Collapse Time of Reinforced Concrete Buildings with Brittle Columns

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Abstract
Buildings with brittle columns are in danger of gravity load collapse and may be hazardous in the event of future earthquakes. To plan for building evacuation, the time required for such buildings to collapse must be known. This study thus uses dynamic analysis to examine the collapse time of reinforced-concrete (RC) buildings designed according to old Japanese codes. Model lateral load vs interstory drift relations are represented based on previous collapse tests of brittle columns. The results reveal that (1) in most cases, the time between shear failure and collapse is very short for individuals to safely evacuate a building, and (2) the collapse time can be long for earthquakes with long durations.

Keywords: reinforced concrete; collapse time; old code; shear failure

1. Introduction
During previous devastating earthquakes, many reinforced-concrete (RC) buildings collapsed because of the gravity load collapse of brittle columns. Most buildings that suffered such collapse were designed according to old seismic codes. However, many buildings with brittle columns still exist and may be hazardous in future earthquakes. Thus, to plan for evacuations from such buildings, it is vital that the time required for these buildings to collapse be determined. In this study, we focus on examining the time between shear failure and collapse (i.e., the "shear-collapse time"). Initially, a wide crack may appear at the point of shear failure, and only then can the damage and potential for building failure be recognized. Thus, shear-collapse time defines the period of time that people have to safely evacuate such buildings. In the past, collapse time was discussed based on the results of pseudo dynamic tests (Nakamura et al., 2012). However, no comprehensive study of collapse time currently exists. Furthermore, we also examine the time between the appearance of minor cracks and collapse (i.e., the "crack-collapse time") to determine the residual time to collapse from the moment oscillations in the building are first perceived. This study uses dynamic analysis to investigate the collapse times of RC buildings. As part of the analysis, the column hysteresis is derived by examining the strength deterioration following shear failure and the subsequent axial collapse commonly associated with brittle columns. Thus, model lateral load vs interstory drift relations are represented based on previous collapse tests (Takaine et al., 2003). The effect of different types of columns, earthquake magnitudes, and earthquake durations is also assessed.

2. Outline of Analysis
2.1 Analytical Model
This study analyzes three-story RC buildings designed according to pre-1971 Japanese codes. In 1971, the regulations for transverse reinforcement ratios were strengthened. Buildings designed on the basis of the pre-1971 code are represented in the equivalent shear building models shown in Fig.1. The height and weight of each story are assumed to be 3600 mm and 753 kN, respectively. The structural properties of the analytical model are summarized in Table 1.

The steps in the analysis are outlined below.
(1) We assumed that the model building consists of a brittle column (clear height $h_0 = 2400$ mm, depth $D = 600$ mm, and $h_0/D = 4$). Fig.2. shows the idealized column. The column is assumed to be twice as large as the tested specimen.
(2) Because the column size for the top two or three stories generally does not vary, the distribution of story strength is assumed to be uniform for all stories. We also assume that in previous earthquakes, only a single story collapsed and the damage to other stories was negligible, and the collapsed story was weaker than the other stories. For the purpose of the analysis, the strength of the first story is reduced to 80% of the other
stories. As a result, the model buildings are expected to collapse at the first story.

(3) The seismic capacity index $I_s$ is computed for each story by using the second-level procedure from the Standard for Seismic Evaluation (Japan Association for Building Disaster Prevention, 2001, and Otani, 2003). The strength of each story is determined such that the value for $I_s$ for the first story is 0.4. In Japan, the value of $I_s$ is commonly used to evaluate the seismic performance of existing RC buildings. It is widely recognized that when $I_s$ is 0.6 or greater, such buildings do not suffer severe damage or collapse even during severe earthquakes. Note that the $I_s$ value for buildings designed under the old code was generally 0.4 (Tamura et al., 2003). As described in the Appendix, the $I_s$ value is calculated based on the product of the strength index $C$ and the deformability index $F$. The index $C$ is defined as the strength of a column divided by the total weight of floors above the column, whereas the index $F$ is determined based on the deformability of a column. The $F$ values of columns that are twice the size of the tested samples are computed to be 1.0. Because of the assumed distribution of story strength, $I_s$ takes on its lowest value at the first story. Hereinafter, $I_s$ for the first story is considered to apply to the entire building.

