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RESPONSE OF MULTI-STORY FRAMES AND INDIVIDUAL COLUMNS TO BLAST LOADING

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ABSTRACT

Columns are critical members in maintaining the structural integrity of building structures under any type of extreme loading. This is more so when the building is subjected to blast loading, since in most cases the columns on the front face of the building experience high blast demands. Response of these members to blast loading is traditionally analysed by modeling the columns as isolated elements with fixed or pinned ends. Therefore, an investigation is carried out to determine how much the ends of the columns displace in reality, and how such displacement affects the response of the columns. For this purpose, a selected two-dimensional 10-story reinforced concrete frame is subjected to two different blast load scenarios. The response of the columns in the multi-story frame is compared to the response of the same columns under similar loading when modeled as isolated columns. The comparison shows that for high blast loads, the response of an isolated column is much larger than the response of the same column when it is treated as a part of the multi-story reinforced concrete frame (global model). When the blast loads are modest, the difference between the response of an isolated column and when it is part of a multi-story frame is not large, although isolated column response is still higher. Therefore, it can be concluded that in a new design, response analysis of isolated column model would provide acceptable, but conservative results. When evaluating existing buildings, however, a global analysis of the multi-story frame may be necessary to obtain realistic results.

Keywords: Blast loading, structural response, reinforced concrete column, moment frame building, finite element analysis.

1. INTRODUCTION

The vulnerability of building structures to blast loading is traditionally assessed based on the response analyses of individual elements. In such analyses, it is usually assumed, especially for the columns, that the ends of the element do not displace in the lateral direction during loading. Thus, the member being analysed is considered as fixed-fixed, pinned-pinned or fixed-pinned (DoA 1986, DoD 2008, ASCE 2010). There has not been much research on the response of columns when they are part of a whole frame or building structure under blast load. However, from the few research studies that are available, it can be observed that the global response of the building structures to blast loads can be quite significant, leading to large inter-story drifts (Nourzadeh et al. 2015, Nourzadeh 2017).

The large inter-story drifts produced in the building structures by blast loading imply that the assumption about lateral fixity of the columns in individual modeling can be inaccurate. This fact questions the accuracy of the results of the analysis of such members when they are modeled as individual members. Therefore, in this paper the accuracy of an analysis of columns when modeled as individual members is tested by means of a selected series of numerical simulations.

A benchmark 10-story building structure is selected for the analysis of its response to two different blast scenarios. To create an analytical model of the building structure and of individual column, OpenSEES software, which is capable of nonlinear modeling of structural systems in dynamic loadings, is used. The building is simulated using a two-dimensional (2D) model, since the accuracy of this assumption has been verified in a previous study (Nourzadeh et al. 2015, Nourzadeh 2017). The 2D model of the structure is subjected to two different (moderate or high) levels of blast loading. In addition, the firststory columns of the building are modeled individually and their response to the same set of loadings determined. The results of the global and individual models are compared to study the differences between the analysis methodologies.

2. GEOMETRICAL MODEL

A schematic view of the geometry of the building models used in the numerical analysis is shown in Figure 1. In earlier studies (Saatcioglu et al. 2009, Nourzadeh et al. 2015, Nourzadeh 2017), the building was designed for the earthquake hazard in Ottawa, for moderate ductility capacities per the provisions of the National Building Code of Canada (NRC 2010) and the Canadian Standard for the Design of Reinforced Concrete (CSA 2014). In detailed response analysis of the building for the design level earthquake loads, it was confirmed that the moment frame building could satisfy both strength and deformation requirements of the design code (Nourzadeh 2017). Therefore, the current model is assumed to be able to represent a satisfactory and valid benchmark structure.

