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ANALYSIS OF A SMALL SCALE RC BUILDING SUBJECTED TO SHAKING TABLE TESTS USING APPLIED ELEMENT METHOD

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SUMMARY

Simulation of collapse process of a scaled reinforced concrete (RC) structure is carried out and compared with the results obtained by shaking table experiments. The experiment was performed using eleven storied RC building model subjected to a series of base excitations whose shapes were same but different amplitudes. The numerical simulation is performed with two-dimensional Applied Element Method (AEM). The simulated structural response shows good agreement with experimental results. Due to the limitations of capacity of the shaking table used, the experiment was performed only up to the start of collapse. However, the numerical analysis using the AEM is extended to simulate the detailed collapse behavior under magnified base excitations.

INTRODUCTION

Applied Element Method (AEM) is a newly developed method for structural analysis. The main advantage of the method is that it can follow the structural behavior from elastic range, crack initiation and propagation, separation of structural elements to total collapse in reasonable CPU time with reliable accuracy. The applicability and accuracy of the AEM in various fields were discussed in many publications [1-5]. The organization of the previous and current researches are shown in Table 1. This table shows all meaningful application ranges, which should be covered by the proposed numerical model. In each application range, the numerical results were compared with theoretical and experimental results whenever possible.

Collision and recontact problems [6] are essential in many cases. For example, "What is the effect of collision between falling structural elements and neighboring structures?", "What happens if structures having different dynamic characteristics collide during earthquakes?", "If structures collapse, where and how do they undergo collapse?" and "What is the time of duration of collapse?". These questions can not be answered unless the numerical technique considers the recontact effects. Although, these effects are very important from the viewpoint of safety and/or security of users, only few numerical techniques can deal with these problems. Many analyses of collision problems were performed using the Extended (or Modified) Distinct Element Method (EDEM or MDEM) [6-8], however, as the simulation using the EDEM needs long CPU time and it is less accurate than the proposed method in small deformation range. Other methods like Discontinuous Deformation Analysis, (DDA) [9] were applied mainly to large deformation of rocks. The applicability of the DDA is still

limited to simulation of behavior of rocks or bricks. In addition, the method requires long CPU time and many experiences to perform the simulation and in many cases, algorithm is not stable [9].

This paper shows the simulation results of collapse process of a scaled reinforced concrete (RC) structure compared with the results obtained by shaking table experiments. The experiment was performed using eleven storied RC building model subjected to a series of base excitations whose shapes are same but different amplitudes. The numerical simulation is performed with two-dimensional AEM. The analysis using the AEM is

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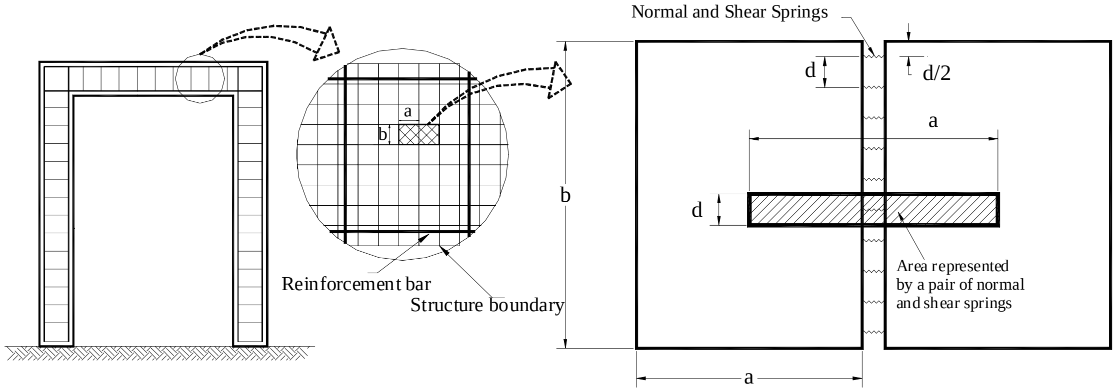
Table 1: Organization of Research Results

		Static		Dynamic	
Geometry	***Material***	***Monotonic***	***Cyclic***	***Monotonic***	***Cyclic***
Small deformation	***Elastic***	***I*** [1]	***III*** [4]	***V*** [5]	***This paper***
	Nonlinear	***II*** [2]			
Large deformation	***Elastic***	***IV*** [3]			
	Nonlinear	***Covered in dynamics***			
Collapse process		***No meaning***			



extended to simulate the detailed collapse behavior under magnified base excitations and realistic results were obtained.

