# Progressive collapse performance of irregular buildings

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# SUMMARY

In this study, the progressive collapse-resisting capacities of tilted or twisted buildings were evaluated by nonlinear static and dynamic analyses. For analysis models, 30-storey tilted buildings with braced cores and 30-storey twisted buildings with reinforced concrete cores were designed, and their performances were compared with those of the regular buildings. According to the analysis results, the progressive collapse potential of the tilted structures varied significantly, depending on the location of the removed column. It was also observed in the tilted structures that the plastic hinges formed not only in the bays from which a column was removed, but also in the nearby bays. Similar results were observed in the analysis of the twisted structures. The progressive collapse potentials of the tilted structures, the twisted structures considered in this study had progressive collapse potentials not very large compared with those of the corresponding regular structures, mainly because more structural elements were involved in resisting progressive collapse when a structural member was eliminated. Copyright © 2009 John Wiley & Sons, Ltd.

# 1. INTRODUCTION

Contemporary tall buildings are characterized by unique and irregular shapes that can be understood as a reaction to boxed forms of modern architecture. Buildings with irregular forms may be advantageous in reducing wind load effects and building responses. The aerodynamic consideration has led to twisting, tapering or other building forms with discontinuities and multi-planar facades. The Puerta de Europa twin towers in Madrid and the CCTV headquarters building in Beijing are typical examples of tilted buildings. Twisted forms in building structures can be found in many cases, such as the Turning Torso in Malmo, Sweden, and the proposed Chicago Spire Project in Chicago. In general, twisted forms are effective in reducing vortex-shedding-induced dynamic response of tall buildings by disturbing vortex shedding (Scott *et al.*, 2007). In terms of static response, however, tilted/twisted forms are not beneficial. Moreover, irregular buildings may have higher vulnerability for seismic load due to unsymmetrical distribution of mass and/or stiffness. Especially in tilted structures, overturning moment can be induced due to P-delta effect, which may lead to collapse of structures during a strong earthquake.

A progressive collapse involves a series of failures that lead to partial or total collapse of a structure. In the *Best Practice for Reducing the Potential for Progressive Collapse in Buildings*, published by the National Institute of Standard and Technology (2006), the potential abnormal load hazards that can trigger progressive collapse are categorized as: aircraft impact, design/construction error, fire, gas explosions, accidental overload, hazardous materials, vehicular collision, bomb explosions, etc. Progressive collapse has been observed in the collapse of the Ronan Point apartment building in London, 1968, the Alfred P. Murrah Federal Building in Oklahoma City, 1995, the Sampoong Department Store in Seoul, 1995, and the World Trade Center towers in New York, 2001. As these hazards have a low probability of occurrence, they are either not considered in structural design or addressed indirectly by passive protective measures. Most of them have characteristics of acting over a relatively short period of time and result in dynamic responses. Providing sufficient continuity, redundancy and/

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or ductility in the structural members of a building is key to mitigating progressive collapse. To prevent the progressive collapse, the Eurocode1 (2002) presented a design standard for selecting plan types and recommended that buildings should be integrated. In the USA, the American Concrete Institute 318 (2002). 'Building code requirements for structural concrete (ACI 318-02) and commentary (ACI 318R-02).' American Concrete Institute, Farmington Hills, MI, requires structural integrity (e.g., continuity insurance of reinforcing bars) so that partial damage by abnormal load does not result in total collapse. The ASCE7-05 (2005) recommended design method and load combination as well as structural integrity. The General Service Administration (GSA) presented a practical guideline for design to reduce collapse potential of federal buildings (GSA, 2003), and the Department of Defence (DoD) also presented a guideline for the new and existing DoD buildings. Unified Facilities Criteria (UFC)-DoD (2005). 'Design of buildings to resist progressive collapse.' Dept. of Defense, Washington, D.C. Recently, a lot of studies have been conducted on progressive collapse of building structures. Sasani and Kropelnicki (2008) investigated progressive collapse potential of an Reinforced Concrete (RC) structure. Kim and Park (2008) and Kim and Kim (2009) assessed progressive collapse-resisting capacity of steel moment frames. Khandelwal and El-Tawil (2007) applied a macro-analysis model to investigate the resistance of seismically designed steel braced frames to progressive collapse. Kim et al. (2009) developed an integrated analysis design system for two-dimensional moment frame structures. Khandelwal and El-Tawil (2007) and Kim and An (2009) investigated the progressive collapse potential of steel moment frames considering catenary action of beams. Most of the previous research regarding progressive collapse of buildings, however, has focused on regular buildings.

