Research paper

Relations between the explosive demolition design variables and progressive collapse resisting capacity of RC frame structures

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Abstract

Progressive collapse is the partial or entire collapse of structures caused by the local damage of structural members arising from an abnormal load such as impact or explosion. Explosive demolition is a method to induce progressive collapse of the whole structure by removing primary structural members through blasting. Unlike progressive collapse, the explosive demolition induces progressive collapse of the structure by controlling the local damage of structural members at appropriate delay time. In this study, the progressive collapse resisting capacity depending on the number of floors in the structure, height of column at the target floor for blasting, and changes in the span length among the explosive demolition design variables of RC frame structure was evaluated. The final collapse pattern of each analysis model with vertical and free fall displacements applied to the direct top elements of the removed columns were analyzed using the AEM (Applied Element Method) based ELS (Extreme Loading for Structures) software. Also, the vertical displacement applied to the direct top elements of the removed columns was compared with the vertical internal force applied to the adjacent columns. The normal stress of reinforcing bar acting on as per time between the girders, which are the removed columns and adjacent columns, and the adjacent column connections were compared, and the progressive collapse resisting capacity by the catenary action was analyzed.

Keywords : progressive collapse analysis, resisting capacity evaluation, explosive demolition, AEM, catenary action

1. Introduction

Progressive collapse is the partial or entire collapse of structures caused by the local damage of structural members arising from an abnormal load[1]. When local damage occurs, the load applied to the structural members is delivered and redistributed to the surrounding structural members, and when the surrounding structural members reach an equilibrium state as catenary action increases after a certain period of time, the collapse of structure does not occur. However, it has a risk of progressive collapse due to the effects of redundancy, integrity, continuity, ductility and load path redistribution which are the main structural characteristics to prevent progressive collapse[2].

Normally, the explosive demolition of RC frame structure carries out the process of removing or weakening columns, which are structural members, on the same floor by blasting, and then the entire collapse of structure is induced by all or partly repeating these process on many floors. Recently, the explosive demolition design of RC frame structure induces progressive collapse of the whole structure by removing or pre-weakening adjacent columns at appropriate delay time before the surrounding structural members reach an equilibrium

Table 1 Target structures for analysis.

Model	Number of floors	Column height [m]	Span length [m]
SL4_H3_5S	5	3	4
SL6_H3_5S	5	3	6
SL8_H3_5S	5	3	8
SL4_H3_10S	10	3	4
SL6_H3_10S	10	3	6
SL8_H3_10S	10	3	8
SL6_H4_10S	10	4	6
SL6_H5_10S	10	5	6

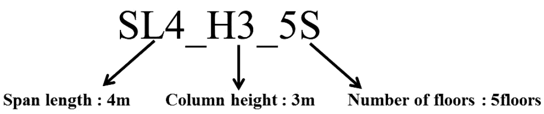
state due to the redistribution of load to occur the collapse behavior of the whole structure progressively due to initial collapse behavior[3].

In this study, the progressive collapse resisting capacity depending on the number of floors in the structure, height of column at the target floor for blasting, and changes in the span length among the explosive demolition design variables of RC frame structure was evaluated. The modeling of target structures for analysis and nonlinear dynamic analysis were carried out using the ELS (Extreme Loading for Structures) software[4] based on AEM (Applied Element Method)[3),5),6].

2. Progressive collapse analyses

2.1 Analysis model

The target structures for analysis are RC frame structures with 4 spans and 4 bays, and they are classified according to the number of floors, height of column and span length as shown in Table 1. An example of sign indicating an analysis model is shown as follows.



Each target structure for analysis was modeled in 3 dimensions according to the dimensions of concrete members and the arrangement of reinforcing bars shown in Figure 1. The same arrangement of reinforcing bar for columns and girders was applied to all analysis models, and the top bar of the slab was arranged each on both sides for 1.3 m, 2.0 m and 2.7 m which were approximately 1/3 of each span length according to the span length. Figure 2 shows the arrangement status of reinforcing bar of columns, girders and slabs in the SH4_H3_5S model. The material properties of concrete and reinforcing bars used in each analysis model were set as shown in Table 2. Table 3 shows the number of elements, the number of springs and the size of elements in each analysis model.

2.2 Analysis method

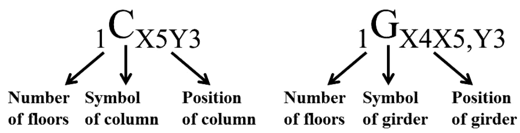
The nonlinear dynamic analysis was carried out for 8 analysis models with different number of floors, height of column and span length. As for the load combination for performing the dynamic analysis, Equation (1) suggested by GSA (General Service Administration) was applied[7].

$$\text{Load}=\text{DL}+0.25\cdot\text{LL} \tag{1}$$

where DL is the dead load, and LL is the live load.

The nonlinear analysis is performed in two stages including the static analysis stage and dynamic analysis stage. The static analysis stage is the stage to analyze the initial deformation of the whole structure from self-load before the removal of columns. The dynamic analysis stage is the stage to perform analysis according to the time increments of dynamic analysis, and the time increments of dynamic analysis was set to 0.001 second and the total analysis time was set to 5 seconds in this study.

