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MAJOR CAUSES OF TYPICAL DAMAGES OF REINFORCED CONCRETE BUILDINGS DUE TO THE 1995 HYOGO-KEN NANBU EARTHQUAKE

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ABSTRACT

Reinforced concrete buildings were severely damaged due to the 1995 Hyogo-ken Nanbu Earthquake, and some of them completely collapsed. Most of these damaged buildings were those designed by the previous Japanese design code. Particularly the damage of medium high-rise buildings designed by the previous code was distinguished. The number of damaged buildings which were designed by the current Japanese design code revised in 1981 and were classified as "severely damaged" or "collapsed" is very few except for the buildings with particular structural layout. This paper describes the major factors on two destructive typical damages of reinforced concrete buildings. One is the complete collapse of mid-height story of medium high-rise buildings from seven to twelve stories which were found only for the buildings designed by the previous code. The other is the collapse of piloti of buildings with the soft first story which were found for the buildings designed by both the previous and current codes. Through this studies, it was concluded that, the location of the story which suffered damage like pancake collapse depended on vibration mode as well as strength distribution along the height of the buildings and insufficient lateral capacity and ductility of columns at that story caused such collapse, and that there were many factors which caused severe damage of piloti buildings, and reinforced concrete walls considered as non-structural walls were one of most important factors. The paper also proposes some design recommendation to avoid such catastrophic collapse.

KEYWORDS

1995 Hyogo-ken Nanbu earthquake; earthquake damage; reinforced concrete buildings; collapse of mid-height story; piloti.

COMPLETE COLLAPSE OF A MID-HEIGHT STORY OF MEDIUM HI-RISE BUILDINGS

Damage example

Photograph 1 shows a typical damage example of the eight-story office building which was designed by the previous code. This building had reinforced concrete framing in the upper four stories supported by steel-encased reinforced concrete(SRC) framing in the lower four stories, and steel in the column was developed up to the mid-height of the fifth story columns. The sixth story completely collapsed. Except sixth story, the damage level of the story was slight or small, as a whole, although some damages were observed such as the fracture of joint steel at the splice of steel due to tension, damage of column bases due to pulling out of anchor bolts at the base plate of steel, and damages of columns at the fifth story due to falling down of sixth story. The size of columns at the sixth and seventh stories was 65 cm × 65 cm and longitudinal steel bars in these columns were eight round bars with a diameter of 22 mm although the size and longitudinal steel bars in fifth story columns were 75cm × 75cm and twenty round bars with a diameter of 25mm, respectively. The ratio of the amount of cross sectional area of the longitudinal steel bars to the cross sectional area of the sixth story column was about 0.8% which was the minimum requirement specified in the previous code. and the amount of the longitudinal steel bars in the sixth story column was about one fourth as small as that in the fifth story column.

Photo.1. Complete collapse of mid-height story of a eight story building.

Design shear forces specified in the current and previous codes.

This damage pattern was found only in the buildings designed by the previous code. Therefore, in this section, the design shear force specified in the previous code is discussed by compared with that in the current code.
In the previous code, allowable stress design was specified for the horizontal earthquake force Fi at i-th floor was given by Eq. (1).

$$F_i = Z \cdot G \cdot K \cdot W_i \tag{1}$$

where,

Z = seismic zone coefficient (= 0.8 ~ 1.0 and 1.0 for Kobe),
G = soil-structure coefficient (= 0.8 ~ 1.0 and 1.0 in the most area in Kobe defined as the largest value of VII of Japanese seismic intensity scale),
K = seismic coefficient (=0.2 to height of 16m and below, and increased by 0.01 for every 4m above), and
W_i = sum of dead and live loads of i-th story.

In the current code, the lateral seismic force in current code is given by Eq. (2).

$$Q_i = C_i \sum_{i=1}^{n} W_i \tag{2}$$

$$C_i = Z \cdot R_t \cdot A_i \cdot C_o \tag{3}$$

where,

Q_i = seismic shear force at i-th story,
n = number of stories,
C_i = story shear coefficient,
Z = seismic zoning factor (= 0.7 ~ 1.0 and 1.0 for Kobe),
R_t = vibration characteristics factor and 1.0 if natural period of a building is less than 0.6 and the type of soil is so soft as that of most area in Kobe defined as VII of the Japanese seismic intensity scale,
A_i = vertical distribution factor, and
C_o = standard shear coefficient, and not less than 0.2 for the strong earthquakes which can occur several times during the lifetime of the building, and not less than 1.0 for the severe earthquakes which could occur once in the life time.

