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# Fast Earthquake Resistance Assessment for School Buildings in Taiwan

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## ABSTRACT

In Taiwan the structural systems are very similar for fundamental schools, middle schools and high schools, as shown in Fig. 1. So most of the school buildings in disaster area of the Chi-Chi Earthquake collapsed with a unique failure mode of strong-beam and weak-column at ground floor, while the structures on and above second floor did not have serious damage. This paper presents a fast earthquake resistance assessment method for typical school buildings in Taiwan by assuming "shear building" mode at ground floor. The nonlinear lateral load and lateral deflection curve is plotted for each single vertical member on ground floor. That is the Q-Δ curve for each RC column, RC wall and brick wall on ground floor. Then, the nonlinear Q-Δ curve of the whole structure is obtained by superposing all the Q-Δ curves of vertical members on ground floor. Applying equal energy process, the equivalent elastic and bi-linear Q-Δ curves are plotted. Finally, the collapse EPA and deflection ductility ratio of the school building are found from equivalent elastic and bi-linear Q-Δ curves. In order to simplify the calculation of EPA, a set of simple equations are derived when brick walls fail or when RC walls fail or when columns beside windows fail in the structure.

## INTRODUCTION

There has been a standard RC structural system for primary school buildings, middle school buildings and high school buildings in Taiwan since 1950. Classrooms are on one side and corridor is on the other side as shown in Fig. 1. In general it is 2 story high in countryside and 3 to 4 story high in metropolitan area. There are many brick shear walls or RC shear walls in the direction perpendicular to corridor, but little walls in the direction parallel to corridor. So the earthquake resistant capacity in the direction perpendicular to corridor is much more than that required by the Building Code of Taiwan. However, the earthquake resistant capacity in the direction parallel to

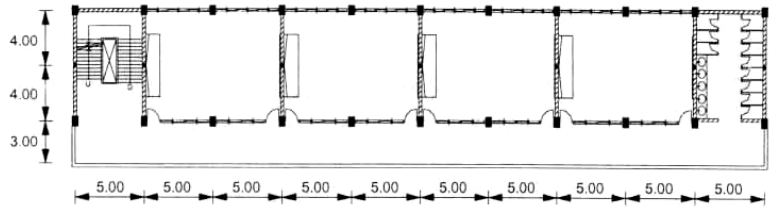



Fig. 1 Typical plan of school buildings in Taiwan

corridor could be just equal to that required by the Building Code, if the design and construction process are sound. So almost all the collapsed school buildings failed in the direction parallel to the direction of corridor in disaster area of the Chi-Chi earthquake. Hence, in this paper the Q-Δ curve of the school building will be examined in the direction parallel to corridor only, not in the direction perpendicular to corridor.

The existing Taiwan Building Code specifies that school building is an important structure which must be designed with strong-column and weak-beam structures. Because the structural designers do not consider that beams are integrally constructed to slabs; some walls are sitting on top of the beam and some walls are hanging from bottom of the beam. So the effective cross-sectional area of RC beam is very huge realistically. In other words, it is very difficult to design RC columns which are stronger than RC beams. Since the dream of strong-column and weak-beam was totally broken in disaster area of the Chi-Chi earthquake, the capacity and stiffness might be considered to be infinitive for RC beams. That is to say, it is reasonable for this paper to presume the pushover curve of vertical member being deflected in "shear-building" mode.

## PUSHOVER CURVE FOR RC COLUMN

The pushover curve of RC column with 2 claimed ends is referred from Chang [1] and Kuo [2]. The following equations are valid for short and long columns.

1. cracking load

$$Q_{cr} = \min(Q_{fc}, Q_{dc}) \quad (1)$$

where $Q_{fc} = 2M_{cr} / H$ (2)

$$Q_{dc} = 0.2\sqrt[3]{f'_c}\,(0.75 + 2.8\,d/H) \cdot (1 + \beta_a + \beta_d)\, b\, d\, m \quad (3)$$

$$\beta_a = \sqrt{100\,\rho} - 1 \quad (4)$$

$$\beta_d = \sqrt[4]{1000/d} - 1 \quad (5)$$

$$m = 0.748\sqrt[3]{1 + N/(3.43A_g)} \quad (6)$$

in which $M_{cr}$ is cracking moment of column (N-mm); $H$ is net height of column (mm); $b$ is width of column (mm); $d$ is effective depth of column (mm); $\rho$ is longitudinal reinforcement ratio; $N$ is axial compressive load (N); $A_g$ is gross cross-sectional area (mm$^2$).