An example of sign indicating columns and girders used an analysis model is shown as follows.



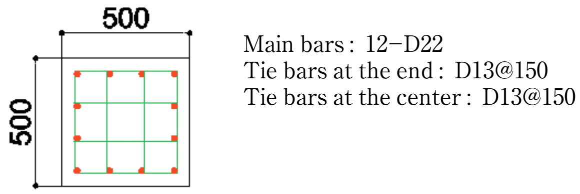
The target columns in each analysis model were 5 column elements (${}_1C_{X5Y1}$, ${}_1C_{X5Y2}$, ${}_1C_{X5Y3}$, ${}_1C_{X5Y4}$, ${}_1C_{X5Y5}$) in the X5 row on the first floor as shown in Figure 3, and these column elements were removed simultaneously for 0.01 second using the IER (Immaculate Element Method) technique.

The final collapse behavior of each analysis model with vertical and free fall displacements applied to the direct top elements of the removed columns (${}_1C_{X5Y3}$) were compared. To evaluate the progressive collapse resisting capacity, vertical displacements applied to the direct top elements of the removed columns compared with the vertical internal force applied to the adjacent columns (${}_1C_{X4Y3}$). Also, the normal stress of the top and bottom reinforcing bars acting on as per time between the girder (${}_1G_{X4X5,\,Y3}$) and the adjacent column (${}_1C_{X4Y3}$) connections was compared to evaluate the progressive collapse resisting capacity.

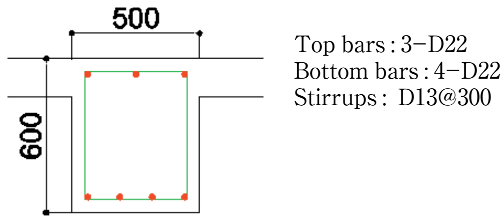
3. Analysis results and discussion

3.1 Final collapse results

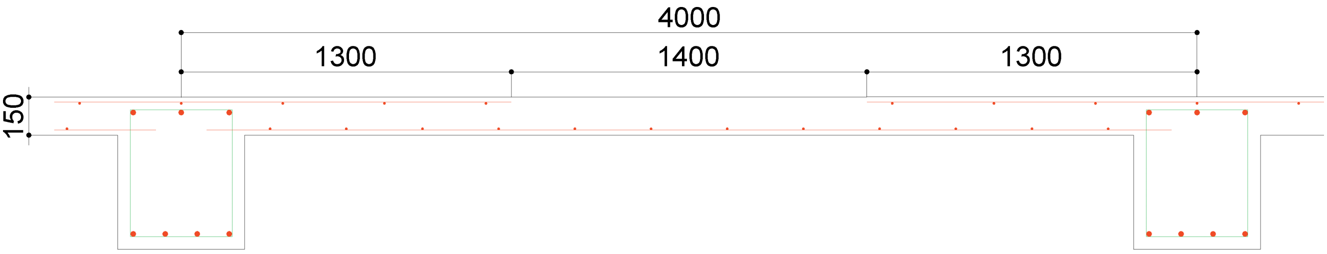
Table 4 shows the final collapse result of analysis model for each condition. No collapse occurred at the model with 4 m of span length, indicating that the model resisted progressive collapse due to the redistribution of load after columns are removed. Partial collapse occurred at the models of 5 floors with 6 m and 8 m of span length respectively. Entire collapse occurred at the model of 10 floors with 6 m of span length, regardless of height of columns. Figures 4 and 5 show the collapse behavior of SL 6_H3_5S model where partial collapse occurred and the SL 6_H3_10S model where entire collapse occurred as per time.



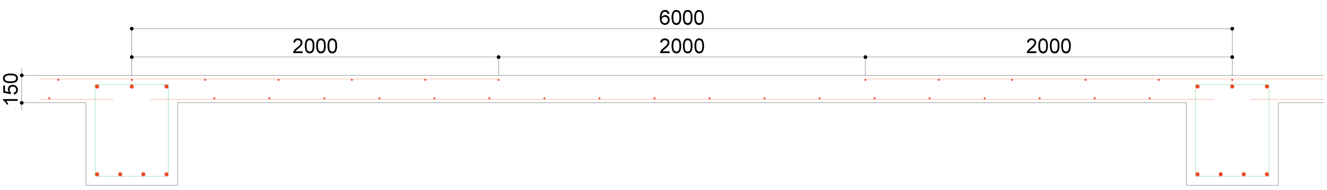
(a) Column



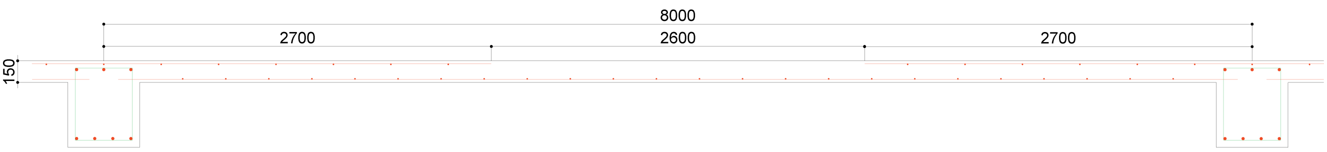
(b) Girder



Top bars : D10@400, bottom bars : D10@300
(c) Slab (in case of span length=4 m)



Top bars : D10@400, bottom bars : D10@300
(d) Slab (in case of span length=6 m)



Top bars : D10@400, bottom bars : D10@300
(e) Slab (in case of span length=8m)

Figure 1 Dimensions of concrete members and reinforcing bar arrangement drawing (unit : mm).