The stresses in the structural members shall be less than allowable stress for strong earthquakes given by Eq. (2) and the lateral load carrying capacity of each story shall be greater than Eq. (4) for severe earthquakes.

$$Q_{un} = D_s \cdot F_{es} \cdot Q_i \tag{4}$$

Where,

Q_{un} = required ultimate capacity for lateral load of a story,
D_s = structural characteristics factor and 0.3 for most ductile reinforced concrete frames, and
F_{es} = a shape factor which considers the rigidity and eccentricity factors and 1.0 for the building whose structural layout is regularly and well-balanced.

Equations (2) and (4) are simply rewritten by Eq. (5) for strong earthquakes and Eq. (6) for severe earthquakes respectively, for the most of low-rise and medium high-rise buildings in Kobe, if the minimum values of Co, Ds and Fes are considered.

$$Q_i = 0.2\, A_i \cdot \sum_{i=1}^{n} W_i \tag{5}$$

$$Q_{un} = 0.3\, A_i \cdot \sum_{i=1}^{n} W_i \tag{6}$$

In the following discussions, it is assumed that the mass of each story is the same, and the story height is 3.0m. Figure 1 shows story shear coefficients calculated by Eqs. (1), (5) and (6), and values of Eq. (1) times 1.5. By comparing the design story shear coefficients by Eqs. (1) and (5) because same allowable stress design is specified for these shear forces, in the both previous and current codes, it is easily found that design shear force specified in the previous codes is considerably smaller than that in the current code, at the upper stories of a building. And if the ultimate capacity for lateral load of each story of a building may be 1.5 times that defined by Eq. (1) and Eq. (6) is minimum requirement for severe earthquakes, the shortage of the ultimate capacity of a story designed by the previous code is much remarkable as the location of the story of a building is higher. However, the ultimate capacity of the stories near the uppermost story of a building designed by the previous code may be more than minimum ultimate capacity required by current code, because of minimum requirement such as size of members, minimum reinforcement, etc.

Estimation of Ultimate Capacity for lateral Load of Buildings Designed by the Previous Code

The ultimate capacity of columns of a story is assumed to be sum of ultimate lateral capacity of columns of a story considered, because the failure mechanism of this damage pattern is the complete collapse of a certain story. It is also assumed that the shear capacity of these columns is higher than their flexural capacity .

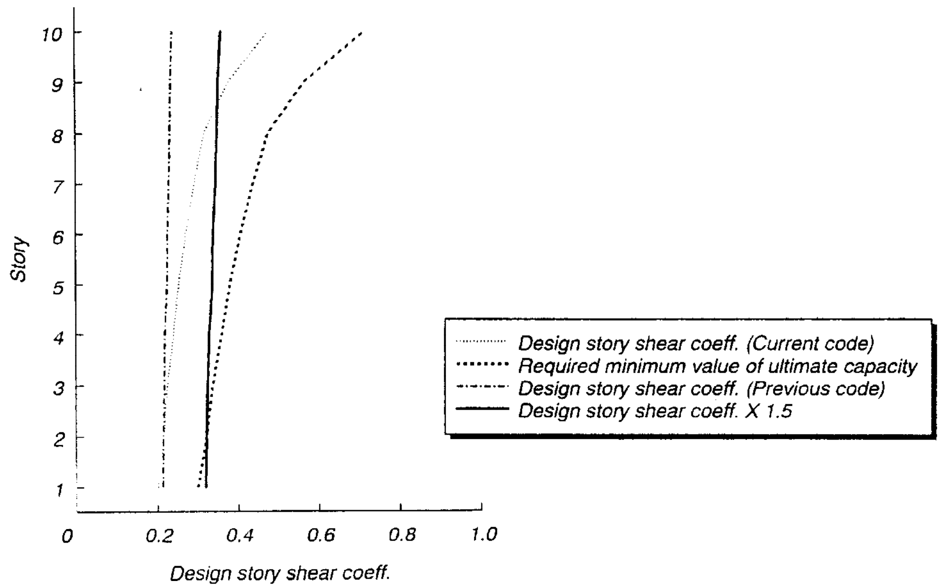

Fig. 1. Design story shear coefficients specified in the previous and current codes.

The flexural capacity of a column is given by Eq. (7).

$$M_u = 0.8\, a_t \cdot \sigma_y \cdot D + 0.5\, N \cdot D \left(1 - \frac{N}{b \cdot D \cdot \sigma_B}\right) \tag{7}$$

where,

M_u = flexural strength of a column (N mm),
a_t = amount of cross sectional area of longitudinal tensile steel bars (mm^2),
σ_y = tensile yield strength of longitudinal steel bars (MPa),
N = axial force in a column (N),
σ_B = compressive strength of concrete (MPa),
b = width of a column (mm), and
D = depth of a column (mm).