2. ultimate load

$$Q_u = \min(Q_{fu}, Q_{su})$$

where $Q_{fu} = 2M_u / H$ (7)

$$Q_{su} = Q_{dc} + (A_h f_{yh} d)/S \quad (8)$$

$M_u$ is ultimate flexural moment of column; $A_h$ is cross-sectional area of hoop (mm²); $S_h$ is spacing of hoop (mm); $f_{yh}$ is yield stress of hoop (MPa).

3. yield load

$$Q_y = 0.85 \min (Q_{fu}, Q_{su}) \quad (9)$$

4. failure terminate load

$$Q_f = (Q_{cr} + Q_y)/2 \quad (10)$$

5. incremental flexural deflection with 2 claimed ends:

$$\Delta_f = \frac{(\Delta Q) \cdot H^3}{12\,(C_f\, E_c\, I_{gt})} \quad (11)$$

where $\Delta Q$ is incremental shear force (N); $I_{gt}$ is the transformed moment of inertia (mm⁴); $E_c = 4700\sqrt{f_c'}$ (MPa) and $C_f$ is reduction factor for flexural stiffness.

at elastic stage (0 ~ $Q_{cr}$): $C_f = 0.78$ (12)

at cracking stage ($Q_{cr}$ ~ $Q_y$):

$$C_f = 0.0006806\, f_c' \sqrt{1 + N/(3.43 A_g)} \quad (13)$$

but $0.15 \le C_f \le 0.78$

at yield stage ($Q_y$ ~ $Q_u$ ):

$$C_f = 0.0006941 \times N/(f_c'\, \rho_s\, A_g) \quad (14)$$

but $0.03 \le C_f \le 0.15$

where $\rho_s = \dfrac{2A_h (b - 2t + d + t)}{(b - 2t)\,(d - t)\, S_h}$

(15)

$\rho_s$ is the volume ratio of hoop; $t$ is thickness of rebar cover (mm).

at failure stage:

$C_f = -0.0026$ when $Q_{su} - Q_{fu} > 10$kN

$C_f = -0.03$
when $-10\text{kN} \le Q_{su} - Q_{fu} \le 10$kN

$C_f = -0.13$ when $Q_{su} - Q_{fu} \le -10$kN

6. incremental shear deflection under incremental load $\Delta Q$:

$$\Delta_s = \frac{2.4\,(\Delta Q)\,(1 + \upsilon)\, H}{C_s\, E_c\, A_{gt}} \quad (16)$$

in which $\upsilon$ is Poison's ratio of concrete; $A_{gt}$ is gross cross-sectional area of column including equivalent transferred area of steel; $C_s$ is reduction factor for shear rigidity.

at elastic stage (0 ~ $Q_{cr}$):

$\upsilon = 0.2$ and $C_s = 0.4$

at cracking stage ($Q_{cr}$ ~ $Q_y$):

$\upsilon = 0.3$, $C_s = 35 N\rho_s / (f_c' A_g)$,
but $C_s \ge 0.07$

at yield stage ($Q_y$ ~ $Q_u$):

$\upsilon = 0.4$, $C_s = 4.5 N\rho_s / (f_c' A_g)$,
but $C_s \ge 0.01$

at failure stage ($Q_u$ ~ $Q_f$):

$\upsilon = 0.5$, $C_s = -1/2500$
when $Q_{su} - Q_{fu} > 10$kN

$C_s = -1/280$
when $-10\text{kN} \le Q_{su} - Q_{fu} \le 10$kN

$C_s = -1/150$ when $Q_{su} - Q_{fu} \le -10$kN

7. total incremental deflection $\Delta_T$ due to incremental load $\Delta Q$, is the sum of $\Delta_f$ and $\Delta_s$:

$$\Delta_T = \Delta_f + \Delta_s \quad (17)$$

## PUSHOVER CURVE FOR RC SHEAR WALL

The pushover curve of RC wall with 2 claimed ends is employed from Kuo [2]. In Kuo's paper, the 45° diagonal shear crack starts from the top of boundary column and stops at top of bottom beam and then goes horizontally to the other boundary column as shown in Fig. 2. And lateral deflection is the sum of flexural deflection and shear deflection.

