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AN ALTERNATIVE EVALUATION FOR DETERMINING SEISMIC VULNERABILITY OF BUILDING STRUCTURES

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ABSTRACT

A theoretical rationalization for ranking reinforced concrete frame buildings with masonry infill walls with regard to seismic vulnerability is presented. The method requires essentially only the dimensions of the structure as input, and is expressed in terms of where, in a two-dimensional plot of masonry wall and column percentages, its attributes are located. It is shown that increasing drift at the ground story (which is a robust expression of increasing vulnerability) is attained by decreasing either attribute.

KEYWORDS

Structural engineering, building vulnerability, drift control, design spectrum, shear frame, Erzincan earthquake

INTRODUCTION

In a paper describing the damage caused by the Tokachi-Oki earthquake of 1968, (Shiga et al., 1968) presented a format for evaluating the seismic safety of low-rise monolithic reinforced concrete construction. In this early treatment of the vulnerability of a given structural system to damaging ground motions, they proposed a critical attribute for seismic vulnerability which was expressed as the weight of the structure divided by the sum of the cross sectional areas of the columns and the walls at base. This format is very attractive because of the ease with which the required data can be acquired, and involved calculations are not necessary. The outcome is also a crisp numerical index which can be matched against a perceived or derived yardstick so that a judgment of vulnerability can be stated. The approach contained in the paper is difficult to generalize because it was derived exclusively in relation to a group of buildings and built in accordance with the Japanese practice of the 1960's. Recalibrating the format suggested by Shiga et al., (1968) can best be accomplished by testing its predictive power against observed phenomena in a collection of buildings with dimensional and material properties based on random choices made during the design and construction stages.

Conventional design procedures focus on arriving at proper component dimensions which will accommodate prescribed forces but a more important issue, at least for the design of ordinary low-rise buildings, is to keep structural displacements within prescribed limits (Sozen, 1989). This idea has been transcribed for the design of structural walls (e.g., Wallace, 1995) by deriving explicit expressions for the percentage of walls needed to limit drift values to within acceptable bounds.

The object in this paper is to expand the drift limitation concept to structural frames containing filler walls. When the approach is used in reverse, it may become an identifying tool for buildings with a high likelihood of severe damage under a given level of seismic loading. Empirical data from a recent earthquake in Turkey which occurred in Erzincan on 13 March 1992 is utilized for comparison although the procedure is generic, and can be implemented in any other environment.

DAMAGE DATA FROM ERZİNCAN

A full description of the effects of the Erzincan earthquake can be found in the EERI Special Publication 93-01 (1993). The strong part of the ground motion recorded in Erzincan lasted about 15 s, and the magnitude-6.8 event produced peak accelerations of 0.5 g and 0.4 g in the two horizontal directions, and 0.24 g in the vertical direction (Gülkan, 1993). Following that earthquake, one of the most comprehensive rehabilitation and reconstruction projects in Turkey has been staged, and nearly $1 billion has been spent for construction in a city with a pre-earthquake population of 90,000. One of the first works to be commissioned by the Ministry of Public Works and Settlement to Middle East Technical University (METU) in the immediate aftermath of the earthquake was to document the structural damage in 46 institutional buildings in the city. Among the pressing needs for the urgency of this mission was the fact that a great many of these buildings were schools.

The METU team developed floor plans for the buildings they inspected which were low-rise reinforced concrete structures one to five stories above ground. Their height distribution is shown in Fig. 1. Story heights ranged from 2.8 to 3.5 m, with some containing special areas with taller ceilings, or inclined roofs. Most buildings had filler walls of solid or hollow brick, or other material such as concrete block or even stone. Some had structural walls, although the use of such walls had been in general very sparse. Material and workmanship qualities were highly variable, but in general both fell much short of what good practice called for.

