Research paper

A study on the load redistribution characteristics of RC structures through progressive collapse analysis

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Abstract

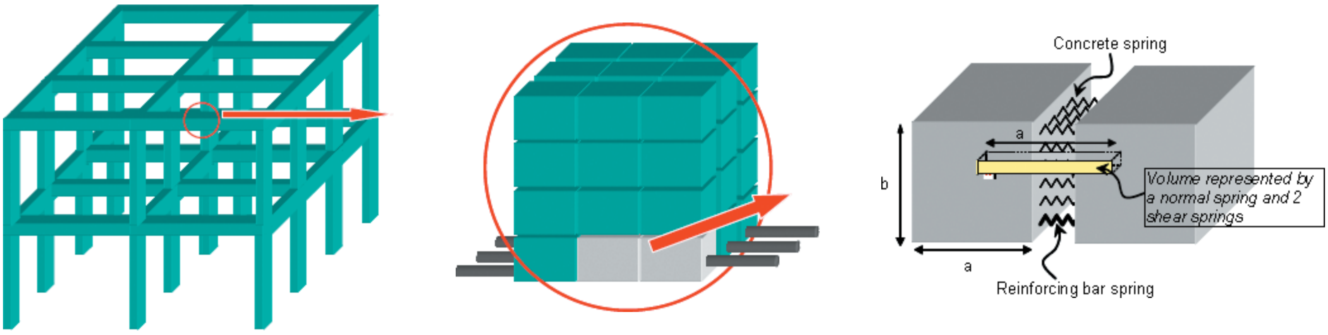
Progressive collapse is a phenomenon in which the local damage of structural members occurring due to abnormal load, such as impact or explosion, leads to the partial collapse or total collapse of the structure. Explosive demolition is a method of inducing the progressive collapse of a structure by removing the main structural members through blasting. Unlike progressive collapse, in explosive demolition the local damage of the structural members can be controlled at appropriate time intervals by blasting, to induce the progressive collapse of the structure and control the collapse behavior. Therefore, to induce the progressive collapse of an entire structure, it is important to select an appropriate time interval for the demolition of surrounding structural members according to the load redistribution of the structure. In this study, a progressive collapse analysis was carried out in order to consider and apply the load redistribution of structural members due to local damages to the explosive demolition design of the reinforced concrete structure. The occurrence of progressive collapse according to the number and position of column elements to remove was examined using ELS (Extreme Loading for Structures) software based on AEM (Applied Element Method). The vertical velocity and displacement of elements on top of the removed columns were compared, and the vertical internal force applied to the columns surrounding the removed column as a result of load redistribution was analyzed.

Keywords : progressive collapse, load redistribution, explosive demolition, appliecl element method, abnormal load

1. Introduction

Progressive collapse is a phenomenon in which the local damage of structural members occurring due to abnormal loads that are not considered in general structural design leads to the partial collapse or total collapse of the structure. An abnormal load is a load that a designer does not consider in the design stage, such as a load due to gas explosion, terrorist bomb explosion or impact, vehicle or airplane collision and fire. When local damage occurs, the structure holds new load paths for load redistribution, and each structural member holds new load after load redistribution. When this new load exceeds the strength of the structural member, partial collapse occurs, and when partial collapse is transferred to the whole structure, it may lead to the total collapse of the structure[1].

Studies on progressive collapse have been carried out, driven in part by three significant collapse accidents. The collapse of Ronan Point apartment building in London following a gas explosion in 1968 sparked interest in the progressive collapse of structures for the first time. After the collapse of Alfred P. Murrah building (US) in 1995 due to the detonation of a bomb by a terrorist on the ground floor, studies were carried out to derive approaches to evaluating a structure's resistance to impact or explosion. Following the collapse of the World Trade Center after airplane collisions in 2001, various studies on progressive collapse were carried out[2]. In 2003, the US GSA (General Service Administration) evaluated the possibility of progressive collapse for federal facilities, and presented the GSA guideline in order to improve structural



(a) Structure. (b) Element generation for AEM. (c) Spring distribution and area of influence of each pair of springs.

Figure 1 Modeling of structure to AEM (Tagel-Din, 2009).

performance.

Explosive demolition is a method of inducing the collapse of a structure by removing columns and bearing walls, which are the main structural members, in consecutive order through blasting. The loads of the structure are redistributed due to main members being blasted in consecutive order, and are applied to the surrounding members as dynamic loads, affecting the structural rigidity distribution. Unlike progressive collapse, explosive demolition can control local damage of the structural members in appropriate time intervals by blasting to induce the progressive collapse of the structure, thus controlling the collapse behavior. Therefore, to induce the progressive collapse of the whole structure due to initial collapse behavior, the surrounding structural members should be blasted and removed at appropriate time intervals before the loads of the structure reach the condition of structural equilibrium. The use of inappropriate time intervals affect the collapse behavior of the structure, causing kick back[3)-4)].

In this study, progressive collapse analysis was carried out in order to consider and apply the load redistribution of structural members due to local damages to the explosive demolition design of the reinforced concrete structure. The ELS (Extreme Loading for Structures) program based on AEM (Applied Element Method) was used for the progressive collapse analysis. The column elements that were the main vertical members were removed by modeling a 10-floor reinforced concrete structure and using the IER (Immaculate Element Removal) technique[5)]. The occurrence of progressive collapse was examined according to the number and position of column elements to remove. For models that resisted progressive collapse, the vertical velocity and displacement of elements on top of the removed columns were compared, and the vertical internal force applied to the columns surrounding the removed column was analyzed.

2. Applied element method (AEM)

AEM is a new modeling technique that adopts the concept of discrete cracking[5)-7)]. As shown in Figure 1, the structure is designed in a group of small elements. Surrounding elements are connected with vertical and shear springs, which transfer normal stresses and shear stresses between different elements. When the spring connection is destroyed, surrounding elements are separated.