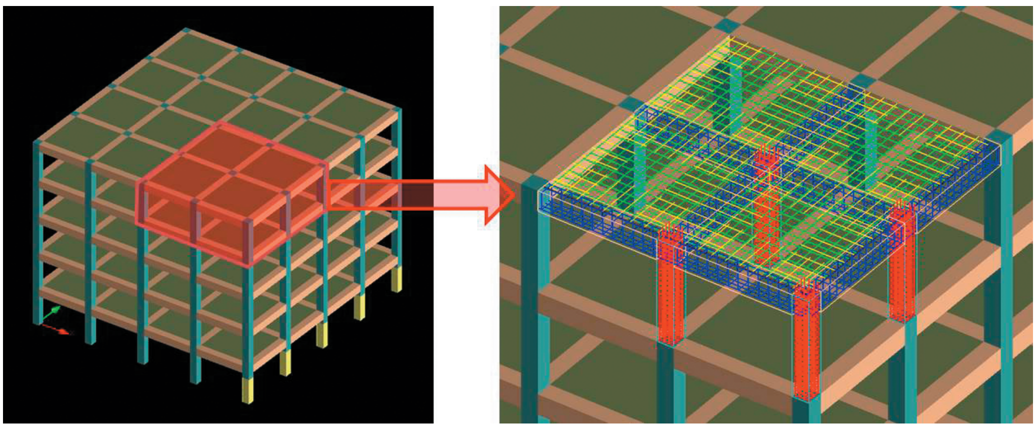


Figure 2 Arrangement status of reinforcing bar in SL4_H3_5S model.

3.2 Comparison between vertical displacement and free displacement

Each comparison result of ratio between the vertical displacement and free fall displacement applied to the direct top elements of the removed columns ($_1C_{X5Y3}$) as per time is as shown in Figure 6. In case the span length and height of column are same and only the number of floors is different (SL4_H3_5S and SL4_H3_10S, SL6_H3_5S and SL 6_H3_10S, SL8_H3_5S and SL8_H3_10S), there was no significant difference in the ratio between vertical displacement and free fall displacement as per time. It's because the load applied to the columns was redistributed to the adjacent structural members through an alternative path when the columns are removed, and the vertical internal force applied to the direct top of the removed columns becomes 0. In case the number of floor and the

Table 2 Material properties of concrete and reinforcing bars used in the analysis model.

Material	Young's modulus [GPa]	Compressive strength [MPa]	Tensile strength [MPa]	Tensile yield stress [MPa]	Ultimate strength [MPa]
Concrete	26.2	24.0	2.0	—	—
Reinforcing bar	200.0	—	—	360.0	504.0

Table 3 Properties of elements in each analysis model.

Model	Num. of tot. elements [EA]	Num. of tot. springs [EA]	Size of elements on 1 st floor [mm]		
			Column	Girder	Slab
SL4_H3_5S	15,707	952,180	250×250×300	250×250×300	250×250×75
SL6_H3_5S	15,707	1,003,806	250×250×300	393×250×300	393×393×75
SL8_H3_5S	15,707	1,022,734	250×250×300	536×250×300	536×536×75
SL4_H3_10S	21,652	1,312,575	250×250×300	250×250×300	250×250×75
SL6_H3_10S	21,652	1,348,431	250×250×300	393×250×300	393×393×75
SL8_H3_10S	21,652	1,381,439	250×250×300	536×250×300	536×536×75
SL6_H4_10S	21,652	1,354,531	250×250×400	393×250×300	393×393×75
SL6_H5_10S	21,652	1,354,531	250×250×500	393×250×300	393×393×75

Table 4 Final collapse patterns of the analysis models for each condition.

Model	Collapse pattern
SL4_H3_5S	Not collapse
SL6_H3_5S	Partial collapse
SL8_H3_5S	Partial collapse
SL4_H3_10S	Not collapse
SL6_H3_10S	Entire collapse
SL8_H3_10S	Entire collapse
SL6_H4_10S	Entire collapse
SL6_H5_10S	Entire collapse

height of columns were same (SL4_H3_5S, SL6_H3_5S and SL8_H3_5S, SL4_H3_10S, SL6_H3_10S and SL8_H3_10S), the vertical displacement became close to the free fall displacement as the span length increased, and in case the number of floor and the span length were same (SL6_H3_10S, SL6_H4_10S and SL6_H5_10S), the vertical displacement became close to the free fall displacement as the height of column increased. It indicates that the vertical displacement becomes close to the free fall displacement and the progressive collapse resisting capacity decreases[8]. And also, the progressive collapse resisting capacity was more significantly affected by the span length than the height of column.

