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Gravity Load Redistribution and Progressive Collapse Resistance of 20-Story Reinforced Concrete Structure following Loss of Interior Column

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Progressive collapse, which is defined as the spread of an initial local failure from element to element, eventually resulting in the collapse of an entire structure or a disproportionately large part of it (ASCE/SEI 7[7]), has drawn attention since the Ronan Point Apartment Building collapsed in 1968. Various procedures are proposed in the literature to simulate the effects of this phenomenon, based on different specific assumptions, such as the independence of the procedure with respect to the cause of the initial failure or the sequence in which the loads are applied. It is significant that the authors presented a full image of the dynamic redistribution of loads following the explosion of an interior ground-floor column in a 20-story reinforced concrete (RC) framed structure. Some findings are interesting to the discussers and are worthy of further discussion.

Dynamic structural response

In the paper, the second-floor strain gauge detected the initial compression strain wave 1 millisecond after the explosion in Interior Column C3 on the ground floor. Then, it detected a tensile strain of 238 $\mu\varepsilon$ 5 milliseconds later, which was 82% of the permanent change in the strain. The measured vertical displacement at that moment, however, was 0.89 mm (0.035 in.)—only 10% of the maximum second-floor displacement. The first compression strain should be caused by the explosion. If the predrilled holes where the explosives were inserted were at the midheight of the ground column, the speed of the axial stress wave (expressed as $\sqrt{E_c/\rho}$, where E_c is the modulus of elasticity of the concrete; and ρ is the density of the concrete) should be approximately 4000 m/s (13,123 ft/s). It takes nearly 1 millisecond for the wave to reach the second-floor column. The second tensile strain wave recorded in the second-floor column should be caused by the loss of the ground column because it induced a sudden downward displacement and the elongation of the column. It likely took a couple of milliseconds for the ground column to completely fail after the explosion. It is still not clear, however, why the vertical displacement response was much slower than that of the stress waves. The speed of axial wave propagation would be higher than that of the flexural wave, but not that much.

One potential reason could be due to the restrained effects from the superstructure. Unlike the axial stress wave, the vertical downward displacement of the column is an action of the frame restrained by the joined beams and the neighboring subframes. Because the building is a 20-story RC frame, it would take longer to redistribute loading among the structural members within the frame after the local damage occurs. The final displacement occurred after the full redistribution of loading, achieving the final balance of the structure. The lag stress phenomenon in the seventh-floor column could also be due to the force transfer and redistribution.

Also, because of the very short duration impulse-type loading caused by the blast, the instantaneous resistance offered by RC is somewhat different. The stiffness and strength of both the steel reinforcement and concrete are likely to increase with a higher rate of loading experienced under blast conditions. This obviously increases the strength of RC members, which translates into higher resistance. The high rate of loading expected during blasts, however, is also likely to significantly reduce the deformation capacity and fracture energy of RC. This translates into a reduction of ductility of RC in blast loading situations.

Static analysis approach

It is not clear whether the dynamic measurement after the initial 0.5 seconds was recorded or not. Only measurements during the initial 0.5 seconds were reported. It appears, however, that both the displacement and axial strains in the columns tended to be stabilizing. Most likely, 0.5 seconds is sufficient for an RC framed structure capable of redistribution to be finalized, as is also illustrated in the study of the Hotel San Diego.[11] As shown in Fig. 5 and 6, it appears that both the vertical displacement and the axial strain in the columns tended to stabilize 0.1 second after the explosion. This should suggest that the dynamic analysis approach can be replaced by an equivalent static analysis approach, such as the alternate load path method, to evaluate the progressive resistance of the structure.

A dynamic impact factor (DIF) is normally used to account for the dynamic effects within a static design framework by magnifying statically computed design forces through the DIF, which is more than 1.0. A DIF of 2.0, assuming linear structural behavior, is typically used to account for the dynamic effects associated with the sudden placement or removal of loads.[5] The sudden removal of a load-bearing element causes a sudden geometric change in the structure, resulting in a release of the potential energy and rapid variation of internal dynamic forces, including inertia forces. When a local primary collapse mechanism is triggered, the portion of the structure above the removed column undergoes a free fall and collides with the floor slabs below. This hammer effect generates elastic waves that can damage the neighboring floor slabs. The scenario of losing the element would have a dynamic effect on other structural elements and may lead to immediate damage in the vicinity of that element. This dynamic effect is not clearly illustrated in the measurements, such as the vertical displacement and axial strain variation, as shown in Fig. 5 and 6. It likely suggests that the commonly used value of 2.0 can potentially be relaxed. As the dynamic multiplier values were found to be influenced by three main ratios (the total plastic rotation ratio, the maximum plastic rotation ratio in the framed beams, and the maximum vertical displacement ratio), a dynamic multiplier of 1.5 would better capture the dynamic effects when a static analysis is performed and will result in more economical designs.[16]