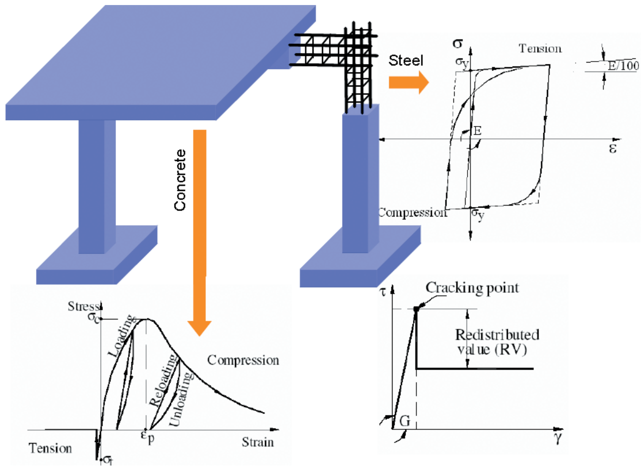


Figure 2 Constitutive models for concrete and reinforcing bar (Tagel-Din, 2009).

Figure 2 shows the model of material used in AEM and the fracture criteria. For concrete in compression, the Maekawa compression model[8)], which is an elasto-plastic and fracture model, is adopted. When concrete is subjected to tension, linear stress-strain relationship is adopted until the concrete springs crack, at which point the stresses drop to zero. The model suggested by Ristic et al[9)]. was used for reinforcing bars. The main advantage of this model is that it can easily consider the effects of partial unloading and Baushinger's effect without any additional complications to the analysis[10)]. The concrete behavior in shear is linear until it reaches the cracking strain, which is calculated based on principle stress criteria. Once the springs reach the cracking criterion, the whole shear strength value at the face of the crack is redistributed (RV=1.0).

For validating the AEM ability to analyze the progressive collapse-resistance of structures, Salem et al[11)]. performed a structural analysis the experimental study carried out by Wei et al[12)]. The authors compared the force versus downward displacement obtained from the experimental results and the AEM analytical results. As shown in Figure 3, the AEM analytical results are validated. Figure 4 compared the collapse limit state of the frame predicted by AEM with that experimental results,

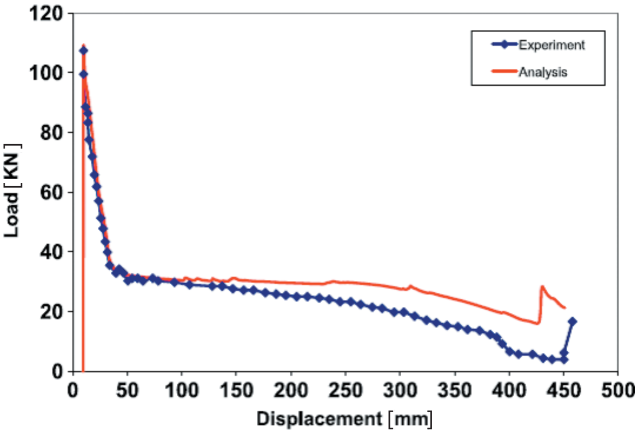


Figure 3 Middle column load vs. displacement of removed column predicted by AEM compared to the experimental results of Wei et al. (Salem et al., 2011).

which also verifies the AEM results.

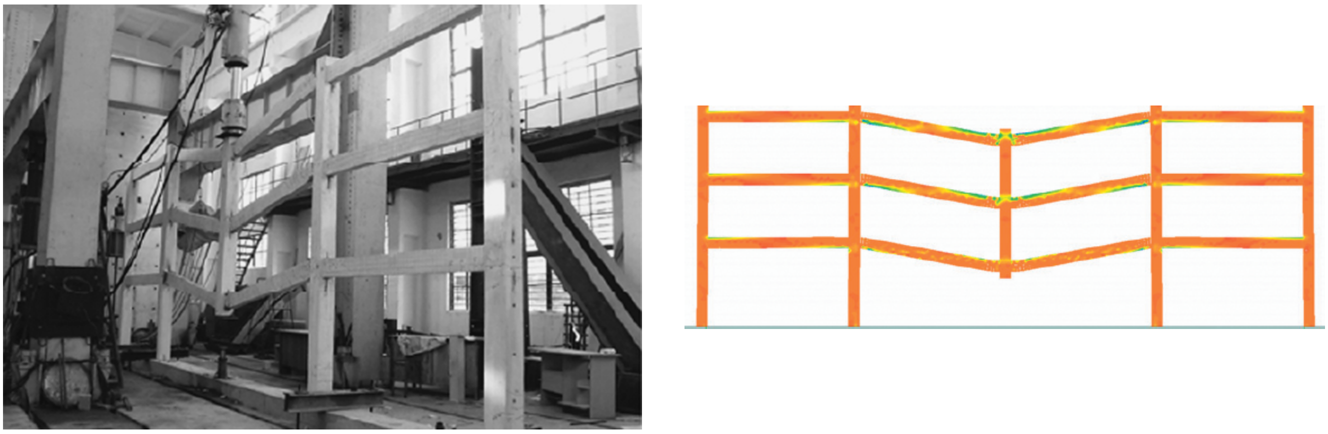
3. Progressive collapse analysis

3.1 Analysis model

The analysis structure is a 6×6 bay, 10-floor reinforced concrete structure, and each bay between columns is 6 m and the floor height is 4 m. The structure is designed according to the KSCE-USD05 reinforced concrete design code using the Midas Civil2012 structure analysis software. The structure was designed in 3D according to the dimensions of concrete members and the arrangement of reinforcing bars given in Figure 5 using the ELS, and Figure 6 shows the analysis model using the ELS. The physical properties of the concrete and reinforcing bars used in the analysis model are as shown in Table 1.

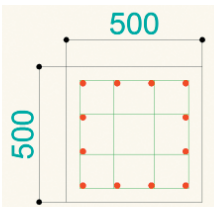
3.2 Analysis method

Nonlinear dynamic analysis was carried out by changing the position and number of columns to remove



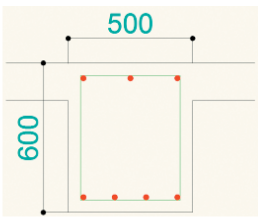
(a) Experiment[12] (b) AEM

Figure 4 Collapse limit state of the frame predicted by AEM compared to the experimental results of Wei et al. (Salem et al., 2011).