OUTLINE OF AEM

With the AEM, structure is modelled as an assembly of small elements, which are made by dividing the studied structure virtually, as shown in Figure 1 (a). The two elements shown in Figure 1 (b) are assumed to be connected by pairs of normal and shear springs located at contact points, which are distributed around the element edges. Each pair of springs totally represent stresses and deformations of a certain area (hatched area in Figure 1 (b)) of the studied elements. In case of reinforcement, two pairs of springs are used, for concrete and for reinforcement bar. This means that the reinforcement spring and concrete spring have the same strain and the effects of separation between reinforcement bars and surrounding concrete can not be easily considered within an element. However, when we look at the behavior of element collection as a unit, due to the stress conditions, separation between elements occurs because of failure of concrete-springs before the failure of reinforcement-springs and hence, relative displacement between reinforcement bars and surrounding concrete can be taken into account automatically. This is a unique point which continuum equation based models, like FEM, do not have. In the proposed method, reinforcement springs can be set at the exact location of the reinforcement bars in the model. It should be emphasized that effects of stirrups, hoops and concrete cover can be easily considered. Each of the elements has three degrees of freedom in two-dimensional model. These degrees of freedom represent the rigid body motion of the element. Although the element motion is as a rigid body, its internal deformations are represented by the spring deformation around each element. This means that although the element shape doesn't change during analysis, the behavior of assembly of elements is deformable. Introduction of the Poisson's ratio effect is illustrated in details in Ref. [1]. The global stiffness matrix is determined by summing up the stiffness matrices of individual pair of springs around each element. Consequently, the developed stiffness matrix has total effects from all of the pairs of springs according to the stress situation around the element. This technique can be used both in load control and displacement control cases.

OUTLINE OF EXPERIMENT

Recently, the size of scaled model specimens for structural tests tends to become larger and larger. A large model test enables us to obtain data similar to those of real structures. However, since it requires large size testing facilities and large amount of research funds, and makes it difficult to execute parametric tests. The studied building is an RC eleven-storied building model with the scale of 1/15 subjected to a series of base excitations [10, 11]. A general view of the model and sections are shown in Figure 2. Deformed reinforcing bars with nominal diameters of 1, 2 and 3 millimeter were specially prepared for this test series. In addition, micro concrete mixture was used for the structure. For more details about the material models, refer to [11, 12].

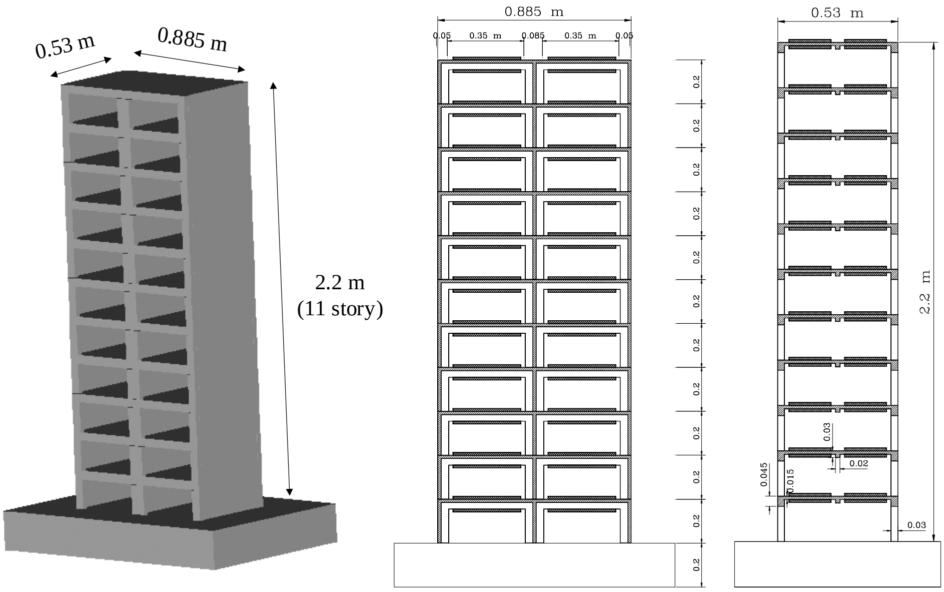


Figure 2: Shape, Dimensions and Loading of a Scaled RC Building Model (1/15) under Lateral Excitation