This paper intended to investigate progressive collapse potential of tilted buildings and twisted buildings subjected to loss of a column. To this end, 30-storey tilted buildings and twisted buildings were designed, and nonlinear static and dynamic analyses were carried out with one of their first-storey columns removed. The analysis results were compared with those of corresponding regular buildings.

#### 2. DESIGN AND ANALYSIS MODELLING OF EXAMPLE STRUCTURES

#### 2.1. Tilted structures

To evaluate progressive collapse potential of tilted structures, four-bay six-storey regular steel moment frames were designed and were simply tilted by 5°, 10° and 15° without change of structural members. Design dead and live loads of 4.5 and 2.5 kN/m<sup>2</sup> were used, respectively, for member design. The design seismic load was evaluated using the seismic coefficients  $S_{DS} = 0.53$  g and  $S_{D1} = 0.34$  g, and the response modification factor of 5.0 in the ICC (2006) format. The beams and columns were designed using SS400 ( $F_y = 23.5$  kN/cm<sup>2</sup>) and SM490 ( $F_y = 32.4$  kN/cm<sup>2</sup>) steel, respectively. Figure 1 shows the six-storey analysis model structure with a uniform storey height of 3.5 m tilted by 15° from the vertical axis.

The 30-storey structures with steel moment frame—core-bracing interaction systems with a tilt angle of  $0^{\circ}$ ,  $10^{\circ}$  and  $15^{\circ}$ —were designed to investigate the progressive collapse potential of tilted



Figure 1. Elevation of six-storey tilted structures ( $\Theta = 15^{\circ}$ ).

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buildings. Figures 2 and 3 depict the configurations of the 30-storey regular (not tilted) and the tilted structures, respectively. Even though the exterior columns of the tilted buildings are sloped, the interior columns and braced core remain vertical. Each of the tilted structures was designed separately, considering the overturning moment caused by the tilted configuration. The structures were designed using the gravity and the seismic loads mentioned above, plus the wind load with a design wind speed of 30 m/s. The response modification factor corresponding to a steel dual system, which is 5.0, was used for seismic design. The buildings have a uniform storey height of 3.5 m. Columns located up to the 15th storey are made of SM520 ( $F_y = 35.3 \text{ kN/cm}^2$ ), and those located from the 16th to the 30th storey are made of SM490 ( $F_y = 32.4 \text{ kN/cm}^2$ ) steel. Girders of the regular structure are designed with SS400 ( $F_y = 23.5 \text{ kN/cm}^2$ ) steel, and those of the tilted structures are made of SM490 steel. All braces are made of SS400 steel.



Figure 2. Thirty-storey regular structure with braced core.



Figure 3. Thirty-storey tilted structure ( $\Theta = 15^{\circ}$ ).

# 2.2. Twisted structures

The 30-storey twisted buildings with reinforced concrete square core walls have a  $30 \times 30$ -m square plan and a uniform storey height of 3.5 m. Each floor of the regular building was rotated by 1.5° and 3° to produce 45° and 90° twisted buildings at the top floors, respectively. Figures 4 and 5 illustrate



Figure 4. Thirty-storey regular building with RC circular core: (a) plan shape; (b) side view.



Figure 5. Thirty-storey twisted buildings: (a) twisting angle =  $45^{\circ}$ ; (b) twisting angle =  $90^{\circ}$ .