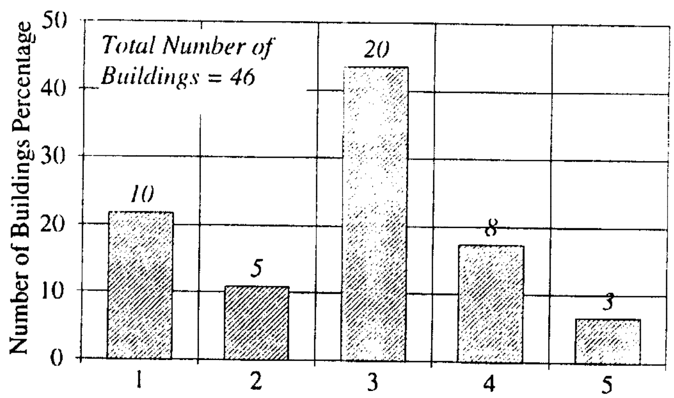


Fig. 1. Distribution of Buildings according to Number of Floors above Grade

It is not possible to reflect the range and variety of damage patterns observed in the 12,000 housing units damaged in the city on the basis of the 46 institutional buildings which comprised the early METU inventory. What makes the inventory valuable is the detail with which precise account of the damage which occurred in them was recorded. Through carefully conducted on-site surveys, an as-built portrait of each was developed.

None of the buildings listed in Fig. 2 suffered total collapse in spite of the strong motions although city-wide about 200 did so. The entries in Fig. 2 contain damage state descriptions encapsulated by the categories of "none", "light", "moderate" or "severe." The judgment concerning the damage state of a given building was the outcome of detailed considerations of the level of damage in structural members as well as "nonstructural" components, how widespread and representative that damage was, and how feasible retrofitting structural systems appeared to the evaluators. As prelude to the rationalization for damage which follows, Fig. 2 contains information on column and wall areas normalized with respect to the floor areas.

GROUND FLOOR DRIFT FOR BUILDINGS WITH MASONRY WALLS

The following rationalization is developed not only as a device to assimilate the experience of the buildings listed in Fig. 2, but as a plausible explanation, using only elementary earthquake engineering and structural mechanics knowledge of why they survived. This assimilation will necessarily gloss over a broad number of parameters which no rigorous treatment would contemplate ignoring but our target is a modest one: if we accept the premise that structural damage or relative lack thereof on the family of reinforced concrete frame buildings with little or no structural walls occurred because of differences of their mean drift response at the ground floor, can we explain that on the basis of a self-contained theory, and does the body of data in Fig. 2 support that theory? It is perhaps justified to be color blind if all we require is to discern whether the simple ratios in the right-side columns of Fig.2 fall into logically differentiable bins.

One of several simplifications is pictured in Fig. 3, a "representative" partial elevation of a building frame which embodies a pair of columns connecting into a pair of girders at their ends, enclosing a masonry wall. The properties of the masonry units, and effects of openings for doors, windows, etc. are not recognized except through the argument that any set of wall properties can be expressed in some equivalent form through the device in Fig. 3. In this representation structural walls of reinforced concrete do not influence the frame behavior but the existence of wall columns is tolerated if they do not influence the dynamic response to shed its fundamental property, to be expressed through Eq. (14) below. Our development is also impervious to such deleterious effects as short column effects, torsional response, and local weaknesses: the assumption here is that a collection of plane structural items such as shown in Fig. 3 translate in unison when excited by the ground waves.