1. cracking load $Q_{cr} = \min(Q_{fc}, Q_{dc})$ (18)

where $Q_{fc} = 2M_{cr}/H$ (19)

$$Q_{dc} = \frac{1}{6}\sqrt{f_c'}\,A_g\left[1+\frac{N}{14A_g}\right] \cdot \{1-0.1\,(1.9H/W_T-2.6)(1+2A_g'/A_g)\} \quad (20)$$

$N$ is axial compression force (N); $H$ is net height of shear wall; $W_T$ is total width of wall including width of boundary columns (mm); $A_g'$ and $A_g$ are cross-sectional area of boundary columns and wall (mm²).

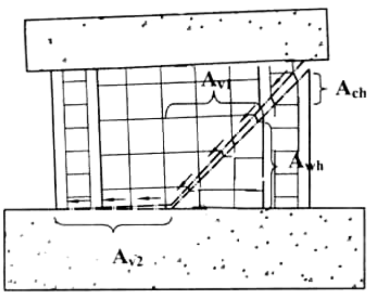



Fig. 2 Rebars cut by diagonal and horizontal shear crack

2. ultimate load $Q_u = \min(Q_{fu}, Q_{su})$ (21)

where $Q_{fu} = 2M_u/H$ (22)

$$Q_{su} = Q_{dc} + 0.324\,(A_{ch} + A_{wh} + A_{v1})f_y + 0.48\,(A_{v2}\,f_y + N) \quad (23)$$

and $A_{ch}$ ($A_{wh}$) is the cross sectional area of horizontal rebars of column (wall) cut by diagonal shear crack; $A_{v1}$ ($A_{v2}$) is the cross-sectional area of vertical rebars of wall cut by diagonal (horizontal) shear crack; $f_c'$ and $f_y$ are specific compression stress of concrete and yield stress of rebars (MPa).

3. lateral deflection at crack load $Q_{cr}$ is the sum of flexural and shear deflections:

$$\Delta_c = \frac{Q_{cr}\,H^3}{3.6\,E_c\,I_g} + \frac{7.2\,Q_{cr}\,H}{E_c\,A_g} \quad (24)$$

4. lateral deflection at ultimate load $Q_u$ is the sum of flexural and shear deflections:

$$\Delta_u = 0.0038\,(8.5H/W_T-1)(1-0.72I_b'/I_g)\,H + 0.011(1-0.37H/W_T)(1-0.5A_b'/A_g)H \quad (25)$$

where $I_b'$ is uncracked moment of inertia of boundary columns for the portion thicker than wall with respect to central axis (mm⁴); $I_g$ is uncracked moment of inertia of boundary columns and wall with respect to central axis (mm⁴); $A_b'$ is the cross-sectional area of boundary column for the portion thicker than wall (mm²).

5. lateral deflection between $Q_{cr}$ and $Q_u$. The lateral deflection ($Q$, $\Delta$) between crack point ($Q_{cr}$, $\Delta_c$) and ultimate point ($Q_u$, $\Delta_u$) is described as a nature logarithmic curve:

$$Q = a\,(\ln\Delta) + b \quad (26)$$

in which $a = \dfrac{Q_u - Q_{cr}}{\ln\Delta_u - \ln\Delta_c}$ (27)

$$b = \frac{Q_{cr} \ln \Delta_u - Q_u \ln \Delta_c}{\ln \Delta_u - \ln \Delta_c} \quad (28)$$

## PUSHOVER CURVE FOR BRICK MASONRY SHEAR WALL

The lateral load and lateral deflection curve of a brick masonry shear wall with thickness $t$, net height $H$ and effective width $W_{eff}$ is calculated from Kuo [2]. The effective width $W_{eff}$ is the minimum of net height and net width of wall. In Kuo's paper [2], the ultimate lateral load $Q_u$ and its corresponding lateral deflection $\Delta_u$ are calculated first. Then, the $Q$-$\Delta$ curve is assumed to be polynomial equation passing through origin and the ultimate point.