Twisting angle	Thickness (cm)	Concrete strength (kN/m <sup>2</sup> )
0°	40	3.0
45°	50	3.0
90°	60	3.5

Table 1. Core thickness and concrete strength of twisted buildings at the first storey.



Figure 6. Imposed gravity load for progressive collapse analysis: (a) static analysis; (b) dynamic analysis.

the configurations of the 30-storey regular and twisted model structures, respectively. The RC core walls were designed to have a circular shape for ease of locating floor beams. All columns and girders were made of SM490 and SS400 steel, respectively. The response modification factor of 5.0 was used to evaluate seismic design load. Table 1 shows the thickness and design strength of core walls at the first storey, where it can be observed that as the twisting angle increases, the required thickness and strength of the core walls also increase. It was also observed that the sizes of the first-storey columns and connected beams showed the same trends.

#### 3. ANALYSIS METHODS FOR PROGRESSIVE COLLAPSE

To investigate progressive collapse potential of the model structures, nonlinear static pushdown analysis was carried out using the programme code SAP 2000 (2004) with one of the first-storey columns removed. The procedure accounts for nonlinear effects without sophisticated hysteretic material modelling, and is useful in determining elastic and failure limits of the structure. The GSA (2003) guidelines proposed the amplification factor of 2 for the static analysis to account for dynamic redistribution of forces. The proposed load combination of the GSA (2003) for static analysis is 2(dead load + 0.25 × live load), which was selected for pushdown analysis in this paper. This amplified load was applied only in the spans from which a column was removed, while unamplified load was applied in the other spans as illustrated in Figure 6(a). In this study, pushdown analysis was applied by gradually increasing the vertical displacement in the location of the removed column to investigate the resistance of the structure against such deformation. At every step during the pushdown analysis, i.e. at each level of the vertical displacement, the amount of equivalent load corresponding to the displacement level was determined. The ratio of the applied load and the GSA-specified load of 2(dead load + 0.25 × Live Load) is referred to as the 'load factor'. The original loading pattern remained unchanged at every step.

Nonlinear dynamic analyses of the model structures were carried out, with the same first-storey column suddenly removed. For nonlinear dynamic analysis, the load dead load +  $0.25 \times$  live load



Figure 7. Time history for nonlinear dynamic analysis.



Figure 8. Force-displacement relationship of structural members for nonlinear analysis: (a) bending members; (b) braces.

was uniformly applied in the entire spans as shown in Figure 6(b). In order to carry out dynamic analysis, the member forces of a column that was to be removed to initiate progressive collapse were computed before it was removed. Then, the column was replaced by point loads equivalent of its member forces. In order to simulate the phenomenon that the column was abruptly removed, the member forces were suddenly removed as shown in Figure 7, while the applied gravity load remained unchanged. Figure 8 shows the force-displacement relationship of structural members for nonlinear analysis, where the hysteretic behaviours of plastic hinges were defined based on FEMA-356 (2000). The parameters a and b shown in Figure 8(a) were determined to be 6 and 8 times of the yield rotation, respectively, based on tables 6–7 of FEMA-356. The residual strength parameter, c, was assumed to be 0.2.

#### 4. ANALYSIS RESULTS

#### 4.1. Tilted structures

Figure 9 shows the pushdown curves of the six-storey structures with various tilting angles when the centre column was removed. The nonlinear static analyses were carried out using the general purpose analysis programme SAP 2000. The load factor of 2.0 implies that the GSA-specified gravity load of 2.0(dead load +  $0.25 \times$  live load) is reached. It can be observed that as the tilting angle increases, the maximum strength and ductility decrease. It should be pointed out, however, that the results would be different if the tilted structures were redesigned considering the P-delta effect caused by tilting. Figure 10 shows the plastic hinge formation of the six-storey untilted structure and the  $15^{\circ}$  tilted structure, by pushdown analysis with a corner column removed. In the regular structure,



Figure 9. Pushdown curves of the six-storey structures with the centre column removed.