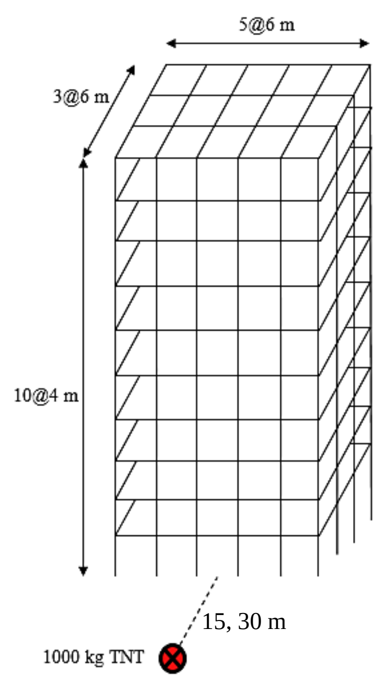


Figure 1. Geometrical model of the building and the blast source (Nourzadeh et al. 2015)

For determining the global response of the structure, the building is analysed using a 2D model. Such modeling is justified because, as shown in Figure 1, the building and the location of the blast charges are both symmetrical. Also, in a previous research (Nourzadeh et al. 2015, Nourzadeh 2017), the results of the 2D models were shown to closely match the results obtained from a 3D model. Since the blast load is perpendicular to the long face of the building, the 2D model comprises the frames along the shorter dimension interconnected by rigid links. Also, only 3 of the 6 frames are modeled to take advantage of the symmetry. The nodes at the base of the building structure are restrained along all degrees of freedom representing fully fixed supports.

Since the discretization of the elements (degrees of freedom) used in the numerical models is important in determining the response of the structures to blast (Nourzadeh 2017, El-Dakhakhni et al. 2009), each beam and column in the model is divided into four sub-elements. The mass of a story, including the self-weight of the members and the mass of a 150-mm concrete slab, is represented by lumped masses at the end nodes of the sub-elements in the beams. For higher accuracy in the response analysis of the critical structural elements, the first and second-story columns are discretized into 16 sub-elements (15 translational degrees of freedom). For individual modeling of these columns, the same 15 degrees of freedom (DOF's) are selected to conform with the global analysis model.

3. BLAST LOADING

As shown in Figure 1, 1000 kg of TNT is selected as the blast source in the analysis. The explosive charge is located at two different standoff distances from the building: at 15 m (B1 scenario) and at 30 m (B2 scenario) from the centerline of the building structure measured parallel to the short side. The blast loads are applied to the beam-column joints and to the mid-height nodes of the columns based on each node's range and incident angle, except for the columns in the first two stories where all 15 nodes of the columns are loaded based on the range and incident angle of the mid-height of the stories. Different times of arrival and time variations are considered in calculating the blast loads at each node. The blast pressures are multiplied by the tributary area of the node to obtain the blast force acting on it. The time histories of the blast pressure for the positive phase are obtained from ConWep.

To ensure that the responses of the individual columns models are comparable to those obtained from the global model, the same loading as used in the global model, that is based on the range and incident angle of the mid-height of the columns is applied to the internal DOF's. Three columns in the first story, as shown in the plan of the building in Figure 2, are loaded based on the blast load characteristics given in Table 1 and 2, for the two selected blast scenarios. In these tables the fictitious duration of the blast wave is determined by idealizing the actual wave shape by a triangular variation and equating the impulse under the triangular variation to that under the actual reflected wave. The latter is obtained from ConWep.

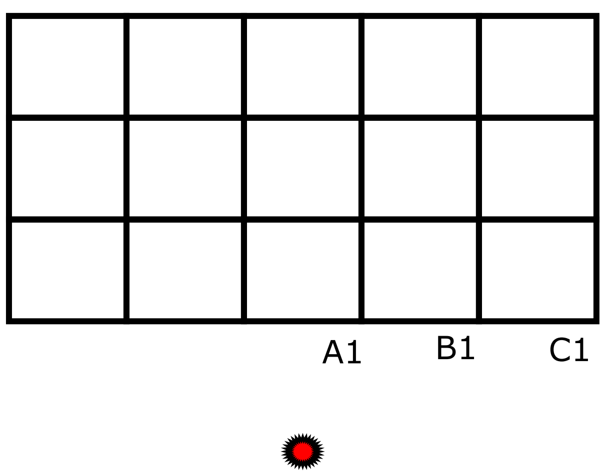


Figure 2. Location of the selected columns for individual models in the building

Table 1. Properties of the blast load applied on a first story column in Blast Scenario 1 (1000 kg TNT at 15 m Standoff)

