_{c,eq} = \frac{3.2}{7.95} \sum A_{bw3} = 0.403 \sum A_{bw3} , \tag{17}$$

where $\sum A_{c,eq}$ represents the equivalent column area. Similarly, the conversion coefficient for the strength of brick walls confined on four sides can be derived on the basis of Table 1, like so:

$$\sum A_{c,eq} = \frac{4.0}{7.95} \sum A_{bw4} = 0.503 \sum A_{bw4} , \tag{18}$$

where $\sum A_{bw4}$ is the sum of the cross-sectional area of brick walls confined on four sides on the ground floor.

Table 1 – Recommended lateral strengths of various members

Unit: kgf/cm^2		Japan [6]	Kuo [7]	Hsu [4]	Su [5]	Sheu [8]	This study
Column	Short column	15	14.1	15	-	7.92	7.95
	Window column	10	9.9				
	Long column	7	5.4				
RC wall	Confined on four sides	30	28.6	-		10.37	21
	Confined on three sides	20	20	24	12		12
	Confined on two sides	10	11.2	-	-		-
Brick wall	Confined on four sides	-	3.9	3	3	3.37	4.0
	Confined on three sides	-	1.6	1.5	2		3.2

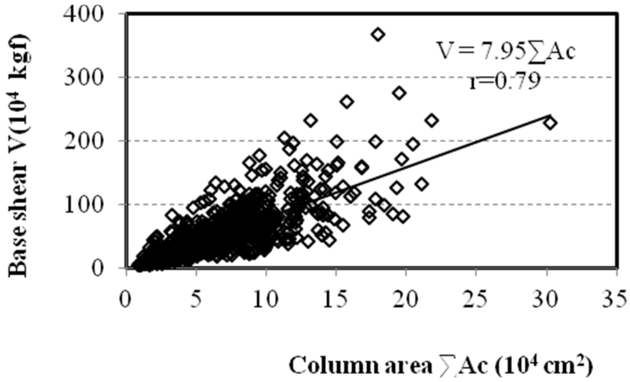
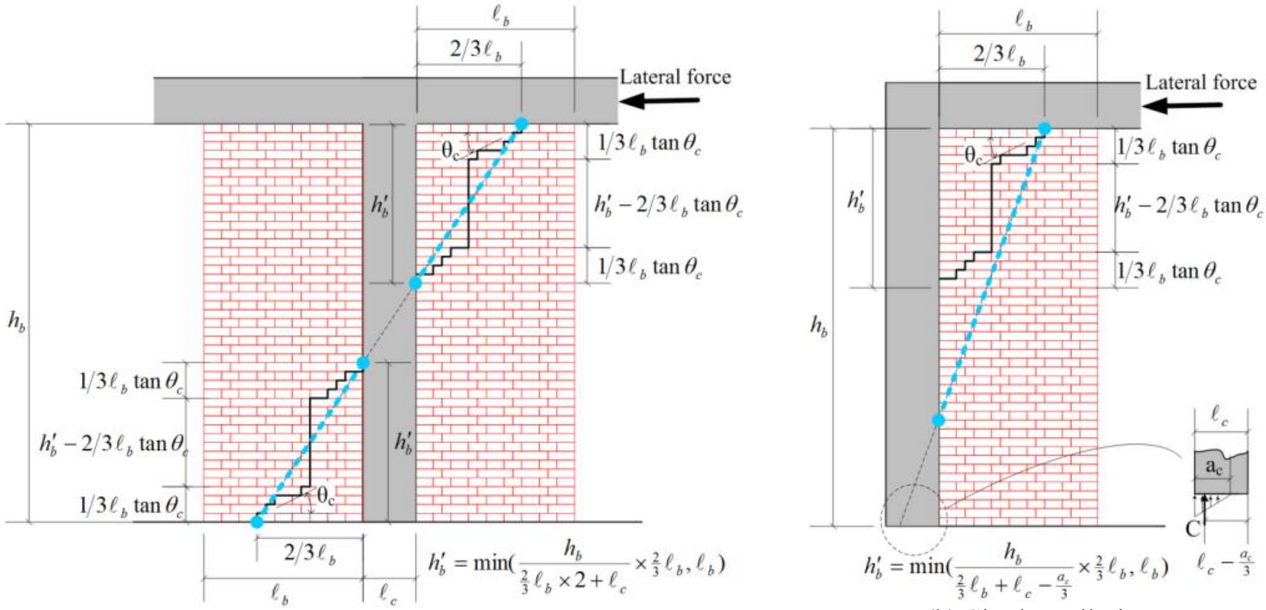