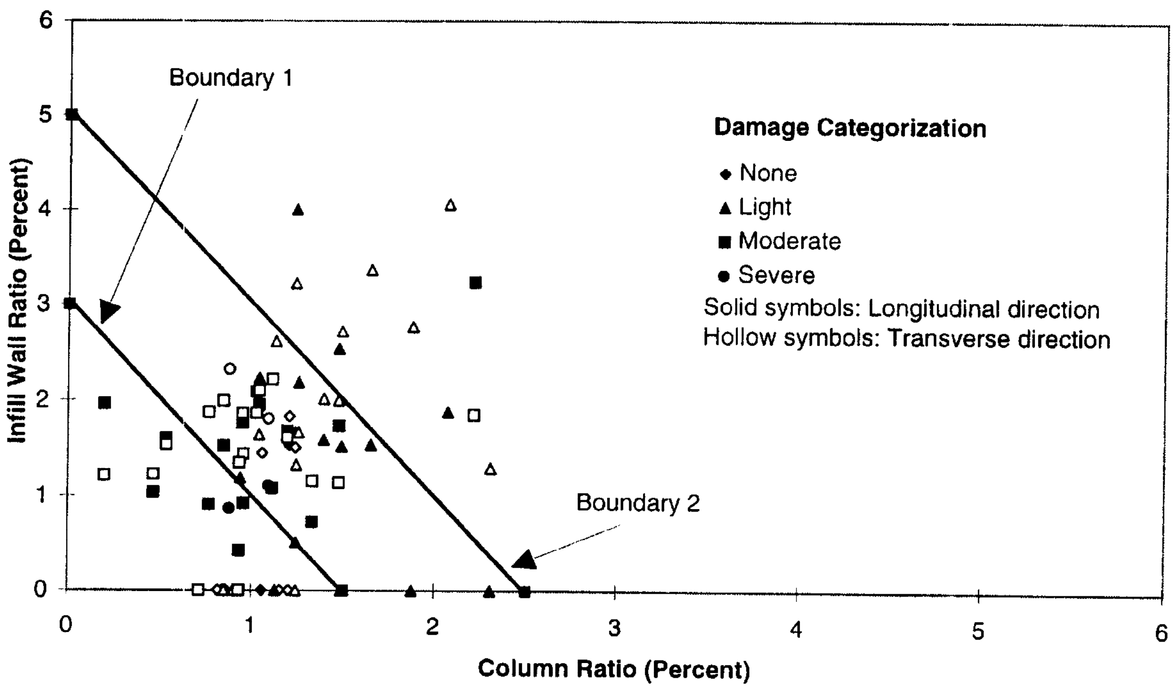


Fig. 2. Damage Data

We define the ground motion in terms of the displacement response spectrum. For ensuring immediate recognition, this will be done in terms of the UBC (1994) equation

$$V = \frac{Z\ I\ C}{R_w}\ W \tag{1}$$

$$C = \frac{1.25\ S}{T^{2/3}} \le 2.75 \tag{2}$$

The acceleration dependent range of the spectrum as seen from Eq. (2) for the average condition S = 1.2 is up to a period of 0.4 s, although the precise value of this transition period is dependent on the spectral properties of a given ground motion. If we denote this period by T_c, then the spectral acceleration S_a for $R_w = 1$ may be written as

$$T < T_c \qquad S_a = 2.75\ Z\ g \tag{3}$$

If, for high seismic regions Z = 0.4 is adopted $S_a = 11$ (m/s²) and

$$S_d = S_a \frac{T^2}{4\ \pi^2} = 0.27\ T^2 \qquad \text{(m)} \tag{4}$$

Working with S = 1 in the velocity dependent part of the spectrum

$$T > T_c \qquad S_a = \frac{6}{T^{2/3}} \qquad (\text{m/s}^2) \tag{5}$$

$$S_d = 0.15\ T^{4/3} \qquad \text{(m)} \tag{6}$$

Equations (4) and (6) match at $T = T_c$, and more importantly, they are representative of the inelastic displacements of yielding systems for ductility ratios of less than about 5 except for the displacement dependent region of the spectrum for which the basic premise of this paper is not applicable because of the importance of higher modes which govern response. Either of Eqs. (4) and (5), or their equivalent forms, have been utilized in expressing the spectral displacements of yielding systems (Sozen, 1989; Wallace, 1995). In view of the approximations involved, the more conservative expression in Eq. (6) will be used in the following.