Column	A1	B1	C1
Range (m)	15.4	17.6	21.6
Incident angle (^{0})	13.5	31.6	45.3
Time of arrival (msec)	10.4	13.4	19.1

Reflected pressure (kPa)	2266.2	1388.7	682.1
Fictitious duration (msec)	4.3	5.3	7.5

Table 2. Properties of blast load applied on a first story column in Blast Scenario 2 (1000 kg TNT at 30 m Standoff)

Column	A1	B1	C1
Range (m)	30.2	31.4	33.6
Incident angle (^{0})	6.9	17.1	26.8
Time of arrival (msec)	42.3	45.1	50.3
Reflected pressure (kPa)	180.0	164.2	140.4
Fictitious duration time (msec)	15.0	15.3	15.9

4. DYNAMIC ANALYSIS AND RESULTS

4.1. Analysis Description

For both global and individual numerical models, displacement-based elements are used to obtain the full extent of response. The displacement-based elements are meshed and divided as discussed earlier, and each sub-element is assigned 3 integration points. To account for large deformations in the structure, the P-Delta effect on the columns is considered in the analysis. The gravity load is first applied to the structure as a static load and a linear analysis of the structure is carried out to obtain the deformations. The rest of the analysis is carried out on the deformed condition of the structure. The same level of gravity loads is also applied to the columns in individual models.

Dynamic analyses are carried out for the two different blast load scenarios mentioned above. The dynamic analysis is performed by means of step-by-step integration using Newmark's average acceleration algorithm. Mass and stiffness proportional Rayleigh damping is considered in the analysis (Humar 2012). A damping coefficient of 5% is selected for modes 1 and 4. The nonlinear iterative procedure is implemented using modified Newton-Raphson methodology. The analysis time step was varied from 0.1 msec. to 0.001 msec. depending on the number of iterations required to achieve convergence, leading to a high computation effort.

4.2. Results

The lateral displacements at column mid-heights and at story levels produced by B1 scenario obtained from the global structural model are shown in Figure 3. As can be seen in the figure, at the time that the columns are experiencing deflections, their ends (story level) are also displacing laterally, which is inconsistent with the fixity assumption in the traditional individual column modeling.

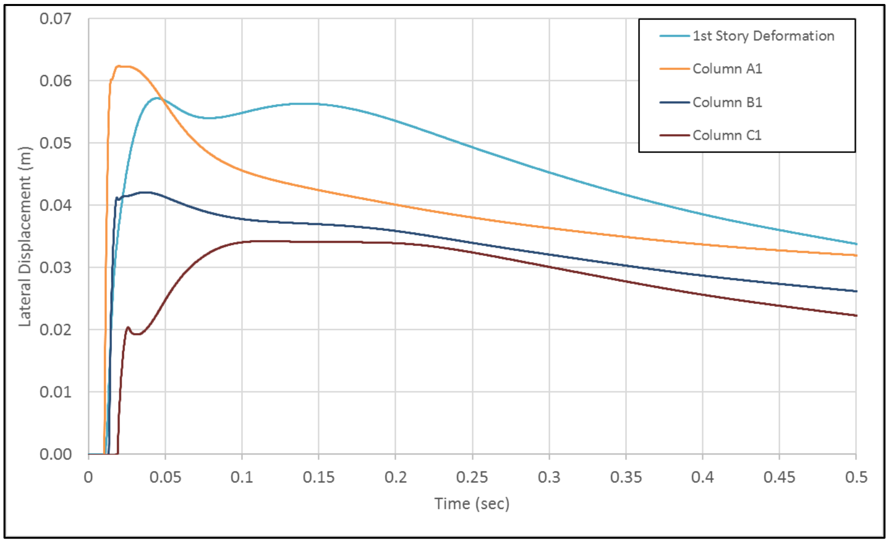


Figure 3. Lateral displacements at mid-heights of columns and the first story (columns end) deformations in the global model in B1 scenario

To compare the response of the columns as part of the global model and as individually modeled, the maxima of the lateral displacements at the mid-height of first story columns are presented in Table 3 for B1 scenario. As shown, there is a very significant difference between the response of columns in individual and global models in this scenario. The columns in individual models are at a collapse level of response, while in the global model the chord angle of these columns barely exceeds 2 degrees.