(4) The initial distribution of story stiffness is the same as the distribution of story strength. The initial stiffness of each story is such that the first mode period is 0.22 s, where the period is computed using the conventional equation $T = 0.02h$, where $h$ is the total building height in meters.

![Fig.1. Analytical Model](image)

![Fig.2. Idealized Column](image)

**Table 1. Structural Properties of Analytical Model**

| Story | Weight (kN) | Initial stiffness (kN/m) | Strength $Q_s$ (kN) | $C$ | $F$ | $1/At$ | $I_s$ |
|-------|-------------|-------------------------|---------------------|-----|-----|--------|-------|
| 3     | 753         | 373000                  | 1130                | 1.50| 1.0 | 0.73   | 1.10  |
| 2     | 753         | 373000                  | 1130                | 0.75| 1.0 | 0.87   | 0.65  |
| 1     | 753         | 299000                  | 900                 | 0.40| 1.0 | 1.0    | 0.40  |

### 2.2 Column Test

Four half-scale models S1, S2, S3, and FS1 were tested to analyze the collapse behavior of old RC columns (Takaine et al., 2003). The longitudinal bar ratios ($p_l$), defined as the total main reinforcement areas divided by the column section, are 2.65% for columns S1, S2, and S3, and 1.69% for column FS1. The transverse bar ratios ($p_t$) are 0.21% for columns S1 and FS1, 0.14% for column S2, and 0.11% for column S3. The models were loaded statically in the lateral direction under constant axial load (axial stress ratio of 0.2). The tests continued until the models were unable to sustain their axial load.

For example, in Fig.3., the relationship between lateral load and interstory drift for model S2 is shown by the solid line. The interstory drift angle is translated from the drift angle by applying the geometric shape shown in Fig.2. The column failed in shear and lost its axial load carrying capacity or collapsed when the interstory drift reached 3.6%. For the purposes of the study, the term "collapse drift" is defined as the maximum interstory drift preceding the collapse.

Fig.4 shows the damage at specific points for model S2. Fig.4.(a) shows the column at its maximum lateral load (see Fig.3). No large cracks appear. Fig.4.(b) shows the column immediately after the maximum lateral load. A wide shear crack appears for the first time. Fig.4.(c) shows the column after the collapse.

![Fig.3. Load vs Drift](image)

![Fig.4. Damage Conditions](image)

### 2.3 Hysteresis Model

The relationship between lateral load and interstory drift, idealized from the test results, is used in the dynamic analysis. The framework of the hysteresis model is shown in Fig.3. with a broken line for column S2. The framework is represented by a quadrilinear function, and we consider the deterioration of strength after maximum loading. The point of collapse is assumed to be the point with the observed collapse drift ($\delta_c$). The value of $\delta_c$ is established to be uniform for all stories. For column S2, $\delta_c$ is assumed to be
We assume that RC buildings collapse when the lateral drift reaches the collapse drift.

Fig.5. shows the hysteresis frameworks for columns S1, S2, S3, and FS1. Columns S1, S2, and S3 fail in shear before flexural yielding and lose the axial load-carrying capacity when the lateral loads decrease to about zero, whereas column FS1 fails in shear after flexural yielding and collapses very suddenly without showing clear strength reduction, accompanied by severe concrete crushing at the column end. Although the column FS1 does not have a wide shear crack that can be recognized, it is used as a comparison to the column S2 that has almost the same collapse drift as the column FS1.

The frameworks prior to maximum loading are the same. Loading at the first break point (crack point) $Q_1$ is 33% of the maximum load. Interstory drift at the maximum load $\delta_2$ is assumed to be uniform for all stories at 0.67%. For columns S1, S2, and FS1, the third break point is assumed to be uniform for all stories. Interstory drift at the third break point $\delta_3$ is 1.3%. Loading at the third break point $Q_3$ of columns S1 and S2 is 50% of the maximum load, whereas that of column FS1 is 100% of the maximum load. Column S3 does not have a third break point. The collapse drift $\delta_c$ is different among the four columns, depending on their deformability. The $\delta_c$ for columns S1, S2, S3, and FS1 is 8.9%, 3.6%, 1.3%, and 3.5% respectively. Loading at the collapse point is assumed to be zero for columns S1 and S2, 40% of the maximum load for column S3, and 80% of the maximum load for column FS1. The collapse point is assumed to be the point with the observed collapse drift $\delta_c$. Note that columns S2 and FS1 have almost the same collapse drift and different types of strength deterioration.