Equation (7) is rewritten by Eq. (8) , if $N / (b \cdot D \cdot \sigma_B)$ is assumed to be 0.1.

$$M_u = 0.8\, a_t \cdot \sigma_y \cdot D + 0.45\, N \cdot D \tag{8}$$

The shear Qm of a column at the flexural strength at both ends is given by Eq. (9).

$$Q_m = 2\, M_u / h_o \tag{9}$$

where,

h_o = clear height of a column (mm).

The ultimate capacity for lateral load of i-th story of a building is given by dividing sum of Qm of each column at i-th story by their sum of axial force, and the shear coefficient $_mC_i$ of i-th story at the ultimate capacity is given by Eq. (10). Here, the following assumptions are used.

i) The dimensions of depth and width of every column are the same.
ii) a_t in each column in a certain story considered is the same.
iii) D/ho is 1/3 for every column.
iv) The ratio of the sum of cross sectional area of the columns to the floor area at a certain story

considered is 1/100.

v) The mean unit mass of a building is 1.2 t / m^2.

vi) σy is 360 MPa.

$$ {}_mC_i = \frac{160}{n+1-i}\,{}_iP_t + 0.3 \tag{10} $$

where,

${}_iP_t = a_t$ / (bd) of the column at i-th story,

n = number of stories of a building, and

i = 1+ number of stories of a building above i-the story.

The first term and the second term in the right-hand side of Eq. (10) represent terms related to the longitudinal steel bars and axial force, respectively. The ultimate lateral capacity of 0.3 of story shear coefficient provided by the axial force may be large enough to satisfy allowable stress for about 0.2 of story shear coefficient specified in the previous code This results means that longitudinal steel bars of most of columns designed by the previous code were arranged by referring to their minimum requirement. However, when the axial force is reduced to be half due to up-down input motion, the second term become 0.15. Thus the ultimate capacity of buildings is remarkably affected by up-down motion.

Discussion

Equation (10) is the result based on flexural strength of columns considering damage patterns caused by the earthquake. If columns will fail before they reach flexural yielding or beams reach their ultimate strength, the ultimate capacity for lateral load of the story become smaller value than that given by Eq. (10) , so values that given by Eq. (10) is one of the possible maximum value of the ultimate capacity. Figure 2 shows the result of p_t of 0.3% which corresponds to the ratio of the minimum requirement of 0.8 % of longitudinal steel bars by comparing the minimum requirement of ultimate capacity specified in the current code given by Eq(6). The shortage of the ultimate capacity comparing to 0.3 of Ds is remarkable at mid-height stories of the building.

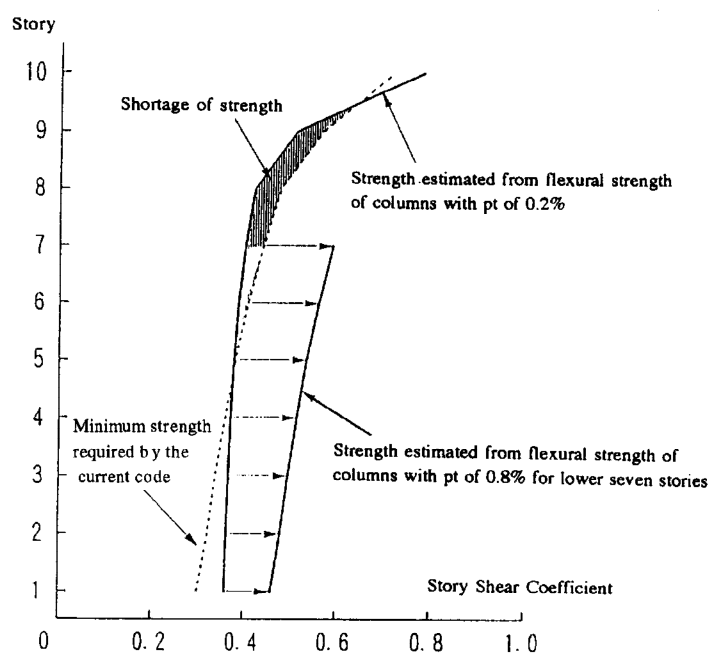

Fig.2. Estimated ultimate capacity of the buildings designed by the previous code and minimum ultimate capacity required by the current code.