Variation of axial force

It is not clear why the axial forces in Column C3 derived from the numerical analysis, as shown in Fig. 12, are sharply reduced from the start of the dynamic event ($t = 0.0$ seconds) to the state when $t = 0.018$ seconds or the final state. For instance, the axial force in ground-floor Column C3 dropped by 87.4% from its peak value after the column removal, and 85% of the dropping gravity load was redistributed over neighboring columns, such as Columns C2, C4, B3, and D3. Columns C2 and C4 were framed to Column C3 via the framed beam with sections of 290 x 560 mm (11.5 x 22 in.), whereas Columns D3 and B3 were linked to Column C3 via the one-way RC slab. Both the floor slab and the framed beams played an important role in redistributing the gravity load after the explosion with the effective tying links between the beams and slabs. It is likely that slabs would be prone to developing plastic hinges because the one-way RC slabs were much weaker than the framed beams. The redundancy of the vertical load carrying the structural system should also help to redistribute the gravity load among the structural members, thereby enhancing the progressive collapse resistance.

Based on the measured permanent strains of the second- and seventh-floor columns—290 με and 150 με, respectively—the corresponding variation in the axial forces can approximately be calculated by $\varepsilon_c A_c E_c$, where ε_c is the measured axial strain; A_c is the area of the column section; and E_c is the modulus of elasticity of the concrete, estimated at 251,00 MPa (3650 ksi) (2708 and 1401 kN [609 and 315 kips] in the second- and seventh-floor columns, respectively). The axial force change in the second-floor column is slightly lower than 2928 kN (659 kips), the analytic estimation of changing the value of the axial force before and after column removal in the ground floor.

It appears that the nonlinear response of the beams and floor slabs was important to resist the progressive collapse of the structure. No plastic hinges were formed in the beams above the fifth floor. Could the authors indicate where plastic hinges formed in the second floor and in the other floor beams? Did they occur at the midspan of Column C3 or at the supports adjacent to the positions of Columns C2 and C4?

The verification of the accuracy of the finite element (FE) analysis procedure is promising. The response of the structure with additional loads and complete column removal was studied. An additional live load of 1.2 kN/m^2 (25 lb/ft^2) in addition to the partition weight exerted led to the maximum vertical displacement of 22 mm (0.87 in.) at Joint C3 on the second floor.

Finally, some figures from the paper need to be further clarified. It appears from Fig. 2 and 3 that there are mismatches or differences between the ground floor plan and the floors above. The distance from Column C3 to the neighboring columns (longitudinal) is 6710 mm (264.2 in.) in the ground floor (Fig. 2), whereas this value changes to 6700 mm (263.8 in.) in Fig. 3. What are the sections of the columns with dimensions of 660 x 660 mm (26 x 26 in.) (Fig. 3) and 610 x 610 mm (24 x 24 in.) (Fig. 2)?

REFERENCES

16. Ruth, P.; Marchand, K. A.; and Williamson, E. B., "Static Equivalency in Progressive Collapse Alternate Path Analysis: Reducing Conservatism while Retaining Structural Integrity," *Journal of Performance of Constructed Facilities*, ASCE, V. 20, No. 4, 2006, pp. 349-364.

AUTHORS' CLOSURE

The authors would like to thank the discussers for their interest in the paper and have provided clarification to the comments made.

Dynamic structural response

As stated in the paper, "the second-floor strain gauge recorded a large compressive strain just one millisecond after the explosion, which was due to the initial axial compressive stress wave caused by the explosion. This phenomenon, with a smaller strain amplitude, was also recorded by the seventh-floor strain gauge." Because the compressive strains were recorded at almost the same time in the first and seventh floors, the use of the simple expression for the speed of axial wave propagation, as suggested by the discussers, is not quite justified.

Regarding the tensile strain wave, the discussers state, "It is still not clear, however, why the vertical displacement response was much slower than that of the stress waves. The speed of axial wave propagation would be higher than that of the flexural wave, but not that much." In the first sentence of this statement, it is not clear to the authors why the discussers compare the speed of the vertical displacement response with that of the stress waves. The vertical displacement response is due to the sum of the axial tensile wave and the flexural wave. In the second sentence, the discussers argue that the speed of the axial wave should not be that much larger than that of the flexural wave. Because this speed difference was in fact observed experimentally and verified analytically in this paper and previous publications,[10,11] it is not clear to the authors what this expectation of the discussers is based on. In any case, possible justifications for this difference are presented in the following two paragraphs in the discussion.

The discussers' first justification, described as "an action of the frame restrained by the joined beams and the neighboring subframes," is presented extensively and in detail in the authors' previous publications[10,11] as the "Vierendeel frame action" and is identified as a major load-resisting mechanism.

Furthermore, it seems that the discussers state that one reason for the difference between the speeds of the axial and flexural wave propagation is the size of the structure. The time to the peak displacement response, however, is not necessarily a function of how large the structure is but depends on other more relevant mechanical measures used in structural engineering, namely the (vertical) stiffness and the mass of the damaged structure and, to some extent, the system damping, which all determine the period of vibration. Note that based on the experimental results, the peak displacement response in this 20-story structure occurred approximately 0.027 seconds after the explosion. During previous experiments in the six-story Hotel San Diego, CA,[11] and the 10-story dormitory in Little Rock, AR,[11] the peak displacements occurred approximately 0.079 seconds and 0.12 seconds after the explosion, respectively, both being larger than the time required for the peak displacement in the 20-story structure discussed in the paper.