1. ultimate lateral load $Q_u$:

$$Q_u = \frac{4at}{15H}\left(W_{eff}^2 + H^2 + \sqrt{W_{eff}^4 + 14\,W_{eff}^2\,H^2 + H^4}\right) f_t \quad (29)$$

where $f_t$ is the maximum tensile strength of brick wall which is the combination of brick block and mortar. That is:

$$f_t = 0.13 f_{tm} + 0.435\,(f_{tb} + f_{mb}) \quad (30)$$

in which $f_{tm}$ is maximum tensile strength of mortar (MPa); $f_{tb}$ is maximum tensile strength of brick block (MPa); $f_{mb}$ is interface tensile strength between mortar and brick block (MPa). $a$ in Eq. (29) is correlation coefficient from experimental test. From Kuo's research [2]:

for 3-side confined brick wall

$$a = 0.1108\ W_{eff} / H \quad (31)$$

for 4-side confined brick wall

$$a = 0.2591\ W_{eff} / H \quad (32)$$

2. ultimate lateral deflection $\Delta_u$ :

$$\Delta_u = \frac{Q_u}{E_u\,t}\left[\frac{H}{W_{eff}}\left(2.375 + \frac{2H^2}{W_{eff}^2}\right) + \frac{3W_{eff}}{H}\right] \quad (33)$$

in which $E_u$ is secant modulus of elasticity of brick masonry wall at ultimate load.

for 3-side confined brick wall

$$E_u = 147.13\left(\frac{W_{eff}}{H}\right)\sqrt{f_b'} \quad (34)$$

for 4-side confined brick wall

$$E_u = 296.53\left(\frac{W_{eff}}{H}\right)\sqrt{f_b'} \quad (35)$$

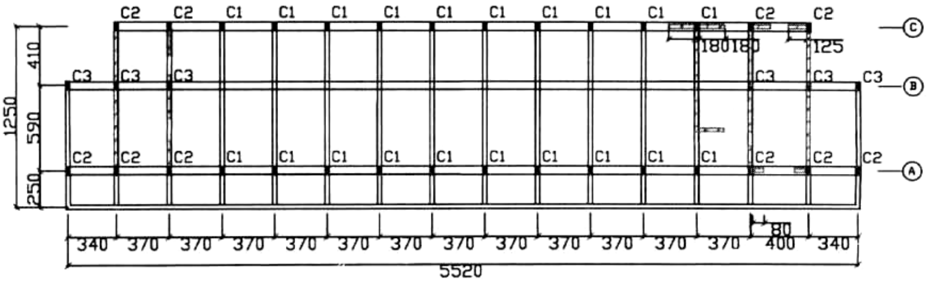
$f_b'$ is uni-axial compression strength of brick block (MPa).

3. lateral load and lateral deflection curve of brick masonry wall is assume polynomial equation as:

$$Q = Q_u\left[3\frac{\Delta}{\Delta_u} - 3\left(\frac{\Delta}{\Delta_u}\right)^2 + \left(\frac{\Delta}{\Delta_u}\right)^3\right] \quad (36)$$

## EXAMPLE

Figure 3 is the structural plan of Machine Lab of Minshyong Vocational School which collapsed in X direction at the after shock of the Chi-Chi Earthquake. Figure 4 shows the dimension and reinforcement of three different columns. Figure 5 shows the elevation of Frame A and Frame C. Thickness of brick masonry wall is 23cm. Widths of brick walls are 180cm, 125cm and 80cm respectively. Short-term strength of rebar and concrete increases 14% and 18% than their long-term strengths. Thus, $f_y$ = 1.14 × 362MPa;