In existing buildings, the dimensions of columns and amount of the longitudinal steel bars in columns are usually lager in the lower stories, and in Japan the lower stories of a building are commonly constructed by SRC if a building is higher than 21m. Such construction practice enhances the ultimate lateral capacity of lower stories of the building. In Fig. 2, the estimated value of ultimate lateral capacity of lower seven-story in which p_t of columns is assumed to be 0.8% considering steel plate in columns is also shown. The shortage of ultimate lateral capacity of mid-height stories is furthermore distinguished. The building shown in Photograph 1 is the typical damage example in which reinforced concrete sixth story above lower SRC stories completely collapsed.

Figure 3 shows the case studies for 10, 8, 6 and 4-story buildings. The solid line shows the story shear coefficient of ultimate lateral strength given by Eq. (10) in which pt is 0.3%, and the dashed lines show α A_i which has A_i distribution profile and comes in contact with the solid line. The multiplier α corresponds to D_s. The more the number of story of a building is, the higher the relative contact point is and the smaller the α is. The relative contact point and the value of α is largely affected by the shear distribution profile during earthquakes. It is expected that this profile is closer to A_i distribution rather than design shear distribution specified in the previous code.

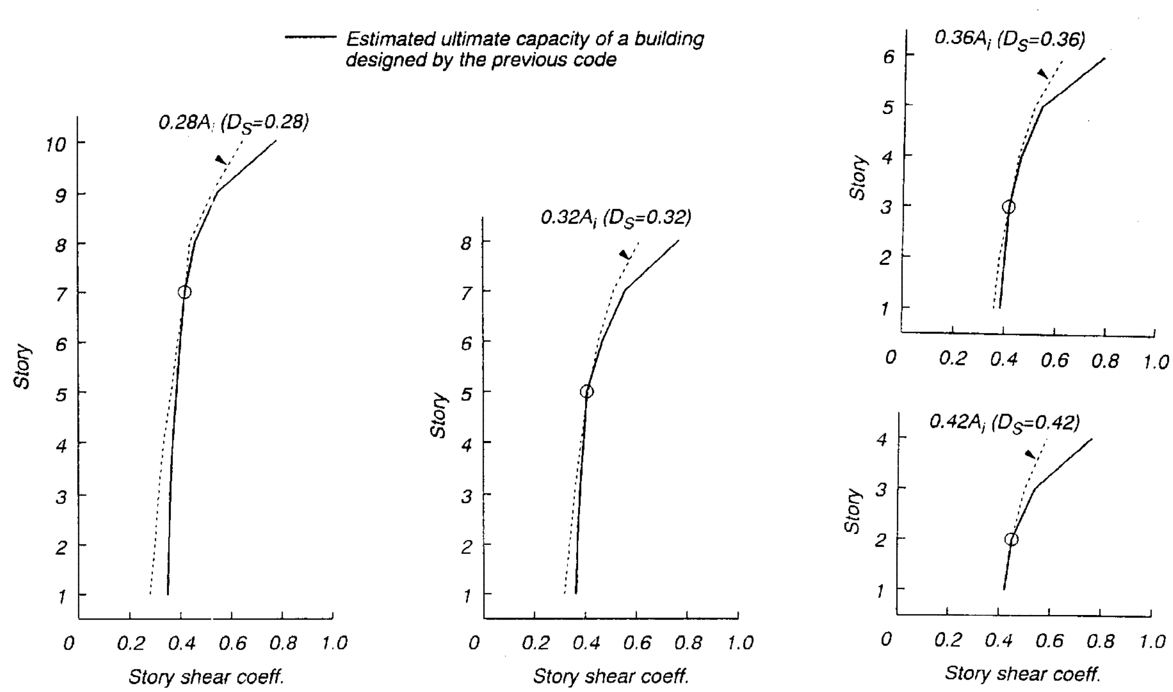

Fig. 3. Estimated ultimate capacity of 10, 8, 6 and 4-story buildings with 0.2% of p_t.

COLLAPSE OF PILOTI BUILDINGS

Damage Example

Photograph 2 shows a five-story building whose first story completely collapsed. This building was designed by the current design code. There are reinforced concrete walls in the upper four stories. This type of structures is called as piloti buildings and they suffered severe damage including those designed by the current code. Any severe damage was not observed in the upper four stories. The structural slit where thickness of concrete wall was half of that of the in-panel wall was placed between columns and in-panel walls. In the structural design, these in-panel walls were considered as non-structural walls and their structural effect was not considered . However, these walls might have worked as shear walls during the earthquake. Four of five piloti buildings which were designed by the current code and were identified as completely collapse by Building Research Institute had similar non-structural walls in the upper stories.

Photo. 2. Complete collapse of a piloti building.

Factors for Damage

There are many factors which might have cased such damages. These factors are discussed below.

1) Shear distribution profile.