3.3 Comparison between vertical displacement and vertical internal force

The result of comparison between the vertical displacement applied to the direct top of the removed columns ($_1C_{X5Y3}$) and the vertical internal force applied to the adjacent columns ($_1C_{X4Y3}$) in each analysis model is as

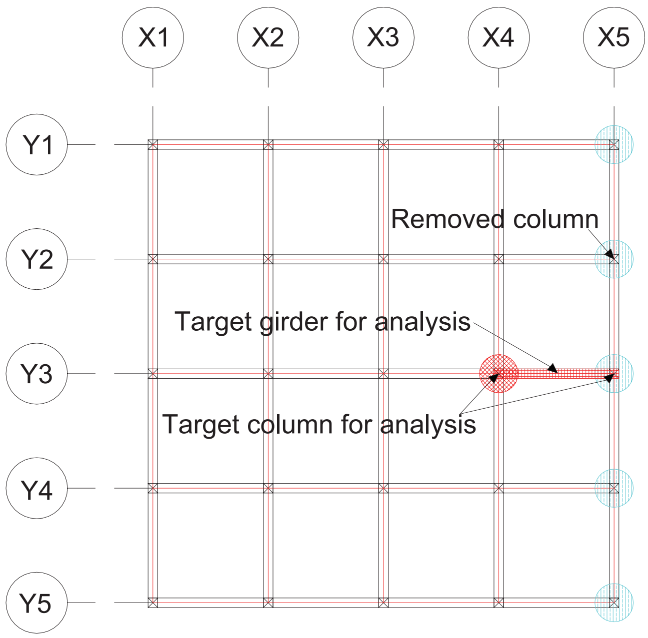
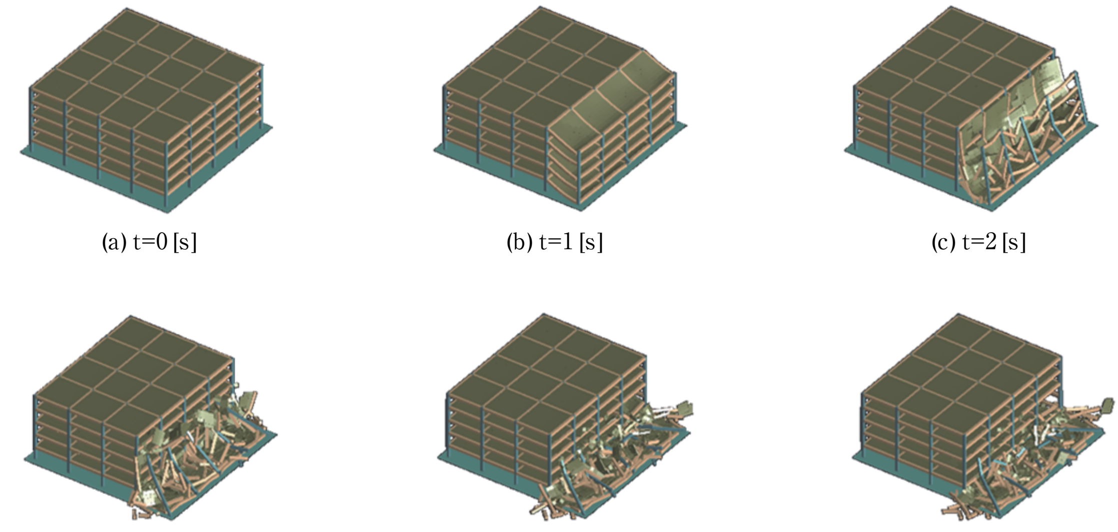


Figure 3 Position of target columns for removal and the position of target column and girder for analysis.

follows.

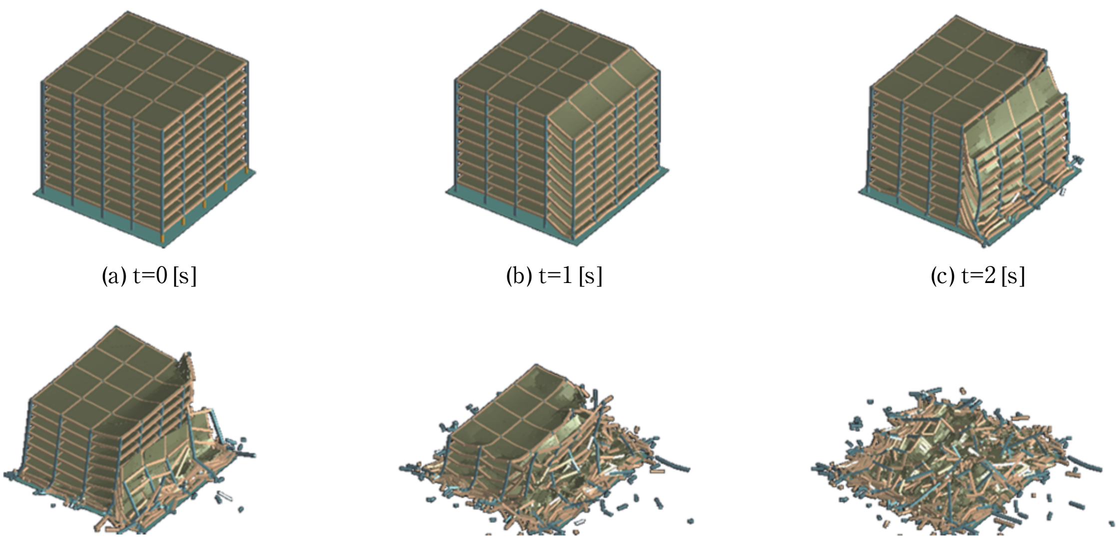
Figures 7 (a) and 7 (b) show the vertical displacement ratio and vertical internal force ratio at 5th floor and 10th floor in the structure with 6 m of the same span length and 3 m of the same height of columns as per time. In case of 5 th floor, the catenary action increased until 0.64 second and then decreased until 0.97 second. In case of 10th floor, the catenary action increased until 0.69 second and then decreased until 0.98 second. After 0.97 second and 0.98 second when the catenary action decreased, the vertical displacement started to increase drastically, showing the progressive collapse resisting capacity decreases. The increasing and decreasing times of catenary action



