ANALYSIS OF BLAST LOADING EFFECTS ON ELEMENTS OF REINFORCED CONCRETE BUILDINGS

D.D. Chamara, L.K.S. Priyadarshani and P.A.K. Karunananda[1]

Department of Civil Engineering, The Open University of Sri Lanka

INTRODUCTION

A bomb explosion within or immediately nearby a building can cause catastrophic damage on the building's external and internal structural frames, collapsing of walls, blowing out of large expanses of windows, and the shutting down of critical life safety systems. Loss of life and injuries to occupants can result from many causes, including direct blast effects, structural collapse, debris impact, fire and smoke. The indirect effects can combine to inhibit or prevent timely evacuation, and thereby contribute to additional casualties. Hence, buildings used by the general public daily must also have satisfactory blast protection. Integrating blast design in-to existing norms for structural design is a challenge, but it is achievable. The consideration of damage of structural components is limited and this is important for assessing the vulnerability of buildings against blast loadings (Draganić 2009).

The analysis and design of structures subjected to blast loads require a detailed understanding of complex blast phenomena and the dynamic response of various structural elements. This study is aimed at modeling and assessing an existing building using a numerical approach. Such a modeling is essential to visualize the structural response of a structure against blast effects and to propose remedial measures to strength the structure.

METHODOLOGY

The main objective of this study is to evaluate blast effects on elements of reinforced concrete (RC) buildings, considering experimentally determined dynamic characteristics. The study consists of three phases; (1) A literature survey on blast loading, (2) The theoretical calculation of the characteristics of blasting effects according to the Unified Facilities Criteria (UFC 2008), and (3) The numerical modal analysis involving non-linear time history analysis carried out on a three-story reinforced concrete building using the SAP2000 v15 general purpose software package.

Figure 1(a) shows a view of the building used in this study, which is the Open University of Sri Lanka (OUSL) CRC building, that was constructed in 2013. It consists of lecture rooms and some offices of the OUSL and lies beside the Nawala-Nugegoda road.

Columns and beams were modeled as frame elements while the slabs were modeled as shell elements. Dimensions of the building are shown in Figure 2. A charging explosive material was assumed to be placed R (m) away from the building in the Nawala-Nugegoda road side. The stand-off distance (R) was considered to be 13.7m (45') and 27.4m (90'). The charging material was TNT and weight (W) varied from 50kg, 100kg and 200kg (Figure 2). Therefore, six blasting scenarios were created. The nature of loading was dynamic and the theoretical calculation was performed using the Unified Facilities Criteria of the United States Army (UFC 2008), and the pressure loading on the four sides of the building that was obtained are shown in Figure 3. Table 1 shows the specimen calculation for charging a weight of 50-kg TNT and a stand-off distance of 13.7m (45').

[1] Correspondence should be addressed to Dr. P.A.K. Karunananda, Department of Civil Engineering, Open University of Sri Lanka (email: pakar@ou.ac.lk)

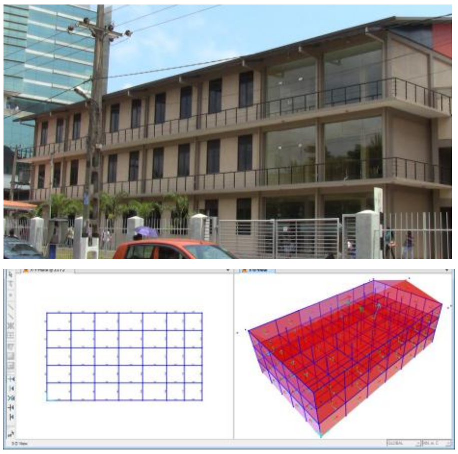

Figure 1(a): A view of the CRC building;
(b) SAP 2000 model of the building.

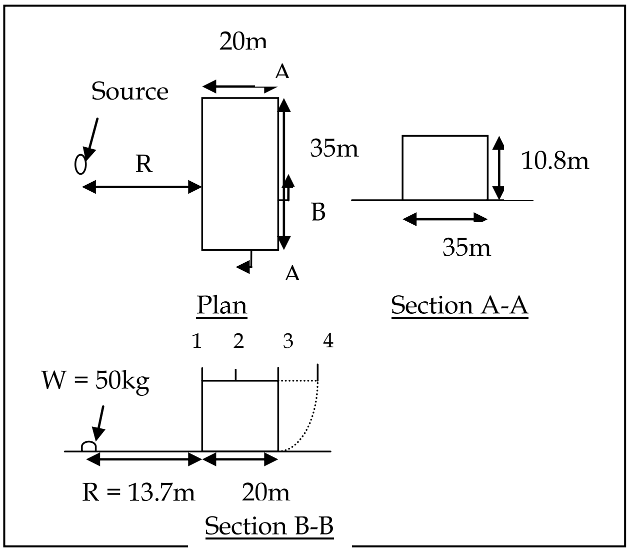

Figure 2: Sectional dimensions of the CRC building

Table 1: Specimen results for W = 50kg, R= 13.7m (45ft)