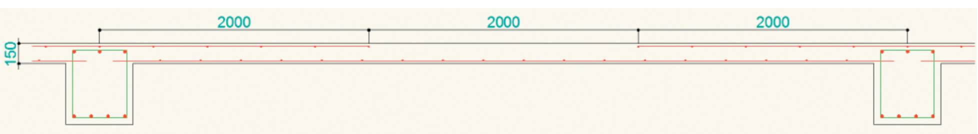
(a) Column.

Main bars : 12-D22
Tie bars at the end : D13@300
Tie bars at the center : D13@300



(b) Girder.

Top bar : 3-D22
Bottom bar : 4-D22
Stirrups : D13@300



Top bar : D10@400
Bottom bars : D10@300

(c) Slab.

Figure 5 Dimensions of concrete members and reinforcing bar arrangement drawing (unit : [mm]).

Table 1 Material properties used in analysis.

Material	Young's modulus [MPa]	Compressive strength [MPa]	Tensile strength [MPa]	Tensile yield stress [MPa]	Ultimate strength [MPa]
Concrete	26,200	24	0.2	—	—
Reinforcement	200,000	—	—	360	504

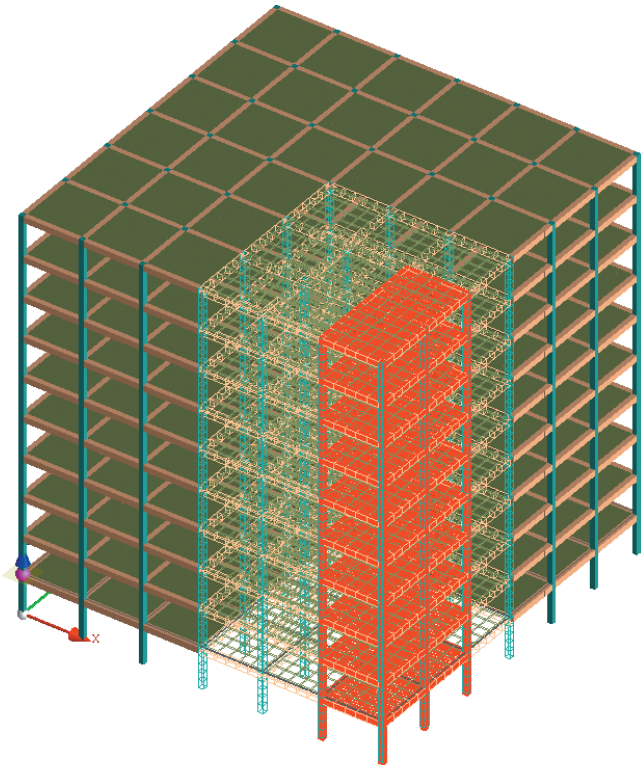


Figure 6 AEM model for the concrete and reinforcing bars.

on 1st floor columns of the analysis model. The column elements of each analysis model were removed at the same time using the IER technique. Figure 7 shows the position of column elements to remove for each analysis model in this study.

For the load combination for carrying out the dynamic analysis, formula (1), which is the dynamic analysis load presented by the GSA (General Services Administration), was applied[13].

$$\textit{Dynamic Analysis Load} = DL + 0.25 \cdot LL \quad (1)$$

where, DL is Dead Load, and LL is Live Load.

The number of elements in the analysis model is 15,650, the number of springs is 926,317 and the number of frames is 511. The analysis is carried out in two stages. The first stage is the step of static analysis to analyze the initial deformation of the whole structure due to its own weight before removing columns. The second stage is the step of dynamic analysis, to carry out nonlinear analysis after removing columns. The time step of the dynamic analysis is 0.001s, and the total analysis time is set to 5s and 10s according to the analysis model used.

The result of the progressive collapse that occurred in each analysis model and the vertical velocity and displacement of the elements on top of the removed columns for the models that resisted the progressive collapse were compared. In addition, the vertical internal force applied to the columns surrounding the removed columns according to the load redistribution was analyzed.

4. Results of analysis, and discussion

4.1 Progressive collapse

The structural collapse result of each analysis model is as shown in Table 2. The analysis of the 1 column removal models and the 2 column removal models showed that no structural collapse occurred regardless of the position of columns removed. This shows that the structure resists progressive collapse due to load redistribution after the columns are removed.

Figure 8 shows the distribution of principal stress that occurred at the slab of 1 column removal models according to the position of the removed column. It shows that tensile stress occurs on the slabs surrounding the removed columns in the diagonal direction due to catenary action of the slabs, resisting the collapse of the structure.

Figures 9~12 show the result of an analysis of normal stress applied to the top bars and bottom bars of girder in 1 internal column 1 model and 2 internal columns 1 model according to the time elapsed after the columns were removed. Before the columns are removed, tensile stress is applied to the top bars and compressive stress is applied to the bottom bars. Immediately after the columns are removed, deformations occur from the position of removed elements to the bottom direction, so the tensile stress applied to the top bars decreases and compression is applied. On the other hand, the compressive stress applied to the bottom bars decreases and is converted to tension. As deformations increase, the tensile stress applied to the bottom bars also increases and the compression applied to the top bars is converted to tension again. In the meantime, the compressed concrete is tensioned again so that it will not be collapsed by compression. This phenomenon shows that the catenary action of the girder increases after a certain time, and reaches a stable equilibrium condition so that the structure will not collapse.

In 3 column removal models and 4 column removal models, partial collapse occurred in the 3 corner columns removal model and the 4 corner columns removal model, and total collapse occurred in other models. When 3 or more columns were removed, progressive collapse occurred due to the rupture of remaining columns according to the load redistribution. Figure 13 shows the collapse behavior of 4 columns removal models according to each analysis time.