Table 2: Analysis Stages of the RC Building

Input excitation	**G40, G200, G400, G600, G800-1**				**G800-2**			**G800-2 x 1.5 (magnified excitation)**
Geometry models	**Small deformation**				**Small deformation**	**Large deformation**		**Collapse behavior**
Material models	**The same material models as the experiment**			**Increased Young's Modulus**	**Without cut of RFT**	**Without cut of RFT**	**With cut of RFT**	**With cut of RFT**
Case	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**
Damping ratio (ν)	0.05	0.02	0.02	0.02	0.02	0.02	0.02	0.02
Time increment (dt)	0.005	0.005	0.0025	0.005	0.0025	0.0025	0.0025	0.0025 before collision and 0.0005 after

A series of base excitations were applied to the structure in order from small to large amplitudes (40 to 800 Gal) as shown in Figure 3. The shape of these excitations is same, namely, their frequency characteristics are same while amplitudes are different. The excitation, G800, was applied twice to the structure with normal (G800-1) and doubled (G800-2) time scale, respectively. A series of input base excitations make the numerical analysis more complicated because cumulative damage to the structural elements affects the response. The damage history obtained by experiment after each excitation was as follows [11]:

G40: A flexural crack was observed at the end of a beam at the second story while a few cracks were observed at the columns.

G200: Flexural cracks were observed at the ends of beams at the intermediate stories.

G400: Many cracks were observed, especially in columns at the first story and beams at intermediate stories.

G600: Yielding of reinforcements in columns at the first story and beams at the seventh floor were observed.

G800-1: The response characteristics were similar to those in case of G600.

G800-2: Lower reinforcement bars in beams broke off, shear cracks were observed in column-beam joints. On the exterior surface of the transverse walls, horizontal cracks along the bottom levels of beams were

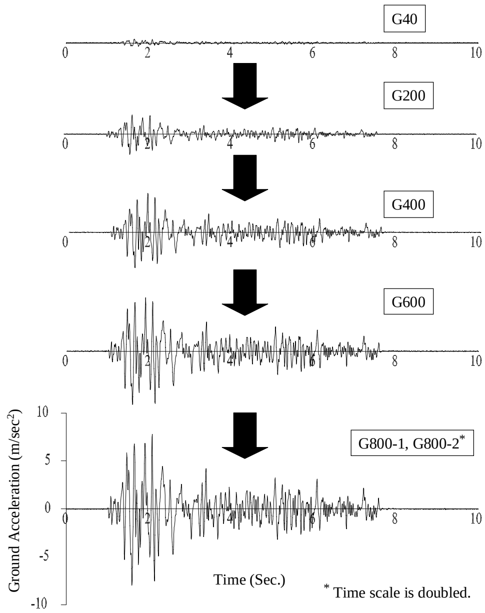


Figure 3: Input Base Accelerations,

G40, G200, G400, G600, G800-1 and G800-2

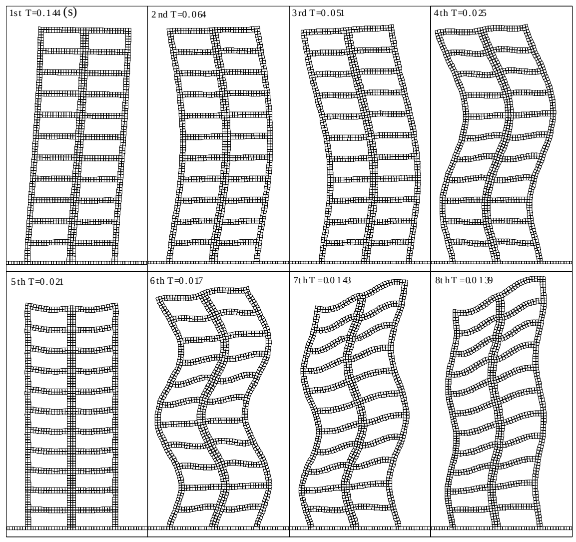


Figure 4: Eigen Value Analysis

(Natural Periods and Modes of the Building Model)