In the last paragraph of the dynamic structural response section, the discussers argue that "also, because of the very short duration impulse-type loading caused by the blast," the structure becomes stronger and stiffer. This argument has several flaws. One is that while the blast occurs in a very short period of time (approximately 1 to 2 milliseconds), the peak displacement response occurs after (and not during) the blast, over a duration that is tens of times longer than the blast duration. Therefore, the loading rate that the undamaged structure will experience depends on the duration up to the peak structural response and not the duration of the blast. The second issue is that if the structure is stiffer, the period of vibration and the time to the peak displacement decrease and not increase, as the discussers seem to suggest. Furthermore, the deformation capacity and any change in its value, as suggested by the discussers, have no relevance to the response of this structure because the deformations are too small to be limited by the deformation capacity of the elements and the structure.

Static analysis approach

The discussers state that because there is no significant vibration after the peak, "this should suggest that the dynamic analysis approach can be replaced by an equivalent static analysis approach." Although an equivalent static analysis can potentially be used to estimate the peak dynamic response of a structure (and not the oscillation after the peak), the discussers' conclusion cannot be made based on the fact that the structural response dies out rapidly. In fact, the response of a system with high damping (as in this structure) is more difficult to predict using an equivalent static analysis because from an analytical point of view, damping force is velocity-dependent, which does not quite lend itself to an approximate estimation based on a static analysis.

The general statements of the second paragraph, which are based on References 5 and 16, while interesting, have no particular relation to the structure and are not within the scope of the paper.

Variation of axial force

To better understand the rapid variation of axial force (obtained both analytically and experimentally through strain measurements) in the columns above the removed column, the authors refer the discussers to previously published papers by the authors.[9-11] Such an explanation is not presented in the paper or in the closure to avoid repetition. The statements that follow the first sentence of the first paragraph of the discussion regarding how the axial load of the exploded column is distributed among the neighboring columns do not seem to be relevant to how rapidly the columns above the removed column lose their axial load.

Regarding the inelastic response of the structure, plastic hinges formed in the second- to fifth-floor beams of Axis C at the face of Column C3.

Finally, the authors would also like to thank the discussers for identifying a discrepancy between Fig. 2 and 3. The span length in the longitudinal direction neighboring the removed column in Fig. 2 is shown to be 6710 mm (264.2 in.), while this value is incorrectly rounded to 6700 mm (263.8 in.) in Fig. 3. Another correction in Fig. 2 is that the column size in SI units is 660 x 660 mm (26 x 26 in.) and not 610 x 610 mm (24 x 24 in.). Note that the values in U.S. customary units (inches and feet) are correctly reported in both figures.

(Editors' note: The PDF of the original paper, available for download at ***www.concrete.org****, has been revised to show the correct figure values.)*

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Tests on Lightweight Concrete Deep Beams

by Keun-Hyeok Yang

Discussion by Andor Windisch

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The author correctly states that "deep beams are disturbed-region members where conventional beam theory does not apply." Nevertheless, the schematic strut-and-tie model (STM) shown in Fig. 6 does not reveal any deep-beam relevant characteristics; the point loads near the supports could occur in a slender beam, too. The most distinguishing feature of a D-region is the position of the horizontal compressive strut C—that is, the inner lever arm jd, which depends on the h/L ratio. In fact, the most typical error in the dimensioning of deep beams is a too-weak flexural reinforcement and/or its poor anchorage at the supports.

To eliminate yielding, the longitudinal reinforcement of the test specimens was far beyond the necessary and practical amount. The amount of the longitudinal reinforcement has quite a substantial impact on the shear strength of reinforced concrete (RC) members. The author strove to have identical longitudinal reinforcement in all the test specimens. Nevertheless, as the yield strength of the 19 mm (0.75 in.) bars was much higher than that of the 22 mm (0.85 in.) bars, the mechanical rate of longitudinal reinforcement of the 1000 mm (39.4 in.) high test specimens was much lower than that of the other bars. Another problem is that the oversized longitudinal reinforcement did not yield at failure. The question arises: which effective "resultant tensile forces" in the bottom nodes were taken into account? How was w_t for Eq. (2) calculated? Please clarify.

The failure of the specimens is specified by the author as "diagonal tensile failure of concrete struts within the shear spans." Nevertheless, the failure planes shown in Fig. 2 run along the inner border of an imaginary direct strut—that is, the strut did not fail at all. Especially typical is the second crack pattern from the top in Fig. 2; the failure did not occur in the neighborhood of the right strut, which showed many cracks, but along the outer side of the left strut, where no load transfer occurred at all. Please clarify.

As the failures were not triggered through the compressive struts, any comparison with Eq. (1) is questionable. Moreover, the angle θ deduced from the assumed jd is questionable, too, as mentioned previously.

Equation (3) proposed by Tan and Cheng[6] is quite strange, as components that resulted from the entire uncracked concrete cross section (last member in Eq. (4)) from yielding