

# 폭발하중을 받는 강구조 중층 건물의 응답 및 해석

## Three Dimensional Responses of Middle Rise Steel Building under Blast Loads

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### 요지

최근 들어 여러 테러에 의한 폭발사건에서 유발된 위험상황에서 보듯이 폭발에 의한 인명피해나 시설물의 손상은 우리가 고려하는 재해수준을 넘는 비참한 결과를 항상 수반한다. 하지만 폭발에 대한 구조물의 설계는 그 연구나 대책이 상당히 미비한 실정이다. 이에 미국건물설계기준(UBC94)을 바탕으로 내진설계(Welded Moment Resistant Frame)된 10층 건물의 폭발에 대한 해석적 모델을 제공하고자 한다. 현재 폭발하중의 정량적인 결과는 미국 육군(U.S.Department of Army)에서 개발된 경험적 방법에 기반을 둔 프로그램을 통해 폭간거리에 따른 하중의 크기와 분포를 알 수 있다. 본 연구에 사용된 폭원의 성격은 반구형 표면 폭발(Hemispherical Surface Burst)의 경우를 사용하였으며, 또한 선형 및 비선형 시간 이력해석을 통해 건물의 변위, 상대변위, 요구/수행비 및 비선형 거동에 대한 해석적 결과를 제공하였다. 또한 현재 사용되고 있는 내진기준(FEMA356)에 적용하여 소성현지의 거동을 통해 폭발에 대한 건물의 성능수준을 예상하였다.

**핵심용어 :** 폭발하중, 모멘트프레임, 반구형 표면폭발, 층간상대변위, 요구/수행 비

### Abstract

It has been suggested that buildings designed for strong ground motions will also have improved resistance to air blast loads. As an initial attempt to quantify this behavior, the responses of a ten story steel building, designed for the 1994 building code, with lateral resistance provided by perimeter moment frames, is considered. An analytical model of the building is developed and the magnitude and distribution of blast loads on the structure are estimated using available computer software that is based on empirical methods. To obtain the relationship between pressure, time duration, and standoff distance, these programs are used to obtain an accurate model of the air blast loading. A hemispherical surface burst for various explosive weights and standoff distances is considered for generating the air blast loading and determining the structural response. Linear and nonlinear analyses are conducted for these loadings. Air blast demands on the structure are compared to current seismic guidelines. These studies present the displacement responses, story drifts, demand/capacity ratio and inelastic demands for this structure.

**Keywords :** air blast, demand/capacity ratio, space frame, dynamic response, story drifts, earthquake

### 1. Introduction and Building Descriptions

The use of vehicle bombs by terroists to attack building structures has become of increasing concern to structural engineers since the bombing of the marine barracks in Beriut(1982), the initial attack on the World Trade Center(1993), and the attack on the Murrah building(1995) among others. Due to the

increasing threat of vehicle bomb attack, structural engineers have developed methods of design and analysis to protect against blast loads. However, the behavior of structures under blast loads is difficult to understand. Current design for air blast loads generally uses simplified analysis procedures(i.e. Single Degree of Freedom) that were developed in the late 1950's. More recent modeling and computational capabilities

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• 이 논문에 대한 토론을 2012년 2월 29일까지 본 학회에 보내주시면 2012년 4월호에 그 결과를 게재하겠습니다.

can readily be used to provide a more exact estimate of the structural behavior under these extreme loads.

The responses of a low rise steel building were investigated by Y.S. Hwang and J.C. Anderson (2008). This study assumes the wall perpendicular to the blast remains intact. This places the maximum horizontal force on the structural frame. However, the demand forces on frames would be overestimated due to assumption even though this study provide simplified method with engineers. The analytical method of low rise building has been expanded to multi degree of freedom system. In addition, it provides that engineers are able to find critical members and develop a method of progressive collapse analysis under blast loads. It has higher frequency compared with high rise building. In this study, air blast loads are applied to a ten story building with welded steel moment frames on the perimeter. It was designed in 1995 for the lateral force requirement of the 94' UBC. In addition, this study considers the size of the blast crater along with the different structural responses that include story displacements, demand/capacity ratio and plastic rotation demands. These parameters are then compared with limit values suggested by seismic guidelines.