![Fig.5. Framework for Hysteresis](image)

The framework of the hysteresis is based on the Takeda slip model (Eto et al., 1977). Fig.6. compares the lateral loading with interstory drift of the test and the results of the analytical model for column S2. In Fig.6., the test result indicates that after shear failure, the orientation after reversal of the load is not toward the preceding maximum deformation point but toward the symmetrical point with respect to the origin of the load reversal point. To represent such hysteresis rules of RC columns with shear failure before flexural yielding (columns S1, S2, and S3), we modify the Takeda slip model to be consistent with the rule of the orientation (see Fig.6., points A to B).

2.4 Dynamic Analysis

We assume that viscous damping is proportional to initial stiffness because, if viscous damping proportional to instantaneous stiffness was used, acceleration (not damping) results in regions of negative instantaneous stiffness. The damping ratio is set at 1%. The numerical integration method was from Newmark's $\beta$ method ($\beta = 0.25$) (Newmark, 1959). Because lateral loads measured in the test (Fig.3.) include the so-called P-Δ effect, this effect is not considered in the analysis.

2.5 Ground Motion

Eight ground motions are used for the analysis (see Table 2.; ELC at the 1940 Imperial Valley earthquake, HAC at the 1968 Tokachi-oki earthquake, JMA at the 1995 Southern Hyogo prefecture earthquake, OGN at the 2004 Mid Niigata prefecture earthquake, TOH, KRY, and SJK at the 2011 off the Pacific coast of Tohoku earthquake, and BCJ-L2). Fig.7. shows the time history of ground accelerations. Note that ELC, HAC, JMA, OGN, TOH, KRY, and SJK represent data recorded during previous severe earthquakes, whereas BCJ-L2 is the earthquake simulated for structural design (The Building Center of Japan, 1992). Table 2. shows the maximum ground velocities $V_{\text{max}}$ from the original level of ground motions. Note that $V_{\text{max}}$ is calculated as the maximum response velocity for an elastic single-degree-of-freedom system with a natural period of 10 s and a damping ratio of 0.707% (Evaluation Committee of High-rise Buildings, 1986). Furthermore, "duration" is defined as the time interval between 5% and 95% of the final energy. This is represented by the integral of the square of the ground acceleration (Trifunac et al., 1975). According to the durations, the strong motions of TOH, KRY, SJK, and BCJ-L2 lasted longer than that of the other four earthquakes.

Upon conducting the analyses, the level of ground motion is adjusted based on the maximum ground velocity $V_{\text{max}}$. In Japan, such normalization based on $V_{\text{max}}$ is commonly used to evaluate the seismic intensity of earthquake motions in buildings. In addition, in Japanese seismic design, ground-motion levels adjusted to a maximum velocity of 50 cm/s are often used to represent severe earthquakes. Fig.8.
shows the spectrum of acceleration for $V_{\text{max}}$ of 50 cm/s. The natural period of the model building for the first mode ($T$) is also shown in the figure.

Table 2. Ground Motions (original level)

| Name | Year, Earthquake | $V_{\text{max}}$ (cm/s) | Duration (s) |
|------|------------------|--------------------------|--------------|
| ELC  | 1940, Imperial Valley | 33.6 | 24.4 |
| HAC  | 1968, Tokachi-oki | 33.9 | 22.6 |
| JMA  | 1995, Southern Hyogo prefecture | 82.6 | 8.3 |
| OGN  | 2004, Mid Niigata prefecture | 64.8 | 14.5 |
| TOH  | | 41.6 | 118.7 |
| KRY  | 2011, off the Pacific coast of Tohoku | 47.4 | 87.2 |
| SJK  | | 19.8 | 81.5 |
| BCJ-L2 | | 53.4 | 65.4 |

3. Collapse Time

3.1 Shear-collapse Time

As stated above, a wide shear crack appears for the first time immediately after the maximum lateral load. Building damage can be recognized only after the shear failure of a column appears as a wide crack. Therefore, the time between shear failure (immediately after the maximum load) and collapse is the time during which people must evacuate the building. This time interval between shear failure and collapse is called the "shear-collapse time" (see Fig.9.). This study mainly discusses shear-collapse time.