4.2 Vertical velocity and displacement

The result of an analysis of vertical velocity and

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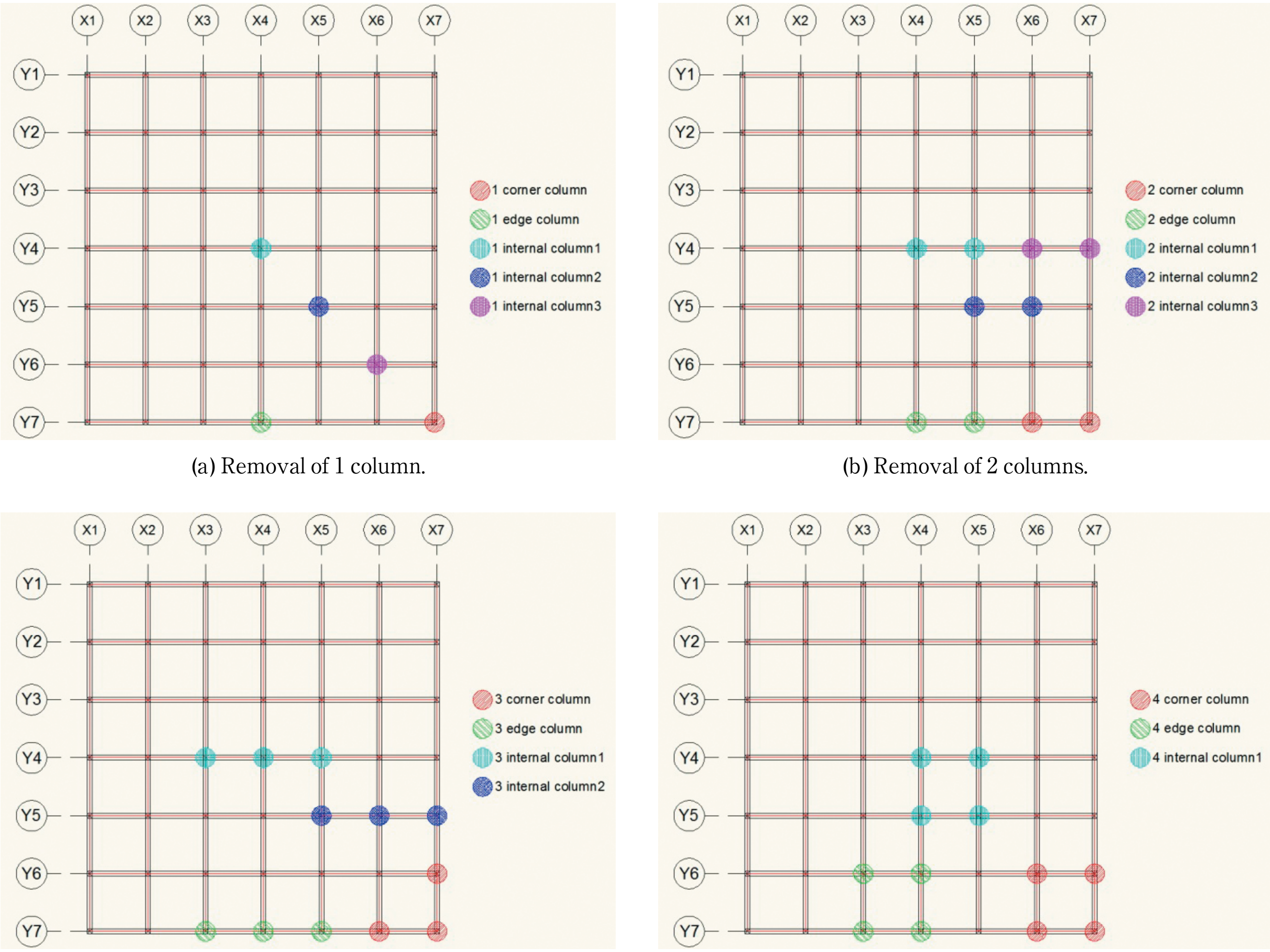


(a) Removal of 1 column.

(b) Removal of 2 columns.

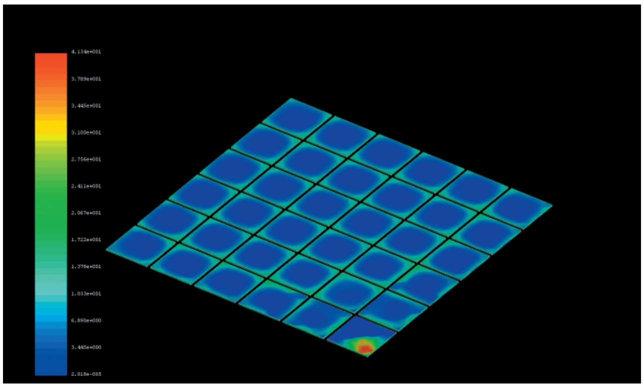
(c) Removal of 3 columns.

(d) Removal of 4 columns.

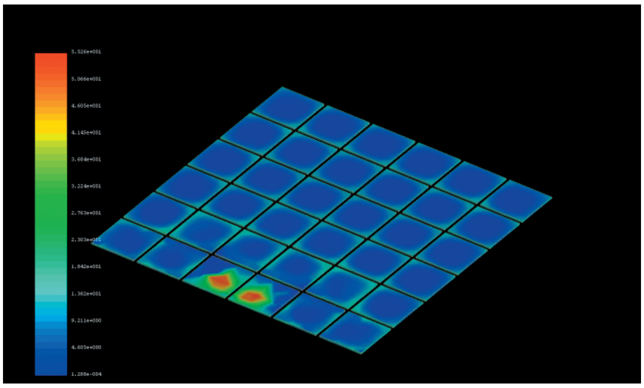
Figure 7 Position of columns to remove for each analysis model.

Table 2 Structural collapse result of each analysis model.