observed. At the second and third stories, cracks due to punching shear were also observed at the intersection of the transverse wall and beams. Although concrete crushed, reinforcement bars buckled and broke off in columns at the bottom of the first story, and yield hinges developed in beams in each floor and columns at the top of the first story, transverse walls could sustain axial force and avoid collapse.

NUMERICAL ANALYSIS AND RESULTS

In this section, numerical analysis of the model under shaking table excitations is performed and the results are introduced. The structure is modelled using 1,232 square-shaped elements and 10 contact points are set between each two adjacent edges. Simulation cases, summarized in Table 2, are performed with considerations of different parameters. Structural behavior is studied from three different viewpoints. First, a series of excitations (G40, G200, G400, G600 and G800-1) are applied to the model structure to study the dynamic properties with different base motions. These series of simulations are carried out by the model without considering geometrical residuals. The G800-2 motion is used in the second step of the simulation where the displacements are large and failure of reinforcement occurs at different floor levels. The last case is for an assumed destructive excitation (G800-2 x 1.5) and the collapse history of the building is simulated. The comparison between the measured and simulated response of different cases is shown in detail in Ref. [12]. Because of the limitation of space, only selected results are introduced. The following factors are considered in the simulation using the AEM:

1. Stress-strain relation of concrete under cyclic loading [13].
2. Stress-strain relation of reinforcement under cyclic loading [14].
3. Additional bending moments because of load eccentricities at columns during tests [3].
4. Large displacements, separation and rigid body motion of structural members during failure [4].
5. Collision of structural members with each other and with the ground [5].

On the other hand, the main assumptions in the analyses are:

1. As the reinforcement and concrete springs are represented using the same element, effects of reinforcement pull out, buckling of reinforcement bars and concrete spalling are not considered.
2. Effects of stiffness of transverse walls and slabs are not considered (only their weight was considered).
3. In collapse analysis, the same assumptions for material modelling stated in Ref. [6] are adopted.

Eigen value analysis was performed to determine the natural modes and frequencies/periods and to compare with those of the model structure. The results for the first eight dominant modes and periods are introduced in Figure 4. Obtaining accurate eigen values and modes indicates that the analysis can be performed in frequency domain as well as in time domain. The calculated natural period using our model, when the slabs and walls effects are neglected, is 0.144 sec., while the measured one is 0.12 sec. Although the lateral walls and slabs are not reinforced to resist bending moments, they increase the structure resistance before crack propagation. This illustrates why the measured natural periods are less than the calculated ones.

Comparison between the calculated and measured responses of the roof during different excitations is introduced. The results of "Case 3", shown in Figures 5 to 10, can be summarized as follows:

Referring to Figure 5, the results showed good agreement till reaching the peak displacement in G40. After the peak displacement, cracks start to propagate and difference between the experiment and analysis increases. It can be noticed that the measured natural period is less than the calculated one.

Analysis Results of G800-2 Excitation:

1. Although the structural behavior in G40 and G200 is close to linear behavior, except for the generation of cracks, results do not show good agreement due to the following reasons:
   a) Although the walls and slabs are not reinforced for bending moment resistance, they share in the global structure resistance during excitations of small amplitude.
   b) After first peak displacement, cracks start to propagate through the walls and connections between beams and slabs, which leads to redistribution of internal stresses between structural elements.
   c) As an evidence of the comments above, results after G200 become better than before. With increasing the amplitude of input excitation, cracking of walls increases and hence, the effects of the wall become smaller. Accuracy of calculated displacements becomes better in G400 and high agreement could be obtained in G600 and G800.
2. After cracking of the concrete walls due to bending moments, their stiffness drastically reduce as they can not resist bending moments. The stiffness of outer frames becomes more dominant after crack propagation in the walls. Numerical results in cases of excitations G400, G600 and G800-1 are closer to the measured ones.
3. From these figures, it is obvious that the natural period of the structurebecomes longer when the response increases. This is due to the damage to structure because of propagation of cracks through the structure, reduction of stiffness and yield of reinforcement during G600.
4. Looking at the free vibration after the main shock of G800, it is obvious that the calculated response is close to the experimental results. This shows that the damping ratio adopted, ν=0.02, is proper and the calculated overall dynamic behavior is similar to the experimental values.