(a) t=0 [s] (b) t=1 [s] (c) t=2 [s]

(d) t=3 [s] (e) t=4 [s] (f) t=5 [s]

Figure 4 Collapse behavior (partial collapse) of SL6_H3_5S model as per time.



(a) t=0 [s] (b) t=1 [s] (c) t=2 [s]

(d) t=3 [s] (e) t=4 [s] (f) t=5 [s]

Figure 5 Collapse behavior (entire collapse) of SL6_H3_10S model as per time.

according to the number of floors were also similar, indicating that the time of load redistribution to the adjacent columns was almost same.

In case of Figures 7 (a) and 7 (c) where the height of column and the number of floors are same and the span length is 6 m and 8 m respectively, the time taken until the maximum vertical internal force was 0.64 second when the span length was 6 m, but when the span length was 8 m, the time taken until the maximum vertical internal force became shorter to 0.18 second. Also, in case of figures 7 (b) and 7 (d), the time taken until the maximum vertical internal force became shorter to 0.69 second and 0.18 second respectively. This indicates that there is almost no effect of catenary action and no progressive collapse resisting capacity in case the span length is 8 m. Therefore, the vertical displacement began to increase drastically from the beginning.

In comparison of the vertical displacement ratio and the vertical internal force ratio according to the height of columns in case of same span length and same number of floors (Figures 7 (b), 7 (e) and 7 (f)), the increasing time of catenary action decreased to 0.69 second, 0.48 second and 0.37 second respectively as the height of column increased. The duration of catenary action also decreased to 0.55 second, 0.31 second and 0.19 second respectively, and the duration of catenary action in case the height of column was 4 m and 5 m decreased by 43.6% and 65.5% respectively based on the case that the height of column was 3 m. This indicates that the progressive collapse resisting capacity decreases as the height of column increases.

Therefore, in order to induce progressive collapse of the whole structure at the time of the explosive demolition of RC frame structure, the detonation should be carried out at adjacent columns before the catenary action which is the time to apply the maximum vertical internal force decreases in consideration of the span length and the height of column.

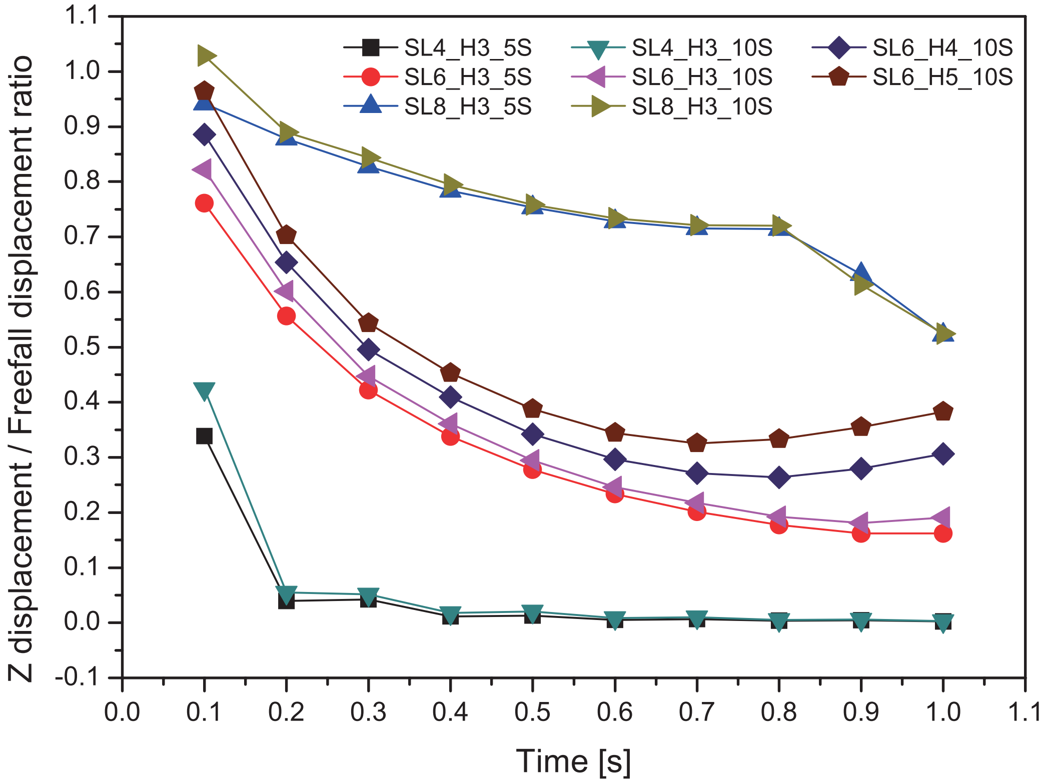


Figure 6 Comparison of ratio between vertical displacement and free fall displacement as per time according to each condition.