Table 3. Comparison of lateral deflections at mid-height of individual columns to that when they are part of the global model in B1 scenario

Deflection location	Column A1		Column B1		Column C1		Story 1 level deflection
Model	Individual	Global	Individual	Global	Individual	Global	
Max. lateral displacement (mm)	1013.9	71.3	657.8	48.2	156.8	38.1	62.1

The differences between the response of columns in global and individual models in B2 scenario are not as significant as in B1. In this smaller blast scenario, in which both models remain in the elastic response level, the mid-height deflections of the columns in both models are close as shown in Figure 4. However, as can be seen in the figure, the individual models still yield larger and more conservative results.

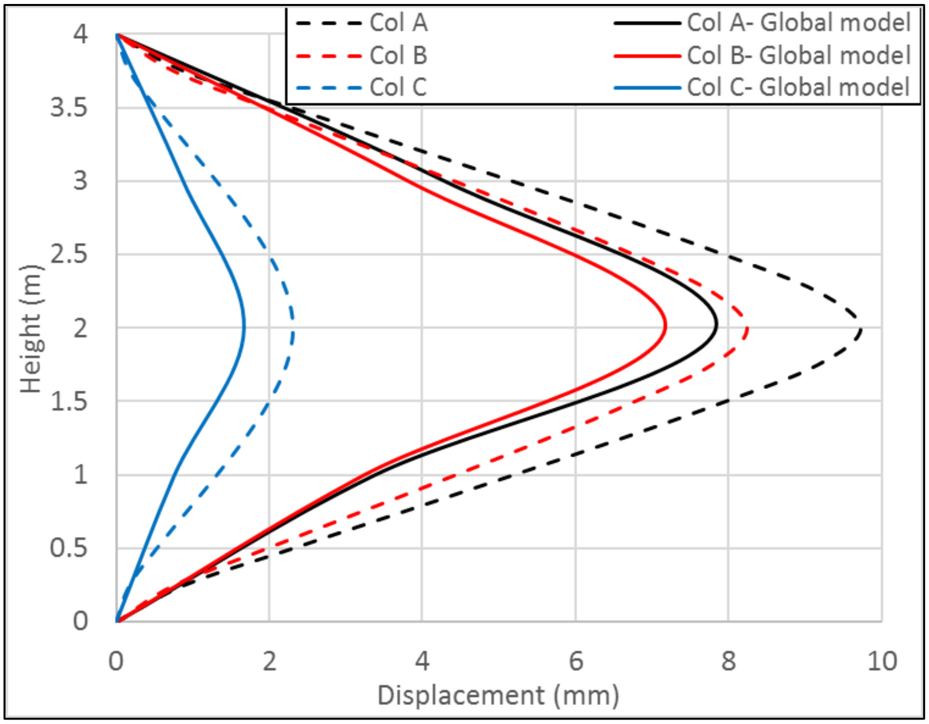
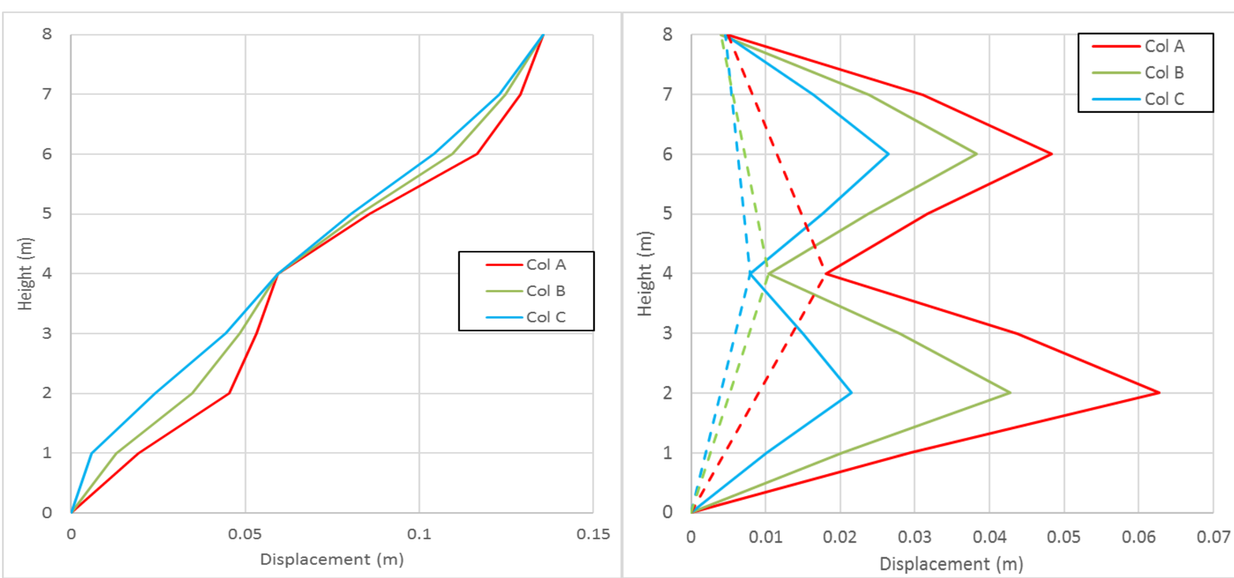