We now turn our attention to the substructure in Fig. 3 which is part of a multi-story building frame. The translational stiffness k_c derives from the contributions of the columns and the masonry wall, referred to with subscripts c and m, respectively:

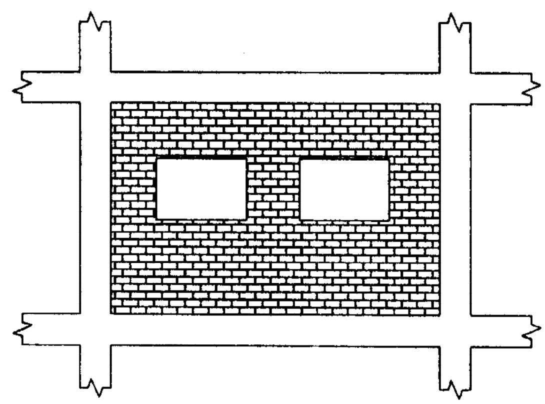


Fig. 3. Frame with Infill Wall

$$k_c = c_c \frac{E_c\ I}{h_s^3} \tag{7}$$

c_c = a constant depending on end fixity (= 12 if girders are infinitely stiff)

Expressing $I = A_c\ r^2$ where r = section radius of gyration and

$$A_c = p_c\ A_f \tag{8}$$

where

p_c = column percentage

A_f = floor area

λ = h_s/r

With these substitutions Eq. (7) becomes

$$k_c = c_c \frac{E_c\ p_c\ A_f}{h_s\ \lambda^2} \tag{9}$$

Wall stiffness derives from both the bending and the shearing distortions:

$$k_m = \frac{1}{\dfrac{h_s^3}{c_m E_m A_m l_w^2} + \dfrac{h_s}{c_{ma} G_m A_m}} \tag{10}$$

In Eq. (10)

c_m := a hybrid constant depending on wall end conditions and masonry cross section properties related to bending ($c_m = 3 \times \frac{1}{12} = 0.25$ for a cantilever wall of solid masonry units.)

c_{ma} := a constant depending on masonry cross section properties related to shear distortions ($c_{ma} = 0.83$ for a wall of solid masonry units.)

Simplifications are possible in Eq. (10). If $G_m = 0.4\, E_m$ and $A_m = p_m A_f$ where p_m = masonry wall percentage then

$$k_m = \frac{E_m p_m A_f}{h_s \left\{ \dfrac{(h_s/l_w)^2}{c_m} + \dfrac{2.5}{c_{ma}} \right\}} \tag{11}$$

Addition of Eqs. (9) and (11) yields the story stiffness

$$k_{story} = \frac{c_c E_c p_c A_f}{h_r \lambda^2} + \frac{E_m p_m A_f}{h_s \left\{ \dfrac{(h_s/l_w)^2}{c_m} + \dfrac{2.5}{c_{ma}} \right\}} \tag{12}$$

The story mass equals the mass per unit area μ times the total floor area.

$$m_{story} = \mu A_f \tag{13}$$

If the frame consists of stories "identical" in their dynamic properties then the shear beam expressions for mode shapes and periods for mode k may be invoked:

$$\phi_k(x) = \sin \frac{2k - 1}{2} \frac{\pi x}{l} \tag{14}$$

$$T_k = \frac{4n}{2k-1} \left(\frac{m_{story}}{k_{story}} \right)^{1/2} \tag{15}$$

n = number of stories
l = total height of frames = n h_s

The portion of the total mass mobilized for a shear beam in the fundamental mode is approximately 80 percent. We may confine our attention to only mode k = 1. The appellation "shear beam" in this context is not entirely inconsistent because although girders rotate at column ends story translations are the only degrees of freedom, and the same uniform properties hold throughout the height. Equation (14) states that the greatest drift occurs at the ground story where x = h_s in the fundamental mode:

$$\phi_1 (h_s) = \sin \frac{\pi}{2n} \quad (16)$$

The spectral displacement multiplies the amplitude of Eq. (14) at x = 1, which is equal to unity. Upon substituting Eqs. (12) and (13) into Eq. (15) for mode k = 1 we obtain

$$T = 4n \left\{ \frac{\mu}{\dfrac{c_c E_c p_c}{h_s \lambda^2} + \dfrac{E_m p_m}{h_s \left\{ \dfrac{(h_s/l_w)^2}{c_m} + \dfrac{2.5}{c_{ma}} \right\}}} \right\}^{1/2} \quad (17)$$