Figure 10. Plastic hinge formation of the six-storey structure: (a) regular structure; (b)  $\Theta = 15^{\circ}$ .



Figure 11. Column numbering of tilted buildings: (a) regular structure; (b) tilted ( $\Theta = 10^{\circ}$ ); (c) tilted ( $\Theta = 15^{\circ}$ ).

plastic hinges formed only in the span from which a column was removed. However, in the tilted structure, plastic hinges formed in every span.

Figure 11 shows the column numbering of the 30-storey tilted structures, and Figure 12 shows their pushdown curves. It can be observed in the pushdown curves that when the number 1 column (which



Figure 12. Pushdown analysis results of tilted buildings: (a)  $\Theta = 0^{\circ}$ ; (b)  $\Theta = 10^{\circ}$ ; (c)  $\Theta = 15^{\circ}$ .

is the corner column in the right-hand side) is removed, the progressive collapse-resisting capacity decreases as the tilting angle increases because additional compression is imposed on that column due to tilting. The progressive collapse potential is the maximum when the number 6 column (the left-hand side corner column) is removed on which tension is imposed due to overturning moment. It can be seen that, generally, the overall strength did not decrease due to tilting of the structure since the tilted structures were designed considering the additional member forces caused by the P-delta effect. However, the corner column located in the tilted direction needs to be strengthened or protected from possible damage to prevent progressive collapse of the whole structure. When the number 4–6 columns of the tilted structures were removed, the maximum strength became even greater than those of the regular (untilted) structure.

Figure 13 shows the distribution of gravity load on each column of the structures, where it can be observed that, compared with the untilted structure, the share of gravity load of the number 1 corner column (located in the tilted side) increases and that of the number 6 corner column decreases as the tilting angle increases. It is interesting to note that the number 6 columns of the tilted structures are subjected to tension since the tensile force resulting from overturning moment is larger than the compression from gravity load. Figure 14 depicts the plastic hinge formation of the untilted and tilted model structures when a corner column (number 1 column) was removed from each structure. In the regular structure, plastic hinges formed only at the girders located in the exterior bay from which a corner column was removed. In the tilted structure, however, the plastic hinges formed at girders and columns more widely spread in many bays.



Figure 13. Column gravity load distribution in the tilted structures: (a)  $\Theta = 0^{\circ}$ ; (b)  $\Theta = 10^{\circ}$ ; (c)  $\Theta = 15^{\circ}$ .

#### 4.2. Twisted structures

Figures 15 and 16 show the pushdown curves of the 30-storey regular and twisted buildings with a square plan shape when one of their exterior columns were removed. It was observed that when the second column from the corner was removed, the maximum load factors ranged between 1.2 and 2.3. When the corner column was removed, the maximum load factors ranged from approximately 0.4 to 0.9, which are smaller than the specified load factor of 1.0. No distinct trend could be observed depending on the twist angle, since twisted buildings were designed with larger members to compensate for the negative effect caused by the twisting of plans.

Figures 17 and 18 depict the nonlinear, dynamic time-history analysis results of the vertical displacements at the joints from which a middle and a corner column were suddenly removed, respectively. The horizontal broken lines denote the vertical displacements when the specified load was imposed statically. It can be observed that when a middle column was suddenly removed, the vertical displacements remained stable regardless of plan twisting angle. This corresponds well with the pushdown analysis results with maximum load factors greater than 1.0. However, when a corner column was removed, the vertical displacements increased unbounded except in the 45° twisted structure in which the maximum load factor obtained from the pushdown analysis reached almost 0.9. Even in this case, the vertical displacement increased more than twice the value obtained when a middle column was removed. In comparison with the pushdown analysis results, it can be concluded that a structure with the maximum strength corresponding to the load factor higher than approximately 0.8 may remain stable after a column was suddenly removed, whereas a structure with the maximum load factor less than approximately 0.5 may progressively collapse due to loss of a column.