Response shear distribution profile along the height of the building due to input earthquake motion may be top heavy distribution like A_i distribution if the structural layout of the buildings are well-balanced and regularly shaped. However, this distribution profile is not appropriate for design shear for piloti buildings because of soft stiffness of the piloti story (see Fig. 4).

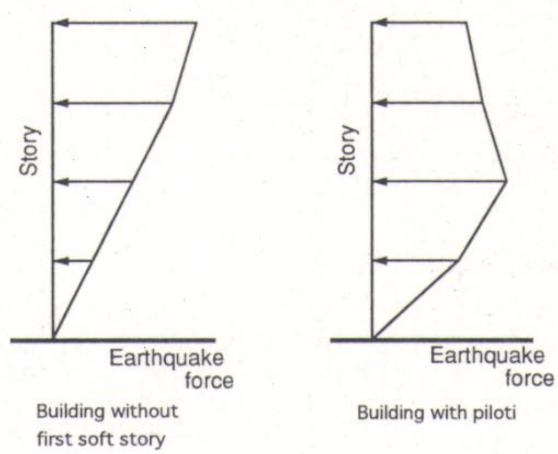

Fig. 4. Examples of earthquake force distribution of building with and without first soft story.

2) Axial force and shear force.

During earthquake response, forces and moments in the piloti columns changes significantly even after yield mechanism is formed and it is not easy to find appropriate design forces and moments.

3) Rigidity Factor and eccentricity Factor.

In the current design code, the effects of non-uniform rigidity in the vertical direction and eccentricity between centers of mass and rigidity on structural performance are specified. However, the rigidity of members, particularly that of shear walls remarkably changes depending on damage degree. Therefore, these effects continuously change during dynamic response because the members in the piloti story may suffer damages such as cracks of concrete and yielding of steel bars, while those in the upper stories may suffer little damage. In the structural design, such change of the rigidity should be considered.

4) Seismic performance of buildings with story collapse mechanism

Input energy is dissipated mostly in the soft story of piloti buildings, so piloti columns should be ductile enough . This required ductility of columns needs to be further researched. Another subjects to be considered is restoring force characteristics of columns subjected to high axial force. In these columns such as piloti columns, the load carrying capacity remarkable decreases after the maximum capacity even if they fail in flexure (see Fig. 5). If the P-Δ effect is considered, it is easily found that this descending slope after the maximum capacity should be emphasized to be considered because it affects significantly the seismic performance of buildings. On the other hand, buildings with beam yielding mechanism have not such problem because the load carrying capacity continuously increases even after beam yielding due to effects of strain hardening of steel bars, yielding of slab reinforcement, etc. Essentially, piloti columns should be designed to remain before maximum capacity.

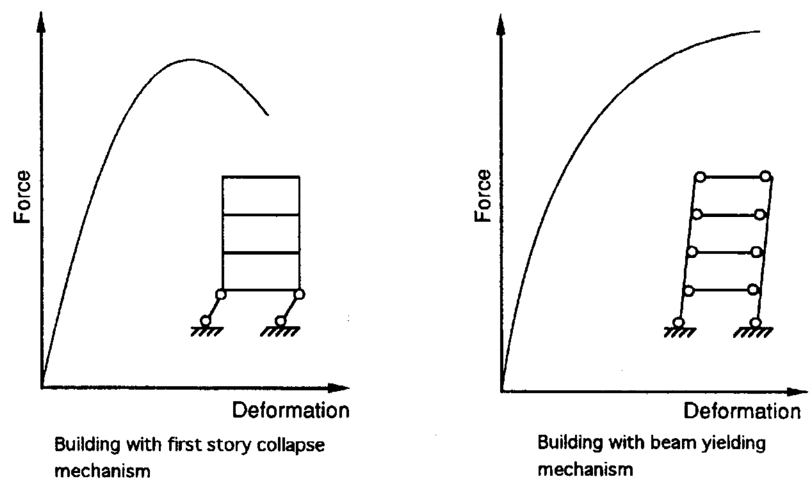

Fig. 5. Idealized restoring force versus deformation relations of the buildings.

CONCLUSIONS

This paper described two types of destructive damage patterns of reinforced concrete buildings which might have caused mass loss of human lives: complete collapse of mid-height story and collapse of piloti of the soft first story. The followings were concluded.

1. The location of the story which suffered severe damage like pancake collapse depends on vibration mode as well as ultimate capacity distribution along the height of the buildings. Insufficient lateral capacity and ductility of columns at certain story caused such collapse.
2. There are many factors which caused collapse of piloti columns. Essentially, piloti columns should be designed to remain before their maximum strength.