Fig. 5 – Relationship between base shear and column area in longitudinal bare frames of school buildings



(a) Double wall piers

(b) Single wall pier

Fig. 6 – Main failure paths in brick wingwalls

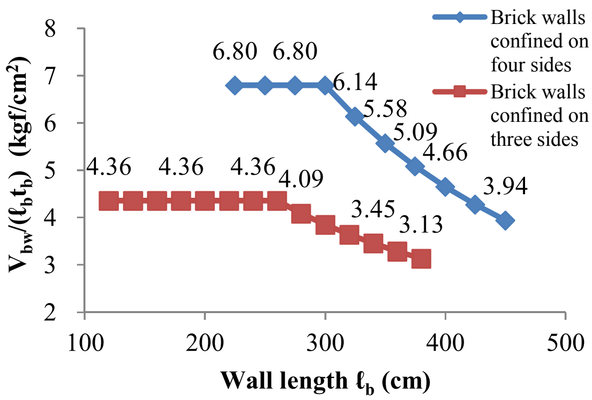


Fig. 7 – Strength versus wall length for typical brick infill walls of street house buildings

The conversion coefficient for converting the strength of RC walls confined on three sides into the equivalent column area is given by:

$$\sum A_{c,eq} = \frac{12}{7.95}\sum A_{rcw3} = 1.509\sum A_{rcw3} \,, \tag{19}$$

where $\sum A_{rcw3}$ is the sum of the cross-sectional areas of brick walls confined on three sides on the ground floor. The conversion coefficient for RC walls confined on four sides is given by:

$$\sum A_{c,eq} = \frac{21}{7.95}\sum A_{rcw4} = 2.642\sum A_{rcw4} \,, \tag{20}$$

where $\sum A_{rcw4}$ is the sum of the cross-sectional areas of RC walls confined on four sides on the ground floor.

Brick walls and RC walls that are confined on three and four sides are included in the ratio of equivalent column to floor areas, as follows:

$$\begin{aligned} CFR_{eq} &= \frac{\sum A_C}{\sum A_f} + \beta\left(0.403\frac{\sum A_{bw3}}{\sum A_f} + 0.503\frac{\sum A_{bw4}}{\sum A_f} + 1.509\frac{\sum A_{rcw3}}{\sum A_f} + 2.642\frac{\sum A_{rcw4}}{\sum A_f}\right) \\ &= \frac{\sum A_C + 0.36\sum A_{bw4} + 0.45\sum A_{bw4} + 1.36\sum A_{rcw3} + 2.38\sum A_{rcw4}}{\sum A_f} \end{aligned}, \tag{21}$$

where CFR_{eq} is the equivalent column-to-floor area ratio and β is the reduction factor for the ultimate strength. Since the ultimate strengths of the various members of the structure will not occur simultaneously, a value of 0.9 is adopted according to Su's suggestion [5]. By substituting the equivalent column-to-floor area ratio for the directions parallel and perpendicular to the street, CFR_{eqx} and CFR_{eqy}, in Equation (1), the performance ground acceleration in the two directions, A_{px} and A_{py}, can be obtained. The seismic capacity of a building is the lesser of the two; that is:

$$A_p = \min(A_{px}, A_{py}) \,. \tag{22}$$

Since the proposed method focuses on low-rise RC buildings, the seismic demand A_T can be determined on the basis of the seismic design code. The short-term horizontal acceleration coefficient, S_S^D, of the building (based on its location), the near-fault correction coefficient, N_A, and the magnification coefficient, F_a, for industrial sites can be looked up in the seismic design code. The short-term design horizontal acceleration coefficient, S_{DS}, can then be obtained. The seismic demand A_T is calculated by:

$$A_T = 0.4S_{DS} \,. \tag{23}$$

The ratio of the seismic capacity to the seismic requirement is given as:

$$E = \frac{A_P}{A_T} \,. \tag{24}$$

This study includes modification factors for the construction time (q_1), eccentricity effect (q_2), weak story effect (q_3), and short column effect (q_4). According to the evolution of the seismic design code, the construction years are divided into four periods. The modification factor for the construction year (q_1) is 0.9 for buildings constructed before 1974, 0.95 for those between 1975 and 1983, 1.0 for those between 1983 and 1999, and 1.05 for those constructed in 2000 or thereafter. A reconnaissance of the 0206 Meinong earthquake, which occurred in 2016 in Taiwan, shows that buildings at the corners of intersections exhibit apparent eccentric rotation, as shown in Figure 8. Therefore, the modification factor for the eccentricity effect (q_2) is chosen to be 0.9 for buildings with corridors on both sides and 1.0 for those with a corridor on only one side. Since removal of some walls on the bottom floor of a building may cause it to collapse, the modification factor for the effect of a weak story (q_3) is taken into consideration; q_3 is 0.9 if any wall in the building frame is removed and 1.0 otherwise. When the net height of a column (H_n) is less than or equal to twice the depth in the loaded lateral direction (2D) (*i.e.*, height-to-depth ratio $H_n/D \le 2$), the column is defined as a short column. The 0206 Meinong earthquake reconnaissance revealed that short window columns often tend to experience shear failure, as shown in Figure 9. Therefore, the modification factor for short columns (q_4) should not be less than 0.5:

$$q_4 = (1 - \text{ratio of short column area to total column area}) \ge 0.5. \quad (25)$$

Finally, the fundamental seismic capacity modification factor Q can be calculated as:

$$Q = q_1 \times q_2 \times q_3 \times q_4\,. \quad (26)$$

The preliminary evaluation index for building seismic capacity I_s is:

$$I_S = E \times Q\,. \quad (27)$$

If $I_s < 1.0$, there is a concern regarding the building's seismic capacity.

Fig. 8 – Buildings at the corner of an intersection collapsed in the 0206 Meinong earthquake owing to eccentric rotation

Fig. 9 – Shear failure of short columns in high windows

4. Validation of the proposed preliminary assessment method

In this study, basic information such as dimensions and damages of 59 low-rise RC buildings, including street houses and school buildings, is collected from NCREE's reconnaissance report, which included the 921 Chi-Chi earthquake, 0304 Jiaxian earthquake, and 0602 Nantou earthquake. With information on the number of stories, total floor areas, column dimensions and quantities, and number of brick walls in the direction along the corridor (X direction) and perpendicular to the corridor (Y direction), the seismic capacities of various buildings can be estimated using the proposed preliminary assessment equations. Assessment results are compared with actual seismic records obtained for these buildings in order to validate the screening of the proposed method.

The preliminary assessment method is applied to the 59 buildings to derive their equivalent column-to-floor ratios, CFR_{eq}, and the seismic performance, A_p. If the value of the seismic performance, A_p, divided by the peak ground acceleration recorded in an earthquake, A_{rec}, is greater than 1, then the building is considered to have no major safety concerns during the earthquake, as the damage may be minor. If this value is less than 1, then the building may be moderately or severely damaged by the earthquake, which may even result in building collapse. Figure 10 presents the assessment results versus the actual level of damage buildings experienced in the earthquake. It shows preliminary assessment results are less than 1 for the majority of buildings with moderate, severe, or building-collapse damage, indicating that the assessment results are in agreement with the actual level of seismic damage. The preliminary assessment results are greater than 1 for most buildings with minor or slight damage. However, some of the buildings with minor or slight damage have a result of less than 1, demonstrating that the results of the preliminary assessment are conservative.

If the seismic performance obtained from the preliminary assessment, A_p, is taken as the seismic capacity and $A_T = 0.4S_{DS}$ from the seismic design code is taken as the seismic demand, then the ratio between the two is the seismic capacity–to–seismic demand ratio. This ratio is plotted against the actual seismic damage level in Figure 11. The average seismic capacity–to–seismic demand ratio is 1.4 for minor damage, 1.06 for slight damage, 0.89 for moderate damage, 0.68 for severe damage, and 0.34 for building-collapse cases. These results agree with the trend that a lower seismic capacity-to-demand ratio indicates greater damage level. The fact that the average of A_p/A_T is less than 1 for collapse as well as severe and moderate damage levels confirms that the proposed preliminary assessment method has an effective screening function.