Figure 14. Plastic hinge formation in the model structures with a corner column removed: (a)  $\Theta = 0^{\circ}$ ; (b)  $\Theta = 15^{\circ}$ .

Figure 19 depicts the plastic hinge formation of the model structures obtained by dynamic analyses with a corner column suddenly removed. It was observed that in the regular structure, without twisting, plastic hinges formed only in beams located in the corner bay. However, in the 90° twisted structure, plastic hinges formed even in some columns as well as in beams, and more structural members participated in resisting progressive collapse.

Figures 20 and 21 plot the horizontal displacement and rotation along the vertical axis at each top corner of the model structures when a corner column was suddenly removed. The numbering of the corners and the location of the removed column are shown in Figure 4(a). When a corner column was removed, only a small amount of lateral displacement and rotation occurred in the regular structure, whereas large lateral displacement and rotation occurred in the twisted structure. Especially, the rotation at the corner from which the first-storey column was removed turned out to be larger than those of the other corners. The displacement and rotation generally increased as the building height increased.

# 5. CONCLUSIONS

In this study, the progressive collapse-resisting capacities of tilted and twisted buildings were evaluated by nonlinear static and dynamic analyses. For analysis models, 30-storey tilted buildings with braced cores and 30-storey twisted buildings with reinforced concrete cores were prepared. The tilted structures were designed with steel braced cores, and the twisted buildings were designed with square RC cores. The progressive collapse of the model structures was initiated by removing one of the



Figure 15. Vertical force-displacement relationship of a 30-storey structure with an RC core with the second column removed: (a) regular structure; (b) twisted structure ( $\Theta = 45^{\circ}$ ) (c) twisted structure ( $\Theta = 90^{\circ}$ ).

first-storey columns. The performances of the irregular structures were compared with those of the regular buildings designed without tilting or twisting.

According to the analysis results, the progressive collapse potential of the tilted structures varied significantly, depending on the location of the removed column. Especially, the corner column located in the tilting direction needs to be strengthened or protected from possible damage to prevent progressive collapse of the whole structure. It was also observed in the tilted structures that the plastic hinges formed not only in the bays from which a column was removed, but also in the nearby bays. Similar phenomenon was also observed in the twisted structures. However, the overall progressive collapse potentials of the tilted or twisted structures considered in this study were not particularly higher than those of the corresponding regular structures. This was partly because the tilted or twisted structures were designed with larger structural members considering their irregularities. Another reason seems to be that, compared with regular structures, more structural elements were involved in resisting progressive collapse when a structural member was eliminated.

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Figure 16. Vertical force-displacement relationship of a 30-storey structure with an RC core with the corner column removed: (a) regular structure; (b) twisted structure ( $\Theta = 45^{\circ}$ ) (c) twisted structure ( $\Theta = 90^{\circ}$ ).



Figure 17. Vertical displacement time histories of a 30-storey structure with an RC core with the second column removed: (a) regular structure; (b) twisted structure ( $\Theta = 45^{\circ}$ ) (c) twisted structure ( $\Theta = 90^{\circ}$ ).



Figure 18. Vertical force-displacement relationship of a 30-storey structure with an RC core with the corner column removed: (a) regular structure; (b) twisted structure ( $\Theta = 45^{\circ}$ ) (c) twisted structure ( $\Theta = 90^{\circ}$ ).



Figure 19. Plastic hinge formation in the model structure obtained by dynamic analysis with a corner column suddenly removed: (a) regular structure; (b) twisted structure ( $\Theta = 90^{\circ}$ ).

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Figure 20. Displacement of regular structure with RC core when a corner column is removed: (a) lateral displacement; (b) rotation.



Figure 21. Displacement of the 90° twisted structure with RC core when a corner column is removed: (a) lateral displacement; (b) rotation.

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