The ten story building with a typical floor to floor height of 13'-0"(4m) with exception of 12'(3.66m) for the 1<sup>st</sup> floor and 18'(5.5m) for the 2<sup>nd</sup> floor. The plane is square in shape with the main roof and the same dimension of typical roof. The floor consists of a concrete over metal deck diaphragm that is assumed to be rigid in its plane. A full three dimensional analytical model of the building is developed for the SAP2000 computer code as shown in Fig. 1.

The lateral force resisting system in each direction consist of a 5 bay, welded steel moment frames on each side of the building perimeter. The remainder of the steel framing is provided for gravity loads shown in Fig. 2. The base of the column is assumed to be fixed. All beams and columns conform to ASTM A572 Gr.50., as specified. Simple shear connections for gravity framing are represented by a pinned connection and included in the computer model.

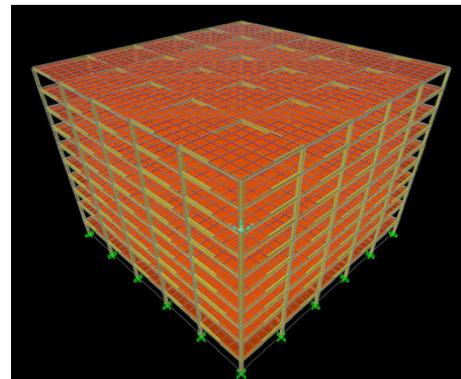
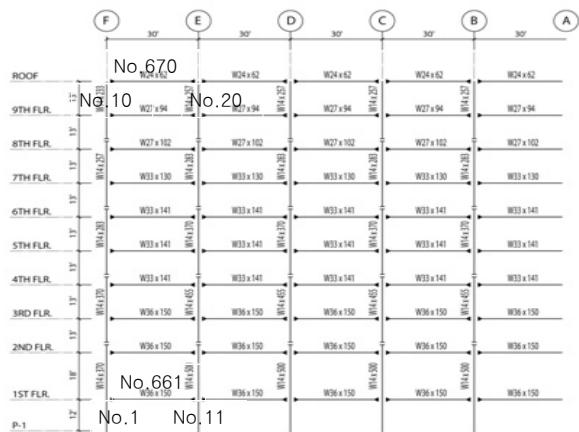
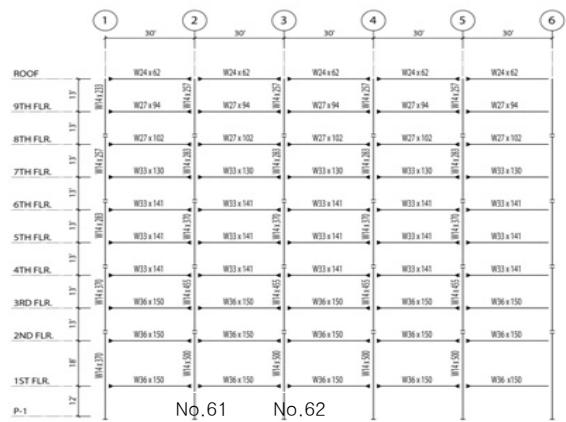


Fig. 1 Description of 10 Stroy Building Model Using SAP2000



(a) Welded Steel Moment Frame(WSMF)- Longitudinal Direction



(b) Welded Steel Moment Frame(WSMF) - Transverse Direction

Fig. 2 Welded Steel Moment Frame

## 2. Air Blast Loads

To investigate the size and extend of the blast crater, TM 5-855-1 is used considering explosive weights