Although the structural behavior, especially during G600 and G800, is highly nonlinear, with cracking, yield of reinforcement and crushing of concrete, excellent agreement between calculated and measured displacement responses could be obtained. Moreover, it is clear that the effects of cumulative damage during the applications of G40, G200 and G400 are considered automatically. This indicates that the AEM can be an efficient and easy tool for nonlinear dynamic analysis of structures.

The results in case of G800-2 are introduced. The excitation of G800-2 is applied after the five input motions, G40, G200, G400, G600 and G800-1. The structural response of G800-2 is much larger than that of G800-1. For example, maximum roof response in case of G800-1 is less than 2.0 cm, while it becomes about 8.0 cm in case of G800-2, which is about 1.5 times as large as the outer column width. The analysis is performed three times with different parameters as shown in Table 2. The results of the three cases are shown in Figures 10, 11 and 12, for Cases 5, 6 and 7, respectively. For Case 5, the followings can be noticed:

1. Good agreement can be obtained till reaching displacement of -0.05 m. Cumulative error increases after this value of displacement. Finally, permanent displacement of the structure occurs because of yield of main reinforcement.
2. The maximum measured displacement, about 8 cm with the reduced-scale structure, is considered very large as it is equivalent to 1.2 m of horizontal displacement in the prototype structure. This indicates that applying the analysis with consideration of geometrical changes may lead to better accuracy.

The natural period calculated through the simulation is generally shorter than that of the experiment. This indicates that the structure damage during the G800-2 is heavier than the calculated damage.

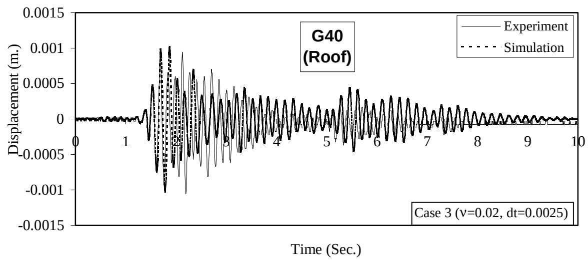


Figure 5: Displacement Response at The Roof for G40 (Case 3)

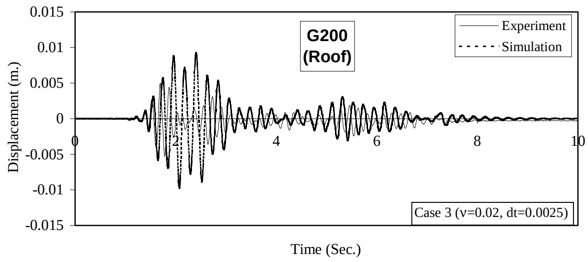


Figure 6: Displacement Response at The Rooffor G200 (Case 3)

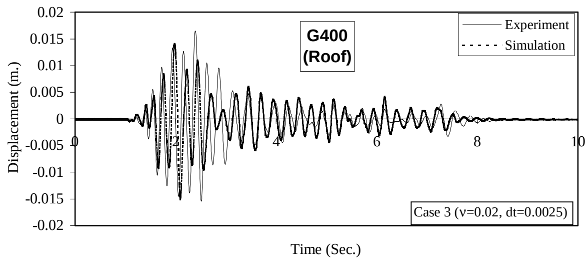


Figure 7: Displacement Response at The Roof for G400 (Case 3)