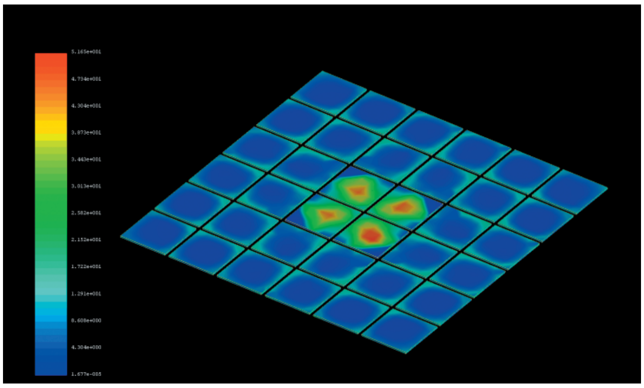
Number of columns removed	Analysis model	Analysis result	Remarks
1 column	1 corner column	No collapse	Resisted against the progressive collapse
	1 edge column	No collapse	Resisted against the progressive collapse
	1 internal column 1	No collapse	Resisted against the progressive collapse
	1 internal column 2	No collapse	Resisted against the progressive collapse
	1 internal column 3	No collapse	Resisted against the progressive collapse
2 columns	2 corner column	No collapse	Resisted against the progressive collapse
	2 edge column	No collapse	Resisted against the progressive collapse
	2 internal column1	No collapse	Resisted against the progressive collapse
	2 internal column 2	No collapse	Resisted against the progressive collapse
	2 internal column 3	No collapse	Resisted against the progressive collapse
3 columns	3 corner column	Partial collapse	Progressive collapse occurred
	3 edge column	Total collapse	Progressive collapse occurred
	3 internal column 1	Total collapse	Progressive collapse occurred
	3 internal column 2	Total collapse	Progressive collapse occurred
4 columns	4 corner column	Partial collapse	Progressive collapse occurred
	4 edge column	Total collapse	Progressive collapse occurred
	4 internal column 1	Total collapse	Progressive collapse occurred



(a) 1 corner column.



(b) 1 edge column.



(c) 1 internal column 1.

Figure 8 Principal stresses contours in the slabs after the column removal.

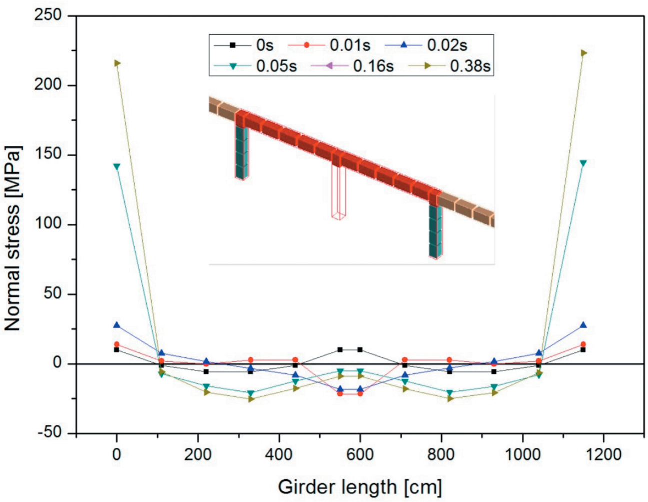


Figure 9 Normal stress distribution along top bars in the girder of 1 internal column model.

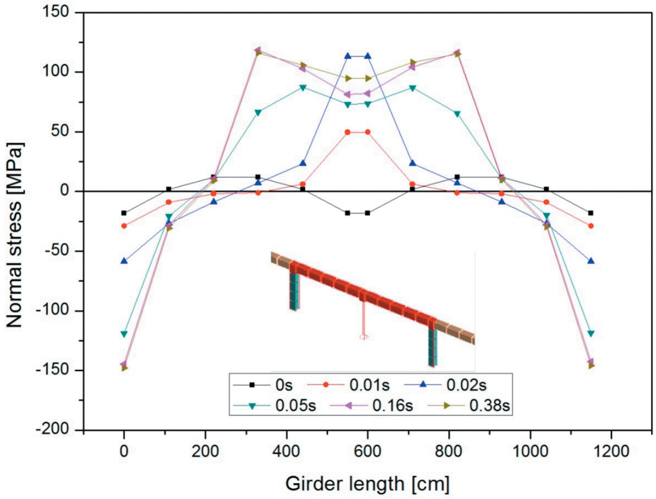


Figure10 Normal stress distribution along bottom bars in the girder of 1 internal column model.

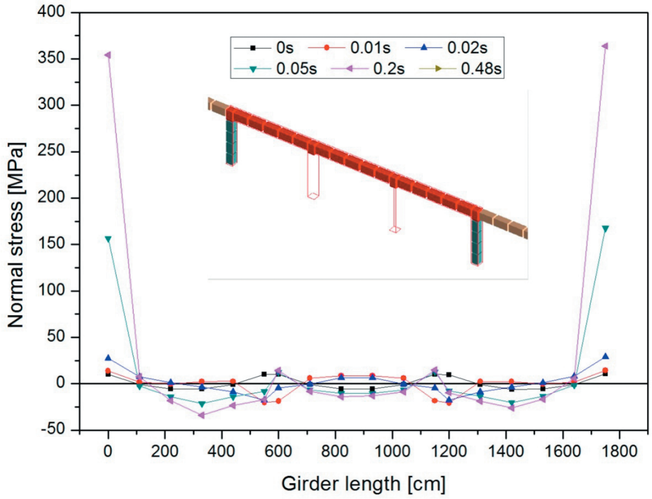


Figure11 Normal stress distribution along top bars in the girder of 2 internal column model.