3.4 Comparison of normal stress

The comparison result of normal stress of reinforcing bars as per time applied to the girder ($_1G_{X4X5,\ Y3}$) between the removed columns and the adjacent columns and the adjacent column connections ($_1C_{X4Y3}$) as shown in Figure 8.

Figures 8 (a) and 8 (b) show the normal stress applied as per time to the top and bottom bars at 5th floor and 10th floor in the structure with6 m of the same span length and 3 m of the same height of columns. After the tensile failure of top bar occurred at 5th floor and 10th floor, the bottom bar was converted from the compression to the tension. Also, the time of tensile failure occurred at the top bar was almost similar as 0.98 second and 0.96 second respectively, and the time of catenary action decreased in the figures 7 (a) and 7 (b) was similar as 0.97 second and 0.98 second respectively. It's because the failure of girder and column connections occurred due to the tensile failure of top bars and the redistributed load path was lost so that the vertical internal force decreased.

The time of tensile failure of top bar occurred in case of figures 8 (c) and 8 (d) where the span length was 8 m was faster than the case of figures 8 (a) and 8 (b) where the span length was 6 m, and after the tensile failure of top bar occurred, the bottom bar did not converted to the tensile and failure occurred. This indicates that enough effect of catenary action on top bar did not occur so that there was almost no progressive collapse resisting capacity.

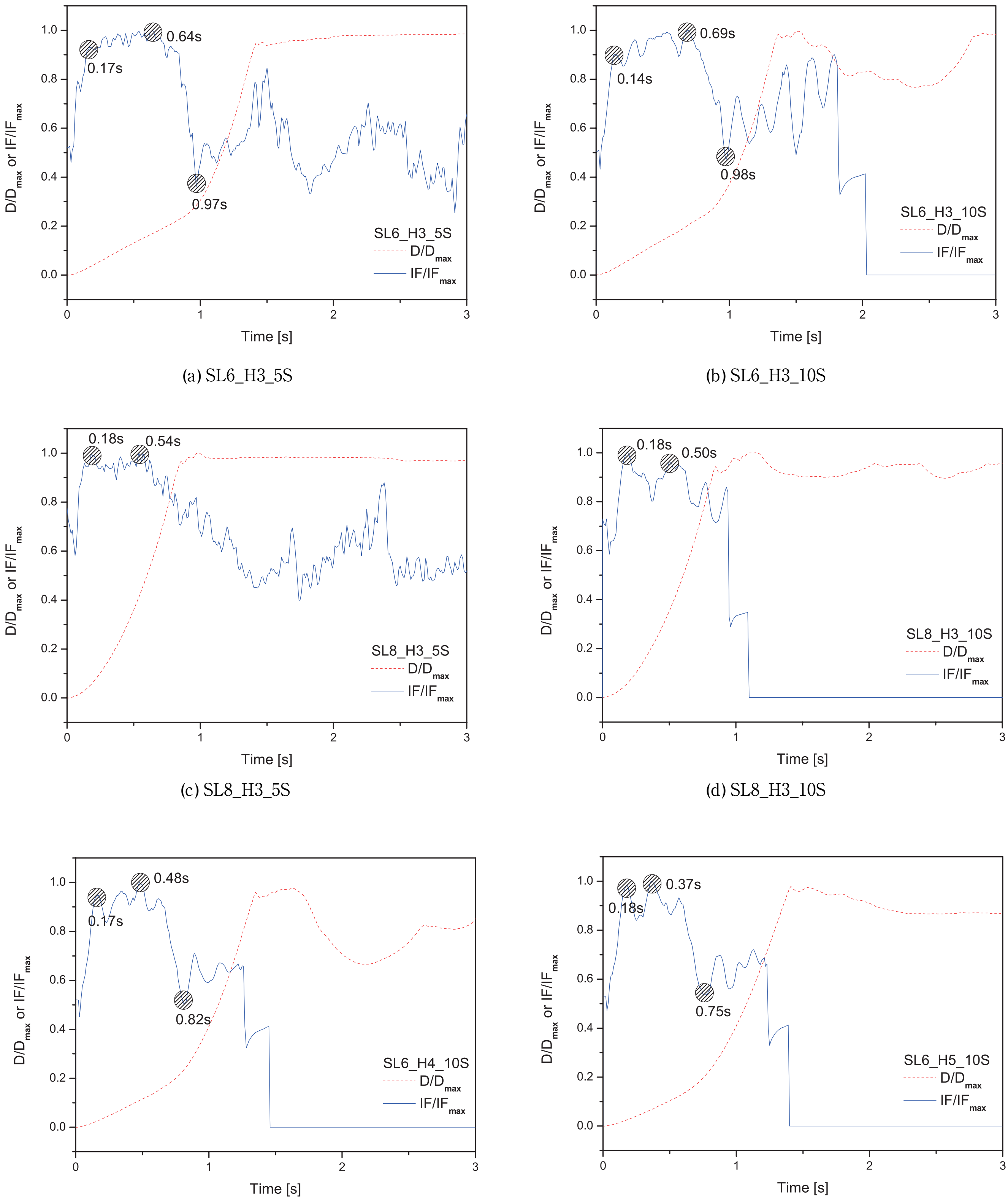
In comparison of normal stress applied to the top bar and bottom bar in case of 10th floors and the span length of 6 m (figures 8 (b), 8 (e) and 8 (f)), the time of tensile failure occurred at the top bar decreased as the height of column increased. Also, the time interval between the time of tensile failure occurred at the top bar and the conversion time of bottom bar to the tensile tends to decrease. Therefore, it is proven that the top bar has more influence on the progressive collapse resisting capacity than the bottom bar at the girder and column connections. Recently court disputes regarding to the failed explosive demolitions caused from miscontrolled local damage has been increased. Normally the full cost method which capitalizes the total cost causing from failed explosive demolition is adopted by old or large demolition company[9]. Successful effects method which covers only the cost in case of successful demolition is adopted by small scale company. In order to select reasonable method for an explosive demolition project, it is necessary to develop a reliable cost measurement method based on the state of art progressive collapse analysis shown in this study.