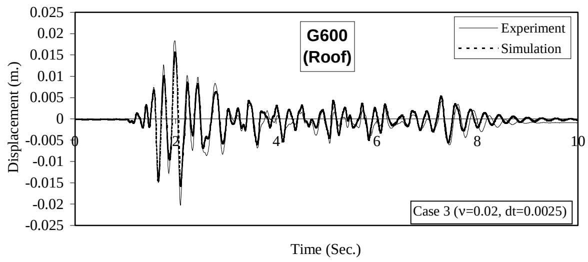


Figure 8: Displacement Response at The Roof for G600 (Case 3)

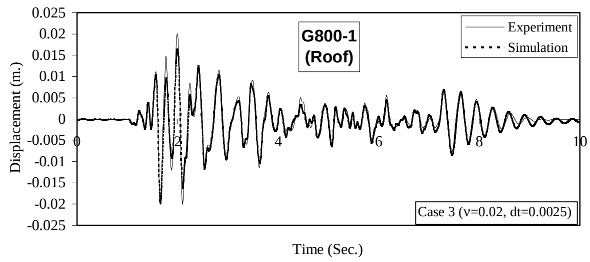


Figure 9: Displacement Response at The Rooffor G800-1 (Case 3)

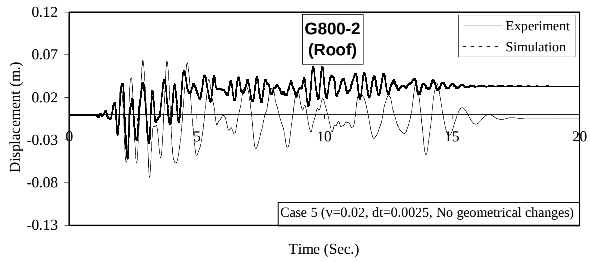


Figure 10: Displacement Response at The Roof f G800-2 (Case 5)

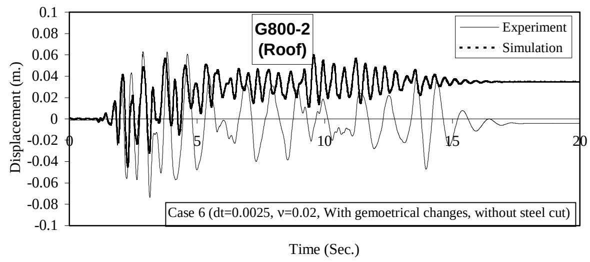


Figure 11: Displacement Response at The Roof for G800-2 (Case 6)

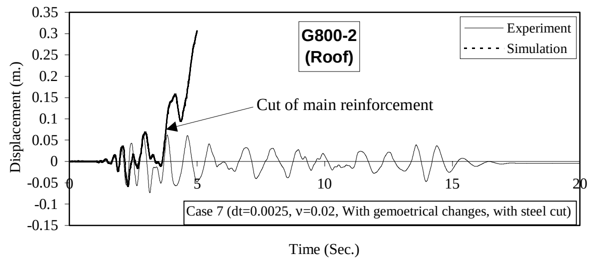


Figure 12: Displacement Response at The Roof for G800-2 (Case 7)

To get better results, analysis is performed for the same structure but using the method based on large displacement formulations. However, cut of reinforcement bars is not permitted (Case 6). The following can be noticed:

1. Comparing with the results obtained when geometrical changes are neglected (Case 5), the obtained displacements are slightly larger because of the considerations of additional moments from the eccentricities of applied vertical load. However, the amount of increase of displacements is not large.
2. The results do not show good agreement with the experiment and the calculated period is still shorter than that of the real structure. This indicates that the effects of cut and buckling of reinforcement may be the reason of this difference. This effect is illustrated in the next section.

To estimate the effects of buckling of reinforcement and spalling of concrete cover, analysis is performed with the condition that large displacement and cut of reinforcement are considered (Case 7). The results are shown in Figure 12. It is obvious that after the cut of reinforcement, very large displacements occur and the structure tends to collapse. Failure of reinforcement is assumed when stresses reach 1.5 times of the yield stress. When reaching the failure criterion, the stresses of the reinforcement-springs are redistributed by applying the total force in the reverse direction. As the reinforcement is the main structural element at this stage, because of crushing of concrete, redistribution of all stresses leads to have very large displacements and unstable behavior start.