To understand the actual effect of the preliminary assessment method, this study utilized the NCREE's database of typical street house buildings in Taiwan [3] to conduct trials. This database contains structural data on 145 street house buildings with information such as the location, total floor area, number of stories, column area on the ground floor, and wall areas in the parallel and perpendicular directions to the corridor. Moreover, it contains information on street house buildings in Taipei City, New Taipei City, Taichung City, Changhua County, Nantou County, Tainan City, and Kaohsiung City. It is a comprehensive database covering all existing representative types of street house buildings in Taiwan, including terraced townhouses, condominium-type street houses, and free-standing townhouses.

The 145 buildings are assessed using the proposed method. The results, namely the performance ground acceleration, A_p, represent the seismic capacity. Depending on the location of the building, the seismic design ground acceleration, A_T, is looked up in the design code and taken as the seismic demand. The seismic capacity-to-demand ratio gives the seismic index *Is*. An *Is* of less than 1 indicates that the building's seismic capacity is of concern, while that greater than 1 indicates no concern. Figure 12 shows the preliminary assessment results for the 145 street house buildings. Among them, 56 buildings have *Is* < 1 (38.6% of the total samples; Figure 13). Therefore, by using this proposed preliminary assessment method to assess all the street houses in Taiwan, approximately 38% of them are found to require further detailed assessment of their seismic capacity.

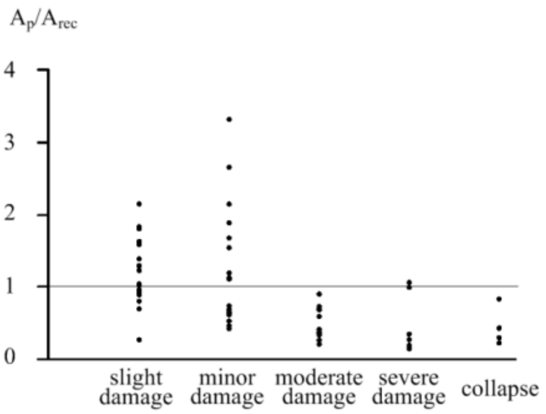


Fig. 10 – Ratio of preliminary assessment results to recorded acceleration versus actual seismic damage level

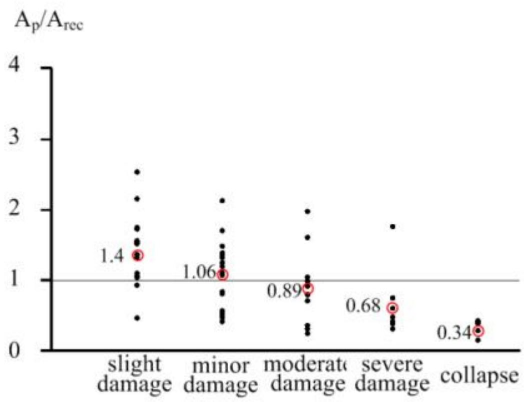


Fig. 11 – Ratio of preliminary assessment results to design acceleration versus actual seismic damage level

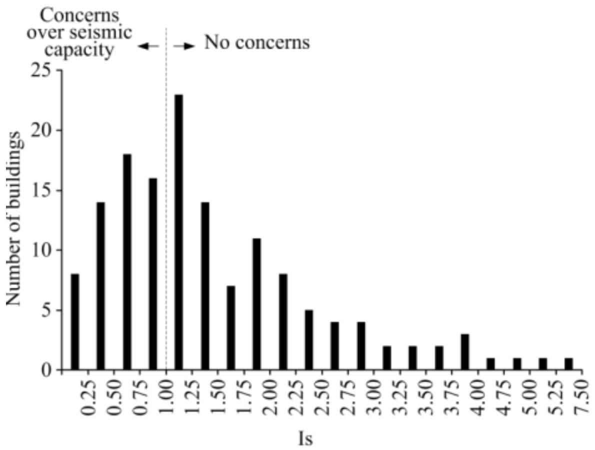


Fig. 12 – Results of preliminary assessment of the seismic capacity of typical street house buildings in Taiwan