3.2 Crack-collapse Time

It is also important to determine the residual time to collapse after oscillations in a building are first perceived. Although the difficulty lies in deciding the time at which the oscillations are first perceived, we assume this to be the time at which a minor crack first appears in the column (point "Crack" in Fig.3.). Therefore, the time interval between occurrence of crack and collapse is called the "crack-collapse time" (see Fig.9.). Crack-collapse is discussed in section 4.6.

4. Analytical Results

The various ground motions in the four model buildings, referred to as models S1, S2, S3, and FS1 are subjected to a dynamic analysis. The calculations terminate when the response drift equals the collapse drift. The first story is computed to suffer the greatest damage in all cases; therefore, we present the analytical results for the first story as follows.

4.1 Collapse Procedure

As an example of the collapse procedure, Fig.9. shows the time history of interstory drift and the relation between lateral load and interstory drift for model S2. The input ground motion for JMA is adjusted such that the maximum ground velocity is 50 cm/s. In Fig.9., the triangle indicates the first appearance of a minor crack, the square indicates shear failure, and the circle indicates collapse. As shown in the figure, the lateral load suddenly decreases after shear failure occurs and the collapse occurs soon after shear failure with a shear-collapse time of 1.6 s. In addition, the crack-collapse time was 1.9 s. According to the definition mentioned above, the crack-collapse time is longer than the shear-collapse time.

4.2 Crack-collapse Time

It is also important to determine the residual time to collapse after oscillations in a building are first perceived. Although the difficulty lies in deciding the time at which the oscillations are first perceived, we assume this to be the time at which a minor crack first appears in the column (point "Crack" in Fig.3.). Therefore, the time interval between occurrence of crack and collapse is called the "crack-collapse time" (see Fig.9.). Crack-collapse is discussed in section 4.6.
4.2 Effect of Ground Motion Velocity

To examine the effect of ground motion velocity, an analysis was conducted to determine the maximum ground velocity, which ranges from 30 cm/s to 100 cm/s. Fig.10 shows the shear-collapse time as a function of maximum ground velocity for model S2. As shown in the figure, if the area of low ground velocity cannot be represented as a line, no collapse occurs. In most cases, the shear-collapse time is less than 10 s. In particular, the shear-collapse time of JMA is at most 1.6 s. In contrast, the shear-collapse times for TOH are approximately 40 s, with maximum ground velocities ranging from 40 cm/s to 70 cm/s. Furthermore, the shear-collapse time of TOH decreases rapidly if the maximum ground velocity exceeds 70 cm/s. The shear-collapse times for KRY are approximately 60 s with a maximum ground velocity of 65 cm/s or more. Overall, shear-collapse time decreases as ground-motion velocity increases. Similar results were obtained from the other building models.

These results suggest that the evacuation time is very short for RC buildings with brittle columns. Note that the model buildings have columns that serve as earthquake-resisting elements. However, real buildings have other components, such as walls that might serve as earthquake-resisting elements and thus increase shear-collapse time.

As stated above, a great difference appears in shear-collapse time for TOH depending on the magnitude of the ground motion velocity. An example can be seen by considering two ground-motion velocities. Fig.11(a) shows the time histories of ground acceleration and interstory drift for model S2 and TOH with a maximum ground velocity of 50 cm/s. Fig.11(b) shows the same for TOH with a maximum ground velocity of 75 cm/s. As seen in the figure, TOH has a long duration time and multiple points of large acceleration. In Fig.11(a), shear failure occurs during the first strong ground motion and collapse occurs during the second ground motion. As a result, the shear-collapse time is rather long (41.3 s). In contrast, Fig.11(b) shows that both shear failure and collapse occur during the first strong motion, resulting in a very short shear-collapse time (3.5 s). These results indicate that for an earthquake with a long duration time and multiple points of acceleration, the shear-collapse time can be long.