4. Conclusion

The result of nonlinear dynamic analysis performed to evaluate the progressive collapse resisting capacity depending on the number of floors in the structure, height of column at the target floor for blasting, and changes in the span length among the explosive demolition design variables of RC frame structure is as follows.

1) The models with 4 m of span length resisted progressive collapse regardless of its number of floors and the height of column, and partial collapse occurred at the models with 5 floors and entire collapse occurred at the models with 10 floors.

2) As the span length and the height of column increase, the vertical displacement becomes close to the free fall displacement and the progressive collapse resisting capacity decreases. And also, the progressive collapse



(a) SL6_H3_5S (b) SL6_H3_10S

(c) SL8_H3_5S (d) SL8_H3_10S

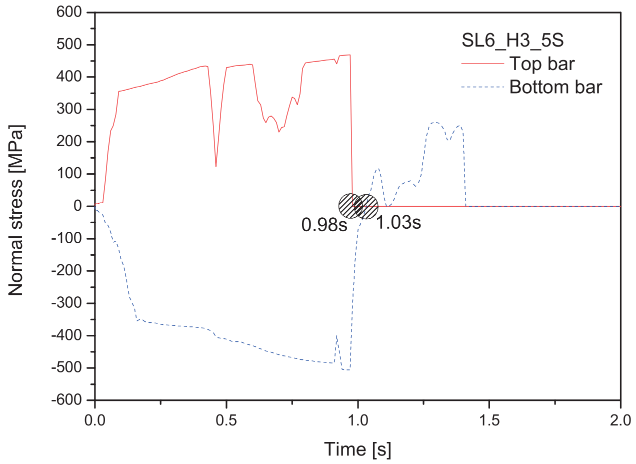
(e) SL6_H4_10S (f) SL6_H5_10S

Figure 7 Comparison between vertical displacement ratio and vertical internal force ratio as per time according to each condition.

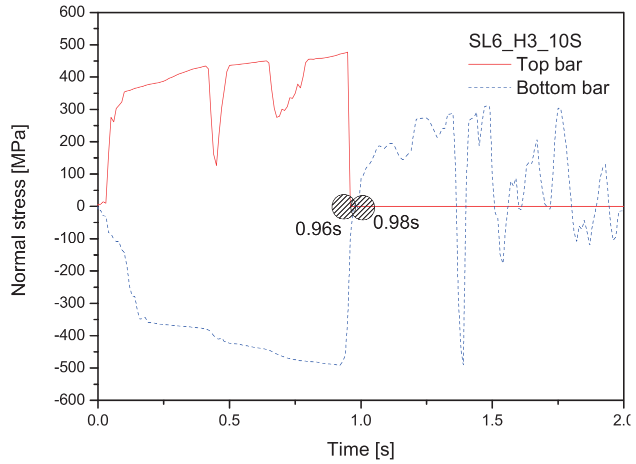
resisting capacity was more significantly affected by the span length than the height of column.

3) The increasing and decreasing times of catenary action according to the number of floors were also similar, indicating that the time of load redistribution to the adjacent columns was almost same. As the span length increases, the time taken to the maximum vertical internal force decreases drastically, and this indicates that there is almost no progressive collapse resisting capacity. Also, as the height of column increased, the increasing time and duration time of catenary action decreased, and this indicates that the progressive collapse resisting capacity decreases.