It should be emphasized that the adopted material models are not sufficient to follow accurately the structural behavior during complicated collapse process because of the following reasons:

a) Referring to Section 3, cut of reinforcement occurred at the beams of 4^{th} and 7^{th} floors. The calculated bending moments through numerical techniques show that bending moments at the beams of first floor become largest. This indicates that failure, from numerical point of view, should occur also in the beams of the first floor. Having failure of reinforcements of the beams of the 4^{th} and 7^{th} floors indicates that there are some stochastic variations of reinforcement and concrete properties that affect the cut of reinforcements.

b) The complicated behavior of reinforcement bars during failure is not considered, like buckling and pull out of reinforcement bars, spalling of concrete cover, etc. In addition, these effects can not be considered properly under assumption of two-dimensional analysis.

Collapse Analysis under Magnified Excitation:

The experiment was performed up to G800-2, which was almost the upper limitation of the capacity of the shaking table used, but the structure did not totally collapse. However, as the AEM is numerical model, any input ground motion can be applied. Therefore, the amplitude of the base excitation of G800-2 is multiplied by 1.5 and applied to the same structure. As the collapse process is very complicated, it can not be represented accurately using two-dimensional model.

However, as the main purpose of the simulation is to show the high potential of applicability of the AEM as numerical tool, relatively simple models for concrete crushing and reinforcement cut are adopted [5]. The analysis is performed using time increment of 0.0025 and 0.0005 seconds before and after recontact between elements occurs, respectively. The collapse process, shown in Figure 13, can be summarized as follows:

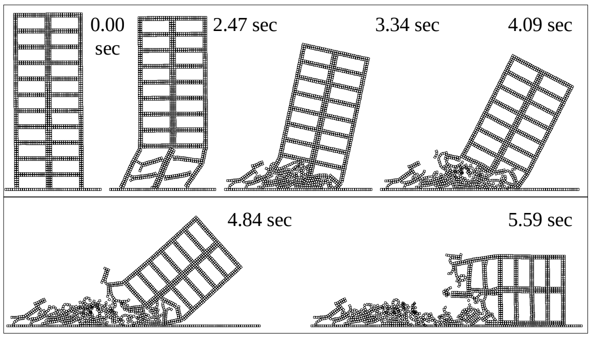


Figure 13: Simulation of Collapse Process (Amplitude: 1,200 Gal, Time Scale: Double)

(In The Experiment, This Case Could Not Be Performed Because of Limitation of Shaking Table Capacity.)

1. Failure starts by excessive cracking, and progresses by yield and cut of reinforcement at base columns and beams.
2. Collision of the failed beams with other structural members during collapse causes excessive damage for the lower floors.
3. Although columns and beams of lower floors suffer complete damage, the upper floors suffer almost no damage but they move together in rigid body motion and rotate around the failed structural elements till they collide with the ground. As the applied vertical loads is smaller than actual cases because of adjustment of scale factor used [10], collision forces of the upper floors with the ground are relatively small and therefore they do not cause additional damage.

CONCLUSIONS

Experimental results of a reduced-scale RC building model subjected to a shaking table loading are compared with the results obtained by the newly proposed method, AEM. It can be concluded that:

1. The AEM can be applied for studying the nonlinear dynamic behavior of structures. Effects of cracking, concrete crushing, yield of reinforcement can be considered.
2. All reinforcement details can be considered without any additional complications to the analysis.
3. The rigid body motion and collision of structural elements during collapse can be followed with reliable accuracy.
4. No previous knowledge about the collapse process is needed before the analysis.
5. For future study of detailed collapse mechanism of structures, additional effects like buckling of reinforcement bars and spalling of concrete cover, etc. are needed to be modeled which are not taken into account in the AEM yet.
6. It is very difficult, or impossible, to follow such behavior using the methods that assume continuum material, like FEM and BEM, where failure analysis is still restricted to cases where crack locations are known before the analysis.
7. The EDEM is one of the few models that can simulate such collapse behavior. The main advantage of the AEM compared to the EDEM is that the time increment can be enlarged before collision starts. In EDEM, the time increment is restricted to a certain small value which is function of the material type and element size. It can not be enlarged even before collision starts. In the AEM, longer time increment can be used before collision and it makes the CPU time required much shorter.

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