4.3 Effect of Collapse Drift

To examine the effect of the collapse drift on collapse time, models with the large load-degrading (S1, S2, and S3) are compared.

Fig.12 compares maximum ground velocity to shear-collapse time for models S1, S2, and S3 in BCJ-L2. The figure shows that the smaller the collapse drift of the column (S1 > S2 > S3), the shorter the shear-collapse time. In addition, the lowest maximum ground velocity that induces collapse is 60 cm/s, 40 cm/s, and 35 cm/s for models S1, S2, and S3, respectively. This result indicates that the ground-motion velocity required to induce collapse decreases as the column collapse drift decreases.

Fig.13 compares the shear-collapse time of models S1, S2, and S3 for eight input ground motions. In Fig.13 (a), the ground motion has a maximum velocity of 50 cm/s. Because of the large collapse drift of column S1, model S1 does not collapse in any of the earthquakes. For model S2, KRY and SJK also do not collapse. For TOH, the shear-collapse times of models S2 and S3 are 41.3 s and 3.0 s, respectively. The shear-collapse time of model S3 is shorter than that of model S2. Similar results are obtained for other ground motions. Shear-collapse
times for model S3 are very short in all cases except for SJK (13.8 s) because this model has very brittle columns.

As shown in Fig.13.(b), the ground motion has a maximum velocity of 75 cm/s. As in the example with a maximum ground velocity of 50 cm/s, decreasing the collapse drift of a column results in a shorter shear-collapse time. It can also be noted that model S1 collapses only for HAC, TOH, SJK, and BCJ-L2. As shown in Table 2., earthquakes TOH, SJK, and BCJ-L2 had longer duration times than the other four earthquakes.

Fig.14. shows the time history of interstory drift and lateral load vs interstory drift relations for model S1, BCJ-L2 at a maximum velocity of 75 cm/s. As shown in the figure, the interstory drift increases gradually after shear failure occurs, and collapse occurs after many large cyclic responses. This likely means that if the duration of ground motion is long, collapse is likely to occur even if a building has columns with a large collapse drift, such as in model S1.

4.4 Effect of Load-degrading
To examine the effect of the load-degrading on collapse time, models with almost the same collapse drift are compared. Fig.15. compares the shear-collapse time of models S2 and FS1 for eight input ground motions. In Fig.15., the ground motion has a maximum velocity of 75 cm/s. According to the figure, the shear-collapse times for model S2 are shorter than those for model FS1, particularly for TOH and KRY. In Fig.17., the values given in parentheses are the differences (in seconds) between the shear-collapse times of model S2 and those of model FS1 for TOH and KRY. This result indicates that the shear-collapse time decreases as the load-degrading increases, even if the columns have the same collapse drift.

4.5 Effect of Ground Motion Duration
The duration of ground motion is summarized in Table 2. Fig.16. compares shear-collapse time as a function of ground-motion duration for four cases: (a) model S1 with a maximum ground velocity of $V_{\text{max}} = 75$ cm/s; (b) model S2 with $V_{\text{max}} = 50$ cm/s; (c) model S3 with $V_{\text{max}} = 50$ cm/s; and (d) model FS1 with $V_{\text{max}} = 75$ cm/s. According to Figs.16.(a), (b), and (d), shear-collapse time tends to increase with an increase in duration.
the duration of ground motion. In contrast, Fig.16.(c) shows that irrespective of the duration, shear-collapse times are very short. This is because model S3 includes very brittle columns. In summary, shear-collapse time increases as the duration of ground motion increases, except for columns with very low collapse drift.

4.6 Comparison of Crack-collapse Time and Shear-collapse Time

4.6.1 Effect of Ground Motion Velocity

Fig.17. shows the crack-collapse time as a function of maximum ground velocity for model S2. As with the case of the shear-collapse time (Fig.10.), the crack-collapse time decreases as ground-motion velocity increases. The longer crack-collapse times range from 40 s to 90 s. Overall, the crack-collapse time is longer than the shear-collapse time, which we discuss in the next subsection.