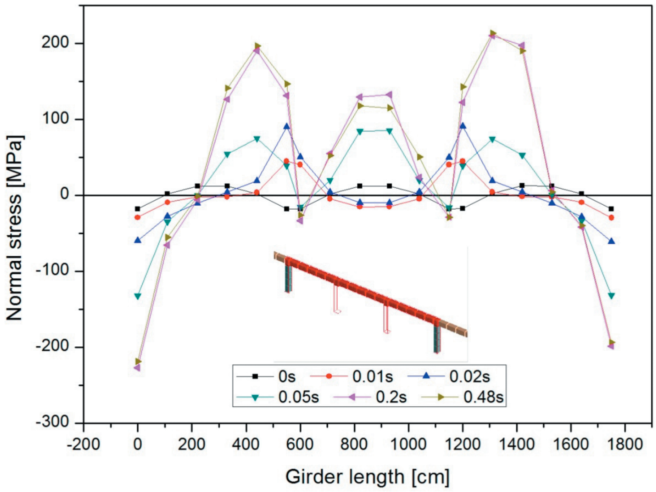
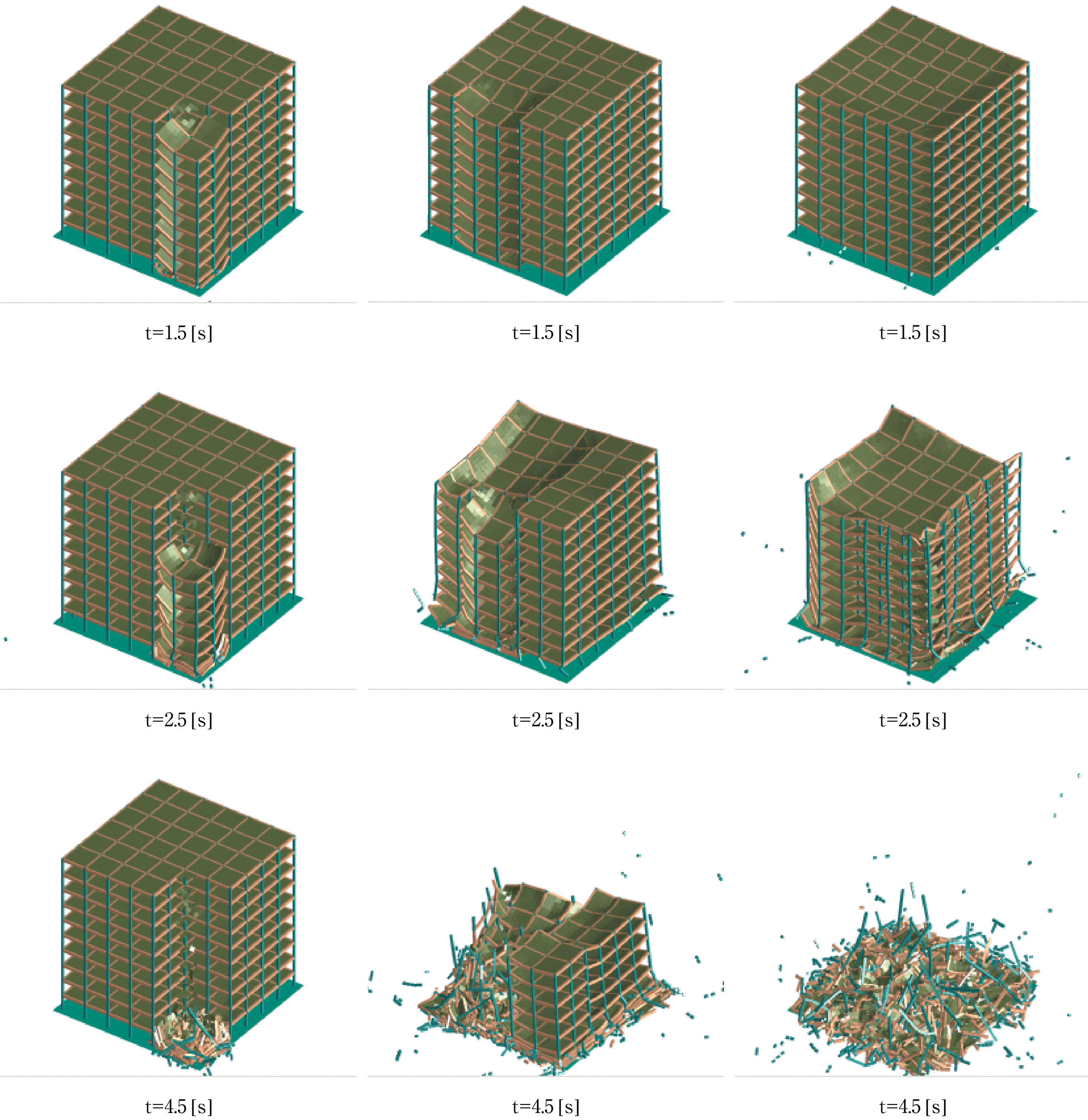


Figure12 Normal stress distribution along bottom bars in the girder of 2 internal column model.

displacement of the elements on top of the removed columns in 1 column removal models and 2 column removal models which resisted the progressive collapse is as shown in Tables 3 and 4.

The influence of the position of columns removed had a



(a) 4 corner column model. (b) 4 edge column model. (c) 4 internal column1 model.

Figure13 Collapse behavior of 4columns removal models according to analysis time.

higher influence on the vertical velocity than the number of columns removed. Also, the vertical velocity was higher in the internal column removal models than in the surrounding column removal models. The initial velocity of the1 internal column1 (X4Y4) model was approximately 27.75% higher than the initial velocity of the 1 corner column (X7Y7) model, and the initial velocity of the 2 internal columns 1 (X4Y4) model was approximately 27.71% higher than the initial velocity of the 2 corner columns (X7Y7) model. The initial velocity of elements on top of the removed columns was proportional to the normal force applied at the top of the removed columns as shown in Figure 14.

The occurrence time of initial displacements was 0.15~0.18s in the 1 column removal models and 0.2~0.38s in the 2 column removal models. The maximum displacement and the residual displacement were highest in the corner column models, and this is considered to reflect the influence of new load paths according to the load redistribution.

For the maximum displacement, the 2 corner columns (X7Y7) model was 76.3% higher than the 1 corner column (X7Y7) model, the 2 edge columns (X4Y7) model was 64.8% higher than the 1 edge column (X4Y7) model, and the 2 internal columns 1 (X4Y4) model was 47.9% higher than the 1 internal column 1 (X4Y4) model.