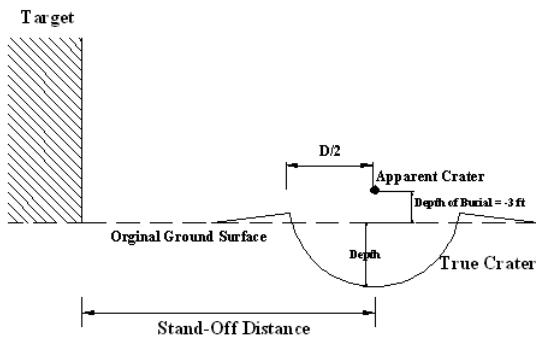


Fig. 3 Shape and Diameter of Blast Crater

Table 1 The Parameters of Due to Different TNT Weight

Charge weight lb(kg)	Depth of Burial ft(m)	Depth ft(m)	Radius ft(m)	Breakage range ft(m)
500(227)	-3(-0.91)	3.7(1.1)	7.16(2.18)	968(295)
1,000(454)	-3(-0.91)	5.19(1.58)	9.5(2.90)	1220(372)
2,000(907)	-3(-0.91)	7.12(2.17)	12.5(3.81)	1537(468)

of 500lb(227kg), 1,000lb(454kg), and 2,000lb(907 kg). The shape of the variable blast crater is shown in Fig. 3. The radii of the crater due to 500lb (227kg), 1,000lb(454kg) and 2,000lb(907kg) of TNT are summarized in Table 1. It can be seen that all three craters are less than the 20 feet(6m) standoff distance used in this study.

To define the blast loads, the CONWEP program is used to provide peak reflected pressure( $P_r$ ) and time duration( $t_d$ ) as a function of the weight of explosive and the standoff distance. Prior to determining peak pressure and time duration, the weight of explosive, direction to target and standoff distance are selected. For example, vehicle bombs of 2,000lb(907kg) TNT at 20ft(6m) and of 4,000lb(1814kg) TNT at 20ft(6m) standoff distance are treated as a hemispherical surface burst having peak reflected pressure and time duration as shown in Fig. 4 and Fig. 5 respectively.

Numbers shown are coded values of the pressures and time duration. Second case considers of 4,000lb (1814kg) TNT at 20ft(6m) standoff distances as shown in Fig. 5. Accordingly the blast wave propagates by compressing the air with supersonic velocity, and it is reflected by the building, amplifying the overpressure. Using these results, time history functions are defined for each node on the side of the

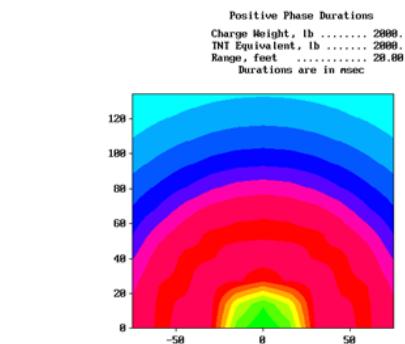
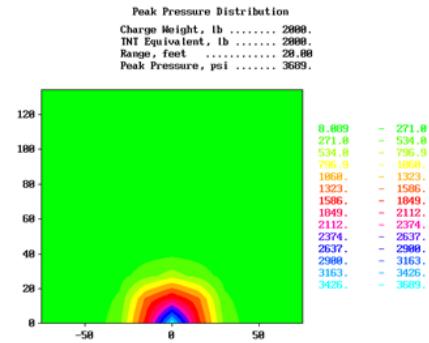


Fig. 4 Peak Reflected Pressure and Time Duration of 2,000lb(907kg) TNT weight at 20ft(6m) Range

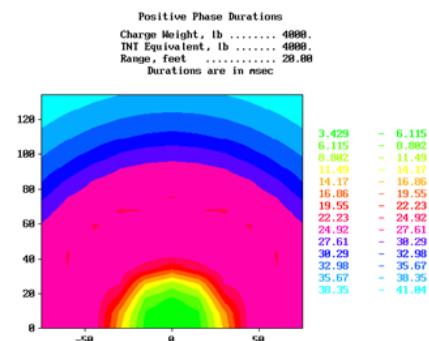
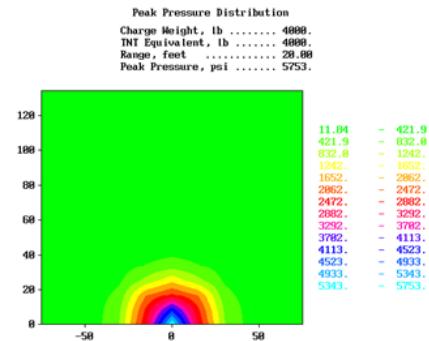


Fig. 5 Peak Reflected Pressure and Time Duration of 4,000lb(1814kg) TNT Weight at 20ft(6m) Range

building facing the blast and used as input data for the SAP2000 software.