The closed form expression for the fundamental period can now be substituted into Eq. (6) which yields the top displacement S_d. We note that the mean ground story drift (MGSD) is equal to the top displacement scaled to the first story displacement normalized with respect to the story height:

$$\text{MGSD} = \frac{S_d}{h_s} \sin \frac{\pi}{2n} \quad (18)$$

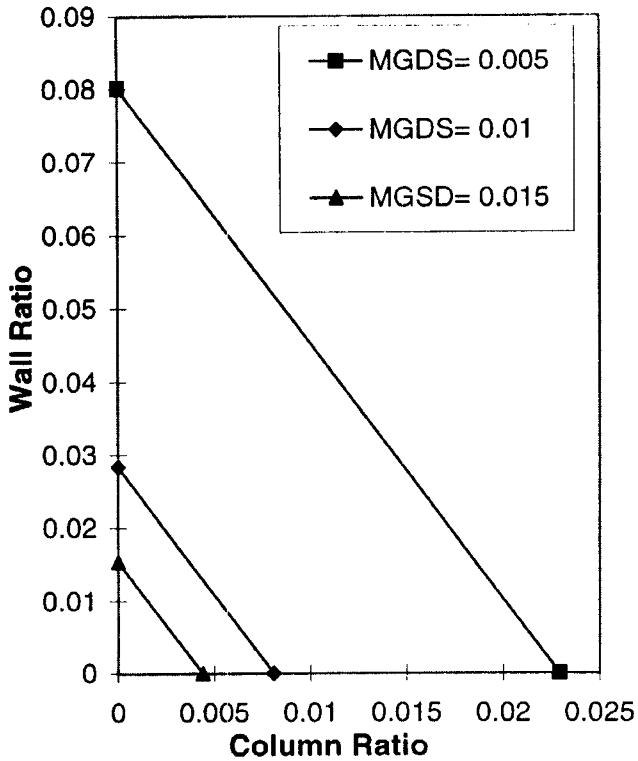
Therefore

$$\text{MGSD} = \frac{0.95\, n^{4/3}}{h_s^{1/3}} \sin \frac{\pi}{2\, n} \left\{ \frac{\mu}{\dfrac{c_c E_c p_c}{\lambda^2} + \dfrac{E_m p_m}{\dfrac{(h_s/l_w)^2}{c_m} + \dfrac{2.5}{c_{ma}}}} \right\}^{2/3} \quad (19)$$

Equation (19) provides a measure of the damage to be expected. The measure is not absolute in that implicit in it are assumptions of a uniform building and of effective peak ground acceleration and frequency content reflected by the displacement spectrum. They do serve to assimilate the evidence, and to identify the relatively more important parameters in design. Many of these parameters have been utilized in current technology to identify patterns of vulnerability. To re-emphasize these parameters we list them:

(a) Number of stories, n
(b) Building weight, μ
(c) Material properties of elements contributing to lateral resistance including nonstructural walls, E_c and E_m
(d) Type and fixity conditions of masonry units, c_m, c_{ma}
(e) Type of framing, c_c
(f) Column and wall slenderness, λ, (h_s/l_w)
(g) Mean story height, h_s
(h) Type of foundation and soil profile (concealed in the spectrum shape)
(i) Ratio of column area to floor area, p_c
(j) Ratio of masonry wall area to floor area, p_m

For paucity of space it is not feasible to display the influence of each parameter but Eq. (19) serves to emphasize the obvious: to keep the mean ground story drift to a given level, the sum of p_c and p_m each multiplied by different constants must be kept a constant. If, for "average" conditions, we assume $h_s = 3$ m, $\mu = 0.8\ \text{t/m}^2$, $E_c = 2e7\ \text{kN/m}^2$, $E_m = 0.025\ E_c$, $l_w = 6$ m, $c_c = 4$, $\lambda = 20$, $c_m = 0.10$, $c_{ma} = 0.4$, $n = 4$ then the variation of column and wall percentages required to satisfy mean ground strong drift limits of 0.005, 0.001, and 0.015 are as displayed in Fig. 4. If the drift limit is kept as 0.01, then the required area ratios for n = 2, 4, 6 and 8 stories are as shown in Fig. 5.