As an example, the time history of ground acceleration and interstory drift for model S3 and KRY with a maximum ground velocity of 50 cm/s is shown in Fig.19. A first minor crack appears at 48 s when the ground acceleration is small. Next, shear failure occurs at 93 s and collapse occurs soon after, when the ground acceleration is large. Thus, the crack-collapse time and shear-collapse time are 46.6 s and 0.7 s, respectively. They are significantly different due to the long duration of the input motion.

These results indicate that if a person in a building with brittle columns starts evacuation soon after having recognized building oscillations, the time available for evacuation may be longer for input motions with long duration.

![Figure 17. Maximum Ground Velocity vs Crack-collapse Time](image)

**Fig.17. Maximum Ground Velocity vs Crack-collapse Time (Model S2)**

![Figure 18. Comparison of Crack-collapse Time and Shear-collapse Time](image)

**Fig.18. Comparison of Crack-collapse Time and Shear-collapse Time**

![Figure 19. Time History of Ground Acceleration and Drift](image)

**Fig.19. Time History of Ground Acceleration and Drift (Model S3, KRY at 50 cm/s)**

4.6.2 Effect of Column Types and Ground Motion Duration

Fig.18. compares the crack-collapse times and shear-collapse times for four cases: (a) model S1 with a maximum ground velocity of \( V_{\text{max}} = 75 \text{ cm/s} \); (b) model S2 with \( V_{\text{max}} = 75 \text{ cm/s} \); (c) model S3 with \( V_{\text{max}} = 50 \text{ cm/s} \); and (d) model FS1 with \( V_{\text{max}} = 75 \text{ cm/s} \). As stated before, the crack-collapse time is longer than the shear-collapse time.

It can be noted that the crack-collapse times for TOH and SJK in Fig.18.(a), for TOH, KRY, and SJK in Fig.18.(b) and 18.(c), and for TOH and KRY in Fig.18.(d) are much longer than the shear-collapse times. In Fig.18., the values given in parentheses are the differences (in seconds) between the crack-collapse times and the shear-collapse times for these cases. According to these results, crack-collapse time is approximately 16 to 80 s longer than shear-collapse time. All cases have input motions of long duration (see Table 2.). Note that the crack-collapse times of model S3 for such input motions are also rather long (18.8 s or longer) as opposed to the shear-collapse times.
5. Conclusions

In this study, we use dynamic analysis to study the collapse time of three-story RC buildings with brittle columns. Different column types, ground-motion velocities, and duration of ground motion are factored into the analysis. The major findings of the study are as follows:

(1) Shear-collapse time decreases as ground-motion velocity increases and collapse drift decreases.

(2) Shear-collapse time decreases as the load-degrading increases, even if the columns have the same collapse drift.

(3) Collapse occurs within 10 s of shear failure for earthquakes with short durations.

(4) Shear-collapse times can be long in earthquakes with long durations but depend on ground-motion velocity. Shear-collapse time tends to increase as the duration of ground motion increases. In this study, the longer shear-collapse times range from 40 s to 60 s.

(5) If the duration of ground motion is long, collapse is likely to occur even if a building has columns with large collapse drift.

(6) Crack-collapse time is much longer than shear-collapse time for input motions with long duration, irrespective of column types. If a person in a building starts evacuation soon after having recognized the building oscillations, the time available for evacuation may be longer.

Based on these findings, we conclude that the time interval between shear failure and collapse is very short for individuals to safely evacuate an RC building with brittle columns. Therefore, urgent measures are needed to strengthen such buildings against collapse in future earthquakes.

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Appendix

The seismic capacity index $I_s$ is given as follows (Japan Association for Building Disaster Prevention, 2001):

$$I_s = E_0 \cdot S_D \cdot T,$$  (1)

where $S_D$ is the configuration index (assumed to be 1.0 for this study), $T$ is the time index, (assumed to be 1.0 for this study), and $E_0$ is determined as follows:

$$E_0 = (1 / A_i) \cdot C \cdot F,$$  (2)

where $A_i$ is the vertical distribution factor of story shear coefficients in Japanese building codes, and $i$ is the story to be studied. The index $C$ is defined as the strength of a column divided by the total weight of the floors above the column, whereas the index $F$ is determined according to the deformability of the column. For the columns in this study, $F$ was calculated to be 1.0.

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