### 3. Linear and Nonlinear Response Analysis

The floor diaphragms in the structure are often assumed to be rigid in their plane. However, they can also be represented by plate membrane elements resulting in the stress contours shown in Fig. 6 for a linear analysis.

These contours indicated how the blast loading that occurs on the face perpendicular to the blast is distributed to the moment frames on the sides parallel to the blast force.

Linear dynamic analyses can also be used to calculate the demand/capacity(D/C) ratios for the structural members. This is the ratio of the maximum bending moment to the plastic moment capacity of the section. This ratio can be used to indicate locations of high demand with ratios greater than unity indicating possible inelastic behavior. Calculated demand/capacity ratio for the loading condition is shown in Fig. 7. After analysis, the largest demands occur in the perimeter moment frames as might be expected as shown in Fig. 6. However, there is also a significant demand in the columns of the transverse frames which are normal to the blast loading as shown in Fig. 7.

The time history of the floor displacements obtained from the linear analyses are shown in Fig. 8. These indicate the maximum displacements vary between 18 inches(46cm) for 2,000lb(907kg) at 20

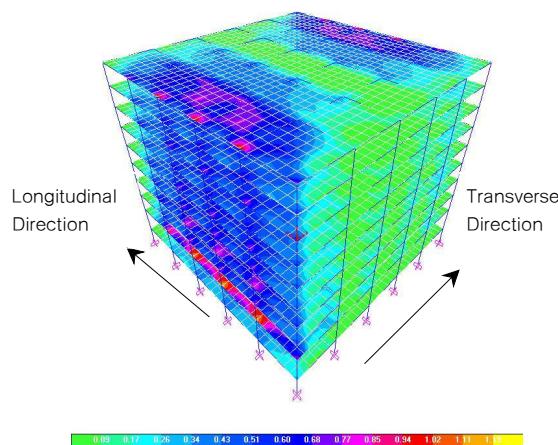


Fig. 6 Shear Force Distribution thru Floor Diaphragm (2,000lb(907kg)\_20ft(6m))

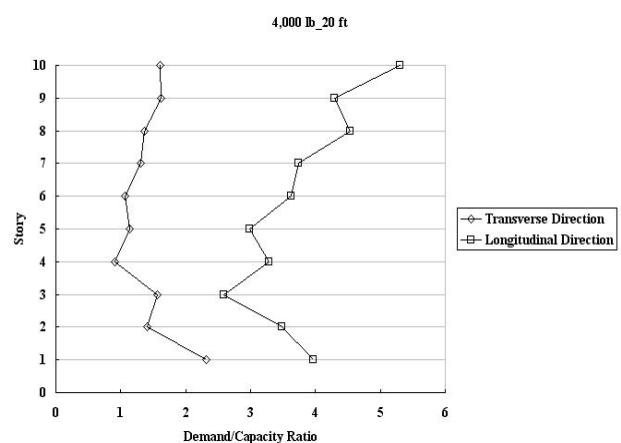
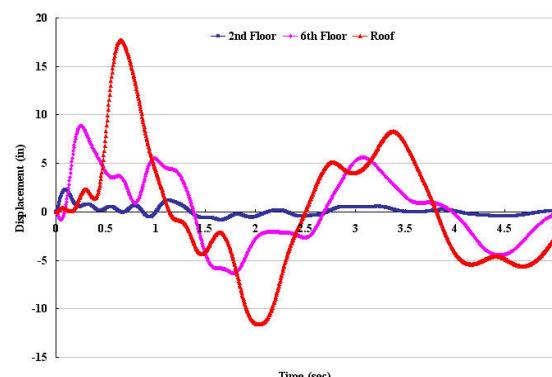
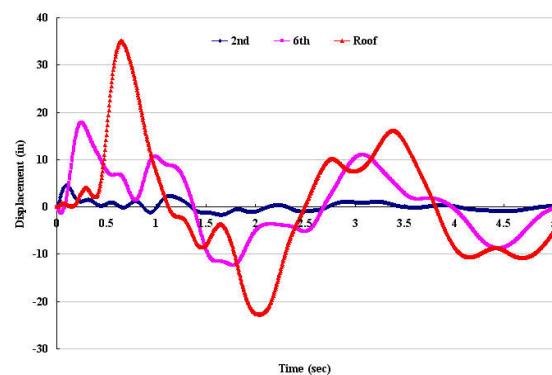