Figure 4. Comparison of deformed shape of peak 1st-story column chord displacements and deflections in single columns modeled individually in B2 scenario

Another important observation from the results of these simulations is that the maximum deflections at the column mid-height and at story level do not occur at the same time. As observed from Figure 3, at the time the mid-height deflections are at the peak in the B1 scenario, the story level deflection is less than 30 mm, which is well below the peak story drifts. The peak column mid-height deflections occur in the early stages of the response (for instance 14.6 msec for Column A1). On the other hand, the maximum deflection at the first story level occurs at 140 msec in the analysis. The deflected shapes of the first and second story columns at the time the first story drifts peaks are presented in Figure 5(A), while the deformed shapes of the columns at the time the mid-height deflections peak are shown in Figure 5(B).



(A) (B)

Figure 5. Deformed shape of 1st and 2nd story columns in global model subject to B1 scenario loads at the time (A) the 1st story deflection peaks, and (B) when the mid-height deflections in 1st story columns peak (dashed lines are chord deflections which are deduced from total deflections of columns to be compared to the peak deformations in individual columns)

5. CONCLUSION

In the traditional practice, it is generally assumed that the response of individual columns on the front face of the building, rather than the global response is more critical. It is further assumed that the individual columns can be analysed for their response by using a model of the isolated column in which the boundary conditions are assumed to be some combination of fixed and pinned, but laterally restrained.

In the numerical analysis presented in this paper, it is shown that for high blast loads, the response of an isolated column is much larger than the response of the same column when it is treated as a part of the global model. This is mainly on account of the inelasticity introduced in the columns, and to some extent because of the assumption of lateral restraint at the boundaries of the column. When the blast loads are modest, so that the column remains elastic the difference between the response of an isolated column and when it is part of the global model is not large, although isolated column response is still conservative.

In a new design the columns will be sized so as not to deform into the inelastic range under the expected blast load. This is to safeguard the vertical gravity carrying components against damage. In this situation, response analysis of isolated column model would provide acceptable, although somewhat conservative results. On the other hand, when an existing building is being evaluated for its resistance to blast, a global analysis may be necessary to obtaining realistic results, particularly when the isolated column analysis shows that the column becomes inelastic. Global analysis is also useful to verify that the story drifts are within acceptable limits and that P-Δ effects are not critical.

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