Table 3 Vertical velocity and displacement of 1 column removal models.

Z-direction	Initial velocity		Maximum velocity		Initial displacement		Maximum displacement		Residual displacement	
	Time [s]	Value [cm s-1]	Time [s]	Value [cm s-1]	Time [s]	Value [cm]	Time [s]	Value [cm]	Time* [s]	Value [cm]
1 corner column (X7Y7)	0.01	-44.55	0.01	-44.55	0.18	-2.56	0.94	-2.87	5.00	-2.75
1 edge column (X4Y7)	0.01	-53.52	0.01	-53.52	0.15	-2.16	0.86	-2.24	5.00	-2.11
1 internal column 1 (X4Y4)	0.01	-61.66	0.01	-61.66	0.16	-2.05	0.38	-2.11	5.00	-1.97
1 internal column 2 (X5Y5)	0.01	-61.20	0.01	-61.20	0.16	-2.16	0.4	-2.29	5.00	-2.18
1 internal column 3 (X6Y6)	0.01	-55.66	0.01	-55.66	0.17	-2.57	0.17	-2.57	5.00	-2.28

Note :*total analysis time

4.3 Vertical internal force

Tables 5 and 6 show a comparison of changes in the vertical internal force applied to the surrounding columns connected to the removed columns with girder in the 1 column removal models and the 2 column removal models. The rate of increase of vertical internal force in 1 column removal models is 25.2~39.2%, while the rate of increase of vertical internal force in 2 column removal models is 32.0~45.2%. The rate of increase of vertical internal force was higher at the exterior columns of the structure than at the interior columns of the structure.

Figures 15 and 16 show the relationship between vertical internal forces applied to the surrounding columns and time. Maximum vertical internal force occurred in the 1 column removal models at 0.11~0.15s, while it occurred at 0.13~0.21s in the 2 column removal models. In some models, the vertical internal force applied to the surrounding columns due to the load redistribution exceeded the design strength of the concrete column but did not exceed the ultimate strength of reinforcing bars, and thus the concrete columns were not ruptured completely.

We can induce the progressive collapse of the structure through load redistribution by applying the above progressive collapse analysis results to the explosive demolition design of the reinforced concrete structure. The position and vertical load of columns to blast should be considered for the drop velocity of the structure in the explosive demolition design of a 10-floor reinforced concrete structure, which is the analysis model in this study. Furthermore, a delay time of 0.1~0.15s between surrounding columns would be appropriate if blasting one column at a time, while a delay of 0.2~0.25s would be appropriate if blasting two columns simultaneously.

5. Conclusions

In this study, the progressive collapse analysis was carried out to consider and apply the load redistribution resulting from the local damage of structural members to the explosive demolition design of the reinforced concrete structure. Based on the analysis of the AEM, the following conclusions are obtained.

The catenary action of the slab and girder increased after a certain time in the1column removal models and the 2 column removal models, regardless of the position of the removed columns, and no collapse of the structure occurred. The vertical velocity of the models that resisted the progressive collapse was influenced by the position of the columns to remove, and the vertical velocity of the internal column removal models increased. Also, the initial velocity applied on top of the removed columns was proportional to the normal force applied at the top of the removed columns. The time of the occurrence of initial displacements was 0.15~0.18s in the 1 column removal models and 0.2~0.38s in the 2 column removal models. The time of maximum vertical internal force in the1column removal models was 0.11~0.15s, while in the 2 column removal models it was 0.13~0.21s. The maximum displacement due to the influence of new load paths according to the load redistribution was highest in the corner column models, and the rate of increase of vertical internal force was higher at the exterior columns of the structure than the interior columns of the structure. Using the results of the progressive collapse analysis, it is possible to select the position of columns to blast and the optimal delay time of blasting according the number of columns blasted at the same time, and apply it to the explosive demolition design of a reinforced concrete structure.

Table 4 Vertical velocity and displacement of 2 columns removal models.

Z-direction	Initial velocity		Maximum velocity		Initial displacement		Maximum displacement		Residual displacement	
	Time [s]	Value [cm s^{-1}]	Time [s]	Value [cm s^{-1}]	Time [s]	Value [cm]	Time [s]	Value [cm]	Time* [s]	Value [cm]
2 corner column (X7Y7)	0.01	-45.66	0.15	-49.13	0.34	-11.92	1.21	-12.11	10.00	-11.68
2 corner column (X6Y7)	0.01	-52.39	0.01	-52.39	0.38	-10.43	1.23	-10.64	10.00	-10.33
2 edge column (X5Y7)	0.01	-54.95	0.01	-54.95	0.24	-6.42	0.60	-6.44	10.00	-5.96
2 edge column (X4Y7)	0.01	-54.87	0.01	-54.87	0.24	-6.35	0.60	-6.37	10.00	-5.91
2 internal column 1 (X5Y4)	0.01	-62.94	0.01	-62.94	0.20	-4.05	0.48	-4.09	10.00	-3.74
2 internal column 1 (X4Y4)	0.01	-63.16	0.01	-63.16	0.20	-3.98	0.49	-4.05	10.00	-3.70
2 internal column 2 (X6Y5)	0.01	-59.79	0.01	-59.79	0.21	-4.70	0.51	-4.73	10.00	-4.24
2 internal column 2 (X5Y5)	0.01	-62.63	0.01	-62.63	0.21	-4.61	0.21	-4.61	10.00	-4.17
2 internal column 3 (X7Y4)	0.01	-54.98	0.01	-54.98	0.25	-6.26	0.95	-6.40	10.00	-5.98
2 internal column 3 (X6Y4)	0.01	-59.60	0.01	-59.60	0.22	-5.85	0.22	-5.85	10.00	-5.42