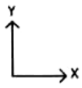




Fig. 3 Plan of machine lab of Minshyong School

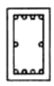
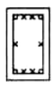
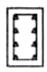
| C1 | C2 | C3 |
|---|---|---|
|  |  |  |
| ○10-#7 | ○ 4-#7 △ 6-#6 | △ 8-#5 |
| #3@25 | #3@25 | #3@25 |
| 30X 50 | 30X 50 | 25X 40 |

Fig. 4 Dimension and rebar of column

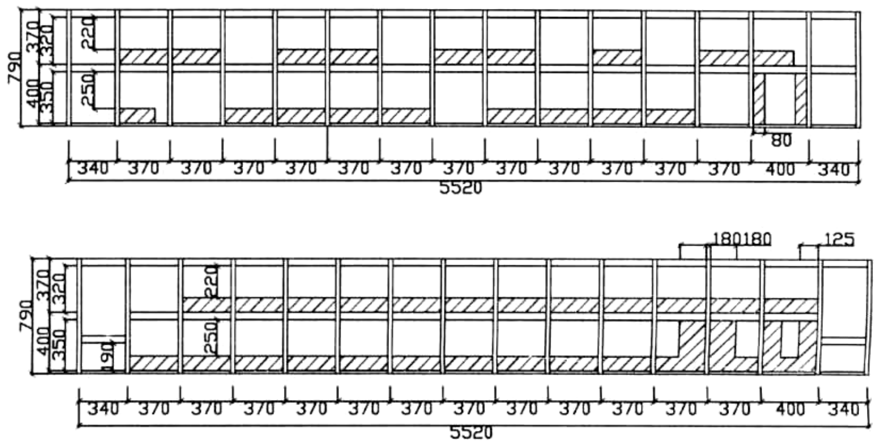



Fig. 5 Elevation of frame A and C

$f_c' = 1.18 \times 10.3$MPa; $f_b' = 13$MPa; $f_{tm} = 2$MPa; $f_{tb} = 2.86$MPa; $f_{mb} = 0.2$MPa. Figure 6 shows the lateral load-lateral deflection *Q*-Δ curves for each single vertical member in X direction. Figure 7 shows the *Q*-Δ curves for each group of vertical member. Superposing *Q* in Fig. 7 with respect to different deflections Δ, we get the solid *Q*-Δ curve of the whole structure in X direction as shown in Fig. 8. There are several peaks on solid *Q*-Δ curve of Fig. 8. Each peak represents the failure of one group of the vertical member. For instance, the first peak in Fig. 8 means the failure of 180cm BW1 brick walls. Since number of BW1 wall is 2 only, the failure of BW1 does not cause fatal collapse of the building. The second, third, fourth and fifth peaks are the failure of BW2, BW3, C3 columns and C1 columns correspondingly. In order to estimate the collapse EPAs at third, fourth and fifth peaks of Fig. 8, equivalent linear and bi-linear *Q*-Δ curves are plotted by equal energy process for each peak. The height of the bi-linear plateau is equal to the ultimate height of the solid curve. Let *W* be total weight acting on vertical members of ground floor; *C* be structural acceleration elastic response spectrum, EPA be effective peak ground acceleration. Then the elastic response earthquake load acting on the structures becomes:

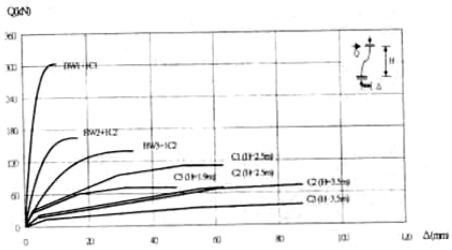



Fig. 6 *Q*-Δ curves of each single vertical member in X direction

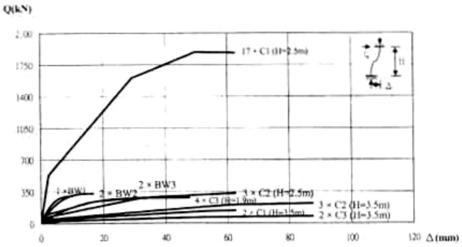
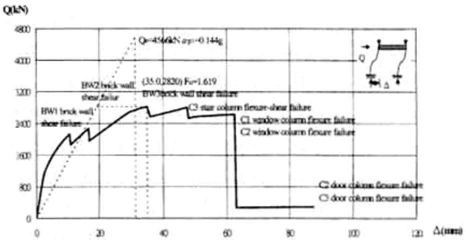
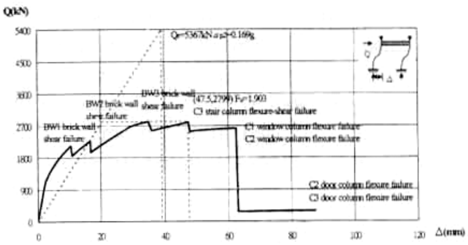


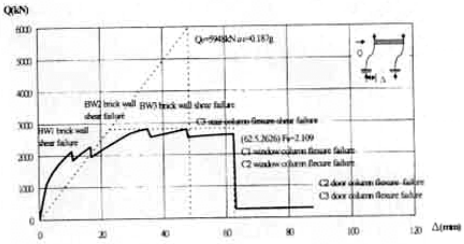

Fig. 7 *Q*-Δ curves of each group of vertical member in X direction




(a) When BW3 brick wall fails




(b) When C3 columns fail




(c) When C1 and C2 columns fail

Fig. 8 *Q*-Δ curve and elastic and bi-linear curve of whole structure in X direction

$$Q_e = \text{EPA}\,(C\,W) \tag{37}$$

$$\text{or}\quad \text{EPA} = \frac{Q_e}{C\,W} \tag{38}$$

The inelastic earthquake load acting on the structure is the height of bi-linear plateau:

$$Q_u = \frac{\text{EPA}\,(C\,W)}{F_u} \tag{39}$$

where $F_u$ is the load reduction factor due to the ductility of the structure. $F_u$ may be estimated by the ratio of $Q_e / Q_u$.

Substituting $C$ = 2.5, $W$ = 12977kN, and $Q_e$ = 4566kN; 5367kN and 5948kN, the maximum elastic loads of Fig. 8, into Eq. (38), we get EPA = 0.144g; 0.169g and 0.187g for different failure peaks. Hence the collapse effective peak ground acceleration of the Machine Lab is 0.187g at the fifth peak when C1 columns fail. The EPA in X direction recorded by the Central Weather Bureau of Taiwan was 0.269g. Consequently, the Lab collapsed at the moment when window columns C1 failed as shown in Photo 1.

Photo 1 Strong-beam and weak-column failure of school building

## SIMPLIFIED DESIGN EQUATIONS FOR SCHOOL BUILDING

For design of any building, the seismic resistant capacity should be larger than the external inelastic earthquake load acting to the structure. The external inelastic earthquake load can be expressed as Eq. (39). In which EPA is the design ground acceleration, 1.25 × 0.33 = 0.41g, required by Taiwan Building Code for school buildings in zone 1 area. $C$ is elastic acceleration response spectrum, 2.5 for low-rise buildings. The seismic resistant capacity of the structure at least should be checked for 3 levels: level 1 for failure of brick walls; level 2 for failure of RC wall; level 3 for failure of window column. At level 1 the capacity of RC walls is not fully developed. It is 89% developed from field observation and theoretical calculation [2]. The capacity of window columns is 81% developed. The capacity of door columns is 61% developed. At level 2 when RC walls fail, the capacities of window columns and door columns are 95% and 88% developed. At level 3, the capacity of door columns is 95% developed when window columns fail. So the design requirements for 3 levels might be written as:

$$\tau_{bw}\sum A_{bw} + 0.89\,\tau_{rcw}\sum A_{rcw} + 0.81\,\tau_{wc}\sum A_{wc} + 0.6\,\tau_{dc}\sum A_{dc} \geq \frac{0.41\,(2.5)\,(w\sum A_f)}{F_u} \tag{40}$$