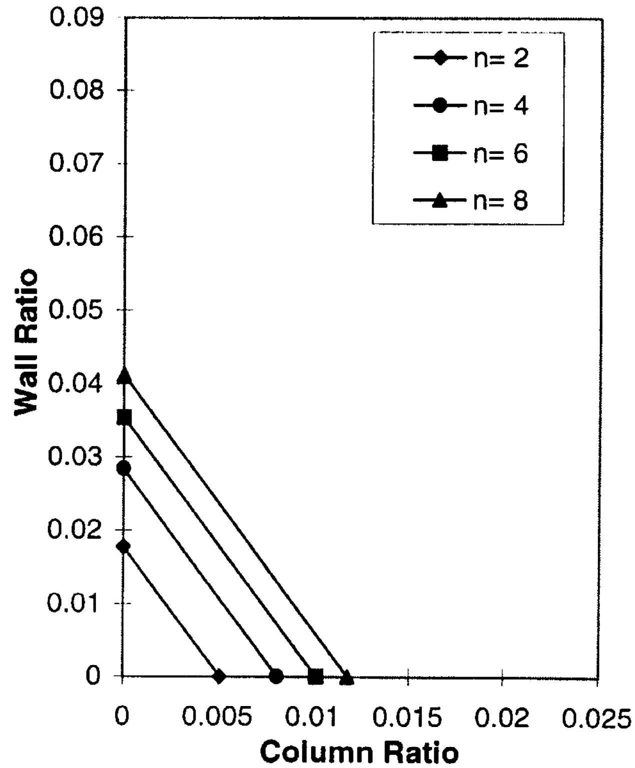


Fig. 4. Column and Wall Ratios Required to Limit Drift

Fig. 5. Effect of Number of Stories

VERIFICATION

The trends displayed in Figs. 4 and 5 are supported by the Erzincan data. In Fig. 2 the axes are the column and wall percentages where there is a definite trend toward greater observed damage for decreasing values of either index. The data in Fig. 2 are identified as "none", "light", "moderate" or "severe", and refer to the 46 buildings in the inventory. For each building, attributes of both directions were plotted. The lines designated as "Boundary 1" or "Boundary 2" define two triangular regions. If the two indices (the wall and column ratios) define a point inside these triangles, a particular building was considered to have been more heavily damaged if its attributes were closer to Boundary 1 than to Boundary 2. This rationalization cannot yet make absolute judgments about structural safety because it has not been calibrated with respect to material quality and detail, but in its current form bears out the generic prediction of Hassan and Sozen (1996). Used in reverse, it holds the potential of serving as a convenient identifier of seismic vulnerability for existing buildings.

CONCLUSIONS

A theoretical rationalization is developed to rank buildings with higher seismic vulnerability in an inventory of low-rise monolithic reinforced concrete buildings. The method requires only the dimensions of the structure as input, and is based on defining the ranking on a two-dimensional plot using column and masonry wall percentages. The wall percentage is the ratio of the effective masonry wall area in a given direction at the base of a building to the floor area above the base but the column percentage is simply the ratio of the sum of the column areas at the base to the floor area above the base. Wall columns can be included within this sum, but because of the underlying assumptions of the deflected pattern for the frames in the fundamental mode, the formulation can not be said to include shear wall systems.

The ranking of damage observed in a group of institutional buildings in Erzincan during the 13 March 1992 earthquake there shows that the data is in good agreement with the method.

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