Note :*total analysis time

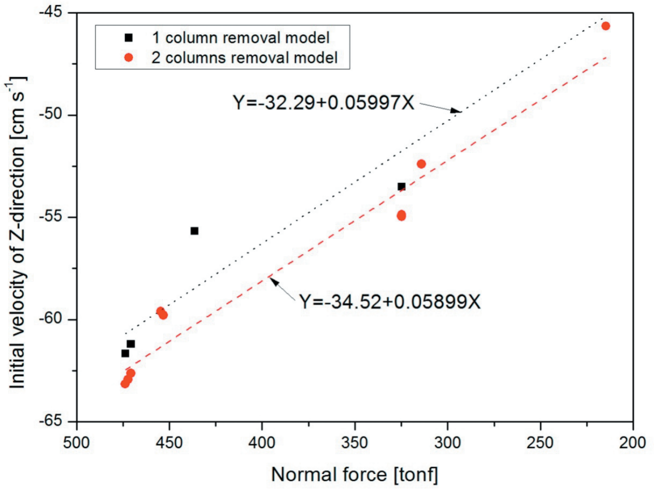


Figure14 The relationship between initial velocity and normal force applied on top of the removed columns.

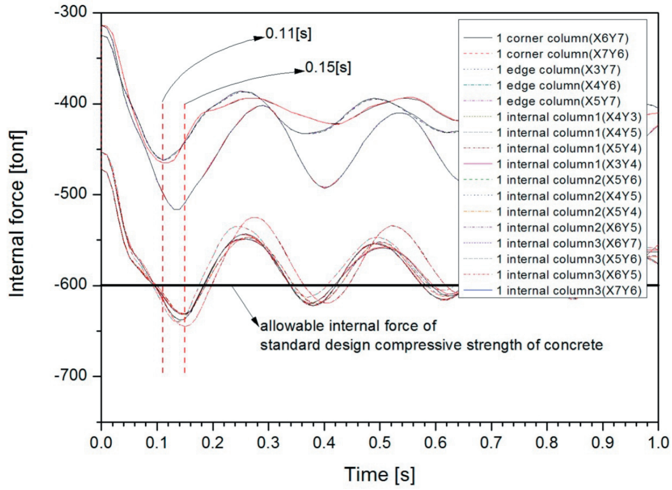


Figure15 The relationship between the vertical internal forces applied to the surrounding columns and time at 1 column removal models.

Table 5 Vertical internal force of 1 column removal models.

Analysis model	Coordinate of surrounding column	Initial vertical internal force [tonf]	Maximum vertical internal force [tonf]	Rate of increase [%]
1 corner column	X6Y7	-314.1	-465.4	32.5
	X7Y6	-314.1	-465.4	32.5
1 edge column	X3Y7	-325.1	-462.3	29.7
	X4Y6	-454.6	-640.2	29.0
	X5Y7	-325.1	-461.9	29.6
1 internal column 1	X3Y4	-472.2	-631.4	25.2
	X4Y3	-472.2	-631.0	25.2
	X4Y5	-472.2	-631.8	25.3
	X5Y4	-472.2	-631.2	25.2
1 internal column 2	X4Y5	-472.2	-632.5	25.3
	X5Y4	-472.2	-631.7	25.2
	X5Y6	-453.2	-638.0	29.0
	X6Y5	-453.2	-637.9	29.0
1 internal column 3	X5Y6	-453.2	-644.6	29.7
	X6Y5	-453.2	-644.7	29.7
	X6Y7	-314.1	-516.1	39.2
	X7Y6	-314.1	-516.2	39.2

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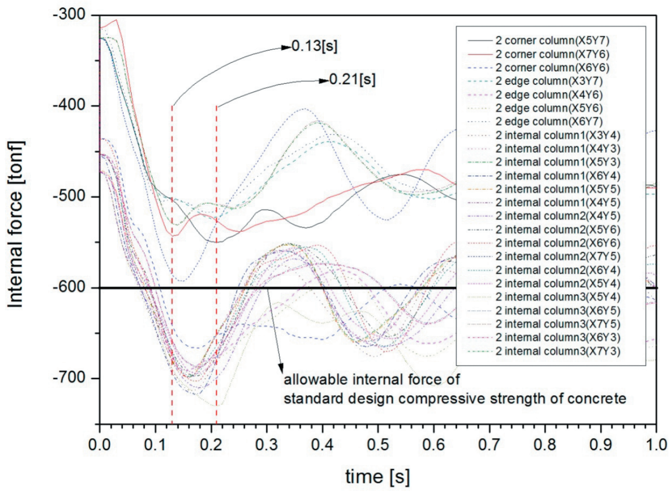

Figure 16 The relationship between the vertical internal forces applied to the surrounding columns and time at 2 columns removal models.

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Table 6 Vertical internal force of 2 columns removal models.

Analysis model	Coordinate of surrounding column	Initial vertical internal force [tonf]	Maximum vertical internal force [tonf]	Rate of increase [%]
2 corner column	X5Y7	-325.1	-549.9	40.9
	X6Y6	-436.3	-666.5	34.5
	X7Y6	-314.1	-542.7	42.1
2 edge column	X3Y7	-325.1	-523.9	37.9
	X4Y6	-454.6	-691.0	34.2
	X5Y6	-453.2	-694.8	34.8
	X6Y7	-314.1	-516.9	39.2
2 internal column 1	X3Y4	-472.2	-713.1	33.8
	X4Y3	-472.2	-698.1	32.4
	X4Y5	-472.2	-698.5	32.4
	X5Y3	-470.7	-698.4	32.6
	X5Y5	-470.7	-698.8	32.6
	X6Y4	-454.6	-716.8	36.6
2 internal column 2	X4Y5	-472.2	-710.2	33.5
	X5Y4	-472.2	-694.2	32.0
	X5Y6	-453.2	-696.9	35.0
	X6Y4	-454.6	-697.8	34.8
	X6Y6	-436.3	-703.4	38.0
	X7Y5	-325.1	-592.8	45.2
2 internal column 3	X5Y4	-472.2	-730.5	35.4
	X6Y3	-453.2	-687.6	34.1
	X6Y5	-453.2	-686.7	34.0
	X7Y3	-325.1	-530.7	38.7
	X7Y5	-325.1	-531.4	38.8