Loading face	Pressure (kNm^{-2})	Time (ms)	Loading face	Pressure (kNm^{-2})	Time (ms)
Front wall loading (1)	211.326	0	Roof loading (3)	0	0
	0	5.313		16.155	6.191
	0	11.719		0	11.719
	-20.690	23.642		0	39.141
	0	55.877		-10.086	29.603
				0	77.957
Side wall loading (2)	0	0	Rear wall loading (4)	0	0
	6.717	12.025		3.848	16.916
	0	43.819		0	51.818
	0	15.846		0	17.833
	-4.303	33.455		-2.538	35.030
	0	81.065		0	81.523

Figure 3 shows the pressure distribution over four sides of the building. This loading was applied to the finite element model (FEM) model of the building. The nature of loading was dynamic, and therefore a dynamic time history analysis was performed. Table 2 shows the peak reflected over pressures for different W-R combinations.

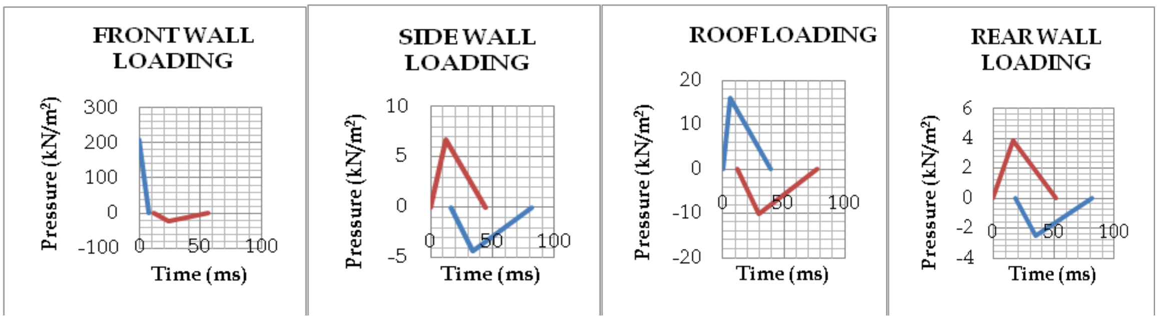

Figure 3: Pressure distribution over four walls of the building

Table 2: Peak reflected overpressures P_r (in kNm^{-2}) with different W-R combinations

W / R (m)	50kg TNT	100kg TNT	200 kg TNT
13.7	211.326	421.931	705.931
27.4	52.524	63.840	95.421

Figure 4 shows a frame labeling node diagram of the front frame of the building according to the SAP2000 model.

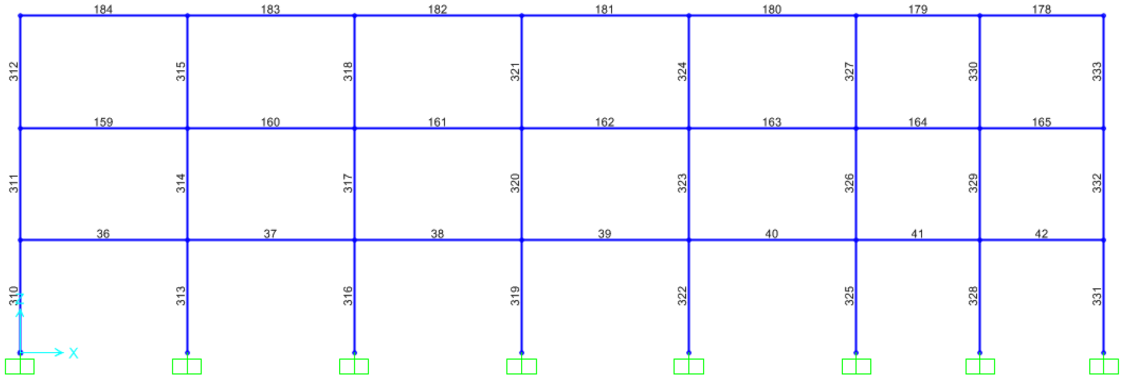

Figure 4: Node diagram for front structural frame

Calculated pressures-time history was applied to the SAP2000 FEM. A design check was performed using the ACI 318-05 code, which considers the plastic hinge formation in the failure. The deformed shape of the building due to W=50kg and R=13.7m is shown in Figure 5. According to the analysis results, six of the beams (Section Nos: 178,179,180,182, 183 and 184) failed due to the combined dead loads, live loads and blast loads, and no further critical columns were identified.