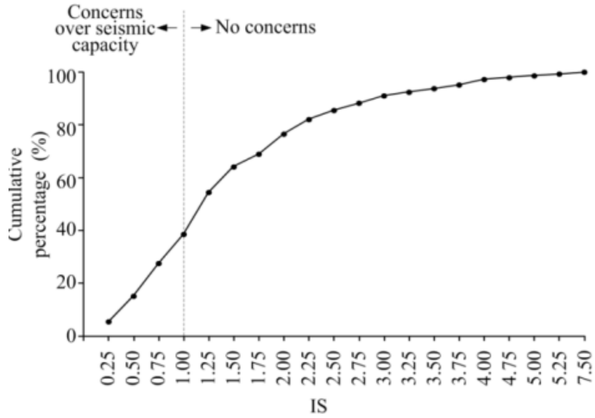


Fig. 13 – Results of preliminary assessment of the seismic capacity of typical street house buildings in Taiwan (accumulative percentage)

5. Conclusion

This study utilized NCREE's seismic database of school buildings to derive a relationship between the parameters of bare frames and the seismic capacity through a regression analysis. Based on the area of the column and wall members, the strength is converted into equivalent column area. Modification factors relevant to the seismic characteristics of low-rise buildings are chosen, and a preliminary assessment method for the seismic capacity of low-rise RC buildings is developed. The proposed method is applicable to RC or brick infill buildings up to six stories high with rigid floor panels. Using the seismic damage database of low-rise RC buildings, this method is verified to be conservative and able to serve as a screening tool. The method can be used in the preliminary screening of a large number of street house buildings according to their seismic capacity.

6. References

[1] Song J-C, Chiou T-C, Hwang S-J (2013): Investigation of column-to-floor ratio for seismic safety of RC school buildings in Taiwan. *NCREE-13-031*, NCREE, Taipei, Taiwan. (in Chinese)

[2] Chung L-L, Yeh Y-K, Chien W-Y, Hsiao F-P, Shen W-C, Chiou T-C, Chow T-K, Chao Y-F, Yang Y-S, Tu Y-S, Chai J-F, Hwang S-J, Sun C-H (2009): Technology handbook for seismic evaluation and retrofit of school buildings (second edition). Report No. *NCREE-09-023*, NCREE, Taipei, Taiwan. (in Chinese)

[3] Chiou T-C, Weng P-W, Shen W-C, Yeh Y-K, Chung L-L, Hwang S-J (2014): Structural databank of typical street house buildings in Taiwan. *NCREE-14-006*, NCREE, Taipei, Taiwan. (in Chinese)

[4] Hsu T-Y, Chung LL, Liao W-i, Chiou CK, Chien W-Y, Chow T-K (2003): Preliminary seismic evaluation method for typical buildings structures of primary and secondary schools. *NCREE-03-049*, NCREE, Taipei, Taiwan. (in Chinese)

[5] Su K-L (2008): Preliminary seismic evaluation method for typical school buildings in Taiwan. Master Thesis, Department of Civil Engineering, National Taiwan University, Taipei, Taiwan. (in Chinese)

[6] Japan Building Disaster Prevention Association (2001): Seismic evaluation and retrofit (English version). Japan Building Disaster Prevention Association, Tokyo.

[7] Kuo HY (2000): Fast seismic evaluation for RC school buildings. Master Thesis, Department of Architecture, National Cheng Kung University, Tainan, Taiwan.

[8] Sheu MS, Chen YH, Kuo HY (2003): Seismic assessment of RC street buildings with brick walls. *First EQTAP Hazard Assessment and Structural Mitigation Workshop*, Tokyo, Japan.

[9] Chiou T-C, Hwang S-J (2015): Tests on cyclic behavior of reinforced concrete frames with brick infill. Earthquake *Engineering & Structural Dynamics*, **44** (12), 1939-1958.