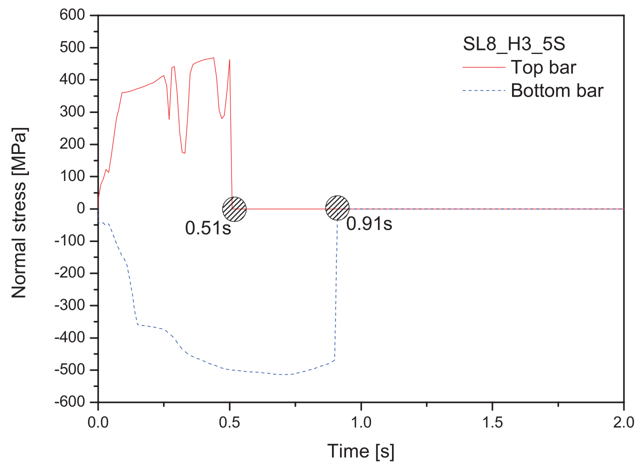
4) As the span length and the height of column increase, the time that tensile failure of top bar occurred at the girder and column connections becomes shorter, and the



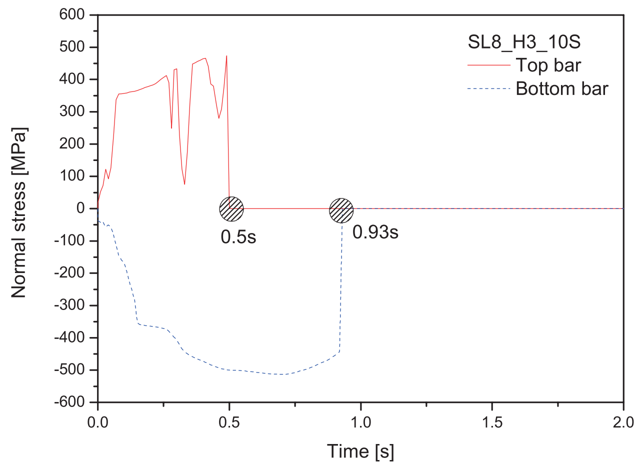
(a) SL6_H3_5S



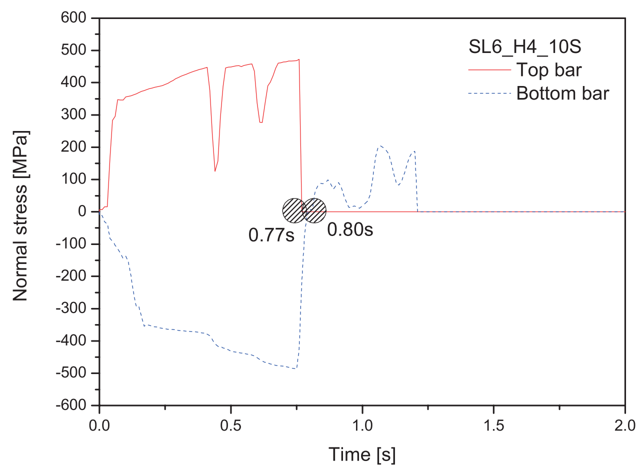
(b) SL6_H3_10S



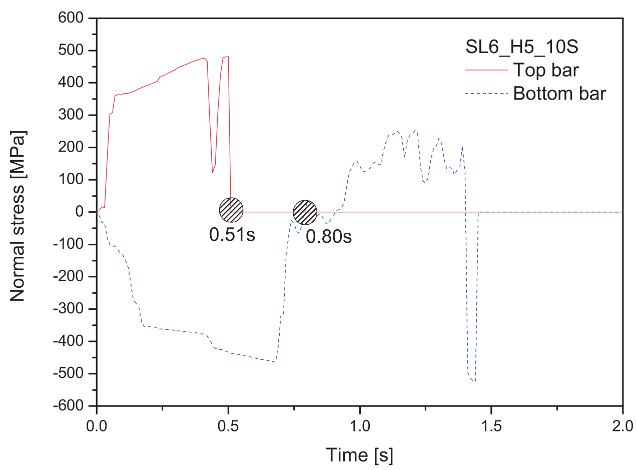
(c) SL8_H3_5S



(d) SL8_H3_10S



(e) SL6_H4_10S



(f) SL6_H5_10S

Figure 8 Comparison of normal stress between top bar and bottom bar as per time according to each condition.

top bar has more influence on the progressive collapse resisting capacity than the bottom bar.

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References

1) ASCE, "ASCE Standard ASCE/SEI7-05, Minimum Design Loads for Buildings and Other Structures", American

Society of Civil Engineering, Reston, Virginia, USA.
2) C. Baciu and M. Lupoae, Constructii Review, 13, 51−61 (2012).
3) H. Park, C. G. Suk, Y. H. Ko, W. J. Jung and S. H. Cho, Proc. 6 th Symposium on Blasting Techniques, 145−152, Hanwha Corp., Seoul (2012) (in Korean).
4) ASI, "Extreme Loading for Structures Theoretical Manual", ASI (2010).
5) H. S. Tagel−Din, "High Fidelity Modeling of Building Collapse with Realistic Visualization of Resulting Damage and Debris Using the Applied Element Method", 75−76, ASI (2009).
6) H. Park, C. G. Suk, Y. H. Ko, and S. H. Cho, Sci. Tech. Energetic Materials, 74, 166−176 (2013).
7) GSA, "Progressive Collapse Analysis and Design Guidelines", 2−9, GSA (2003).
8) M. Loizeaux and A. Osborn, J. Performance of constructed Facilities, 20, 391−402(2006).
9) R.L. Watts and J. L. Zimmerman, "Positive Accounting Theory", 244−250, Prentice−hall (1987)