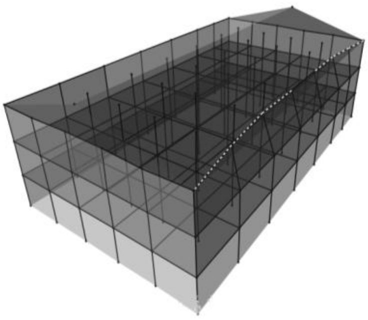

Figure 5: Deformed shape of building due to W=50kg, R=13.7m

The section No. 178 beam was a very critical element. The resultant beam design data of beam 178 with length is tabulated below;

Table 3: Resultant element forces on beam 178

Beam	Length m	Step Type	P kN	V_2 kN	V_3 kN	T kN-m	M_2 kN-m
178	0	Max	4.69	0.22	1.48	42.39	18.09
178	0.5	Max	4.69	0.22	1.48	42.39	18.98
178	1	Max	4.69	0.22	1.48	42.39	19.87
178	1.5	Max	4.69	0.22	1.48	42.39	20.76
178	2	Max	4.69	0.22	1.48	42.39	21.65
178	2.5	Max	4.69	1.39	1.48	42.39	22.54
178	3	Max	4.69	2.72	1.48	42.39	23.44
178	3.5	Max	4.69	4.04	1.48	42.39	24.33
178	4	Max	4.69	5.37	1.48	42.39	25.22
178	0	Min	-9.51	-5.23	-1.82	-26.05	-8.13
178	0.5	Min	-9.51	-3.91	-1.82	-26.05	-8.82
178	1	Min	-9.51	-2.58	-1.82	-26.05	-9.51
178	1.5	Min	-9.51	-1.26	-1.82	-26.05	-10.20
178	2	Min	-9.51	-0.17	-1.82	-26.05	-10.89
178	2.5	Min	-9.51	-0.17	-1.82	-26.05	-11.58
178	3	Min	-9.51	-0.17	-1.82	-26.05	-12.27
178	3.5	Min	-9.51	-0.17	-1.82	-26.05	-12.96
178	4	Min	-9.51	-0.17	-1.82	-26.05	-13.65

Notation: P = Axial force T = Torsion V2 =Shear force in 1-2 plane
M2 =Bending moment in the 1-3 plane (about the 2-axis) *V3* =Shear force in 1-3 plane

RESULTS AND DISCUSSIONS

There is a significant lateral deformation of front columns at the 2nd floor level. Further, the beam tie beam arrangement at roof level is beyond the capacity, and resulted in a structural failure through the plastic hinge formation that would be imminent under such a blast loading.

The Finite element analysis results for the CRC building's stage II building, due to a blast explosion, with different charge weights (W) with various stand-off distances (R) can be summarized as below:

Table 4:Summary of analysis results

Charge Weight (kg)	Stand Off Distance (m)	Number of Failure Object		
50	13.7 (45')	0	Column	all members pass
		6	Beam	178/179/180/182/183/184
	27.4 (90')	0	Column	all members pass
		0	Beam	all members pass
100	13.7 (45')	3	Column	318/321/324
		6	Beam	178/179/180/182/183/184
	27.4 (90')	0	Column	all members pass
		5	Beam	178/179/180/182/184
200	13.7 (45')	5	Column	315/318/321/324/327
		1	Beam	176/177/178/179/180/181/182/183/184/185/186
	27.4 (90')	0	Column	all members pass
		6	Beam	178/179/180/182/183/184

CONCLUSIONS AND RECOMMENDATIONS

A blast load for a near-by explosion was determined and simulated on an FEM building model using SAP2000, a general purpose software package. Loading was defined as a non-linear pressure-time history. It revealed that the model building was not capable of withstanding the given blast loading, and hence a partial collapse of the building occurs. Based on these findings, it is recommended that the guidelines on abnormal load cases, such as blast loadings and provisions on progressive collapse prevention, should be included in the current Building Regulations and Design Standards.

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