$$\tau_{rcw}\sum A_{rcw} + 0.95\,\tau_{wc}\sum A_{wc} + 0.88\,\tau_{dc}\sum A_{dc} \geq \frac{0.41\,(2.5)\,(w\sum A_f)}{F_u} \tag{41}$$

$$\tau_{wc}\sum A_{wc} + 0.95\,\tau_{dc}\sum A_{dc}$$

$$\geq \frac{0.41(2.5)(w\sum A_f)}{F_u} \tag{42}$$

From field observation and correlation calculation, the ultimate strength for brick wall $\tau_{bw}$ = 0.161MPa; for RC wall $\tau_{rcw}$ = 1.96MPa; for window column $\tau_{wc}$ = 1.149MPa; for door column $\tau_{dc}$ = 0.639MPa. For typical school buildings in Taiwan, $F_u$ varies from 1.5 to 2. In this paper, $F_u$ taken as 1.5 when brick walls or RC walls fail; 1.9 when window column fail. $w$ is weight per unit floor area, taken as 0.0098MPa. Because RC shear walls were not commonly used for existing school buildings, so Eq. (41) might be neglected. Substituting $F_u$, $w$, $\tau_{bw}$, $\tau_{rcw}$, $\tau_{wc}$, $\tau_{dc}$ into Eq. (40) and Eq. (42), we get the design requirements which must be satisfied by one of the following:

$$\frac{\sum A_{bw}}{\sum A_f} + 4.5\frac{\sum A_{wc}}{\sum A_f} \geq 0.042 \tag{43}$$

$$\frac{\sum A_{wc}}{\sum A_f} \geq 0.0055 \tag{44}$$

in which, $\sum A_{bw}$ is the sum of cross-sectional area of brick walls on assessed floor in the direction for assessment; $\sum A_{wc}$ is the sum of cross-sectional area of window columns on assessed floor; $\sum A_f$ is the sum of floor area above assessed floor. In Eq. (43), RC wall is convertible to brick wall by assuming 1cm$^2$ of $A_{rcw}$ may be replaced by 10cm$^2$ $A_{bw}$. Door column is also converted into window column by assuming 1cm$^2$ of $A_{dc}$ = 0.5cm$^2$ of $A_{wc}$. On right-hand sides of Eq. (43) and Eq. (44), 0.042 and 0.0055 depend on the design EPA. They are not constants. If EPA changes, the values on right-hand side of Eqs. (40), (42) ~ (44) should be changed correspondingly. Figure 9 shows the relationship among design EPA, $\sum A_{wc}/\sum A_f$ and $\sum A_{bw}/\sum A_f$ which are obtained from Eqs. (43) and (44) by putting different design EPA to right-hand side. In Fig. 9, Prof-Sozen's simplified equation [3] is also plotted in dotted line for comparison. His equation is

$$\sum A_w + 0.5\sum A_c \geq 0.001\sum A_f \tag{45}$$

The collapse EPA of Eq. (45) is about 0.1g to 0.2g, if the structural system and structural material are identical to those of typical school buildings in Taiwan.

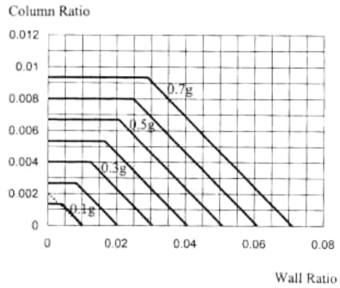



Fig. 9 Collapse EPA for different column ratio and wall ratio

## CONCLUSIONS

1. Eqs. (43), (44) and Fig. 9 provide fast earthquake resistant assessment for school buildings in Taiwan. From Fig. 9, it is very quick to assess the approximate collapse EPA for typical school building.
2. The proposed clear and simple $Q$-$\Delta$ curve, assuming shear-building model, provides more accurate assessment for collapse EPA than Fig. 9 does.
3. The collapse EPA from Fig. 9 and from $Q$-$\Delta$ curve were verified by some school buildings in disaster area of the Chi-Chi earthquake.

## ACKNOWLEDGEMENT

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