Fig. 7 Max. Demand/Capacity Ratio under Blast Loads (4,000lb(1814kg) TNT Weight at 20ft(6m))



(a) Case 1: 2,000lb(907kg) TNT weight at 20ft(6m)

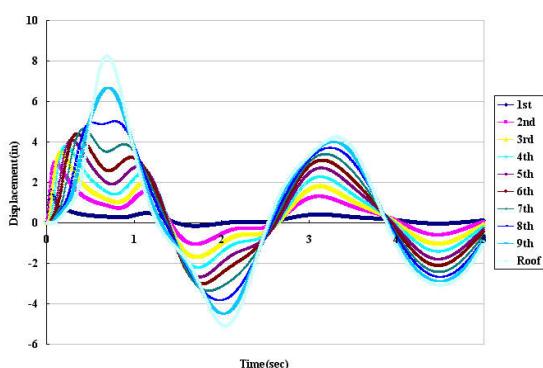


(b) Case 2: 4,000lb(1814kg) TNT Weight at 20ft(6m)

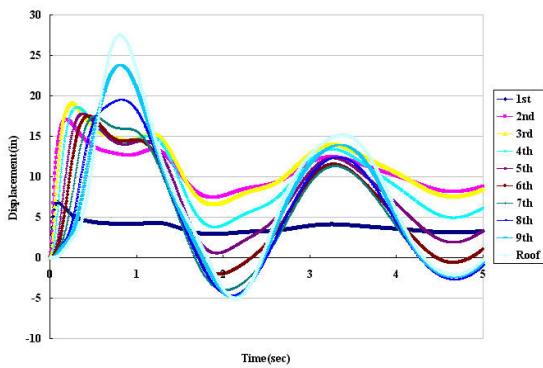
Fig. 8 Elastic Time History Curve

feet(6m) to 36 inches(91cm) for 4,000lb(907kg) at 20 feet(6m). Both curves also indicate that there is higher mode response during the first two cycles, but this is damped out during the third and succeeding cycles.

Similar displacement plots at each story obtained from the nonlinear analyses are shown in Fig. 9.



(a) Case 1: 2,000lb(907kg) TNT weight at 20ft(6m)



(b) Case 2: 4,000lb(1814kg) TNT weight at 20ft(6m)

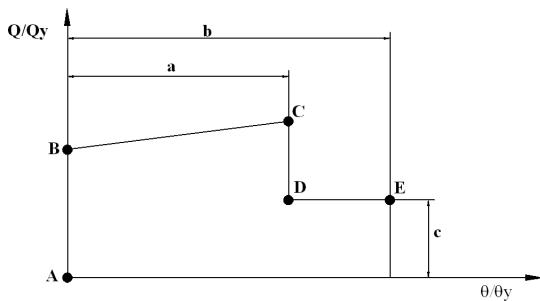
Fig. 9 Inelastic Time History Curve  
(4,000lb(1814kg)\_20ft(6m))

Fig. 10 Generalized Force-Deformation Relation for Steel Elements or Components (FEMA 356, Fig. 5-1)

Here it can be seen that displacement for the 2,000lb (907kg) @ 20 feet(6m) has actually been reduced from 6.8 inches(17cm).

It can also be noted that the response of the higher modes does not appear in the 1<sup>st</sup> floor. It has been damped out the inelastic deformations that have occurred throughout the frame.

The default plastic hinge properties in SAP2000 are used for the analyses. These properties are based on the recommendations made in FEMA356

Table 2 Modeling Parameter and Acceptance Criteria for Nonlinear Procedures

Component	Modeling Parameter		Acceptance Parameter			
	Plastic Rotation Angle (radians)		Plastic Rotation Angle (radians)	IO	LS	CP
Beams & Columns	$9\theta_y$	$11\theta_y$	$1\theta_y$	$6\theta_y$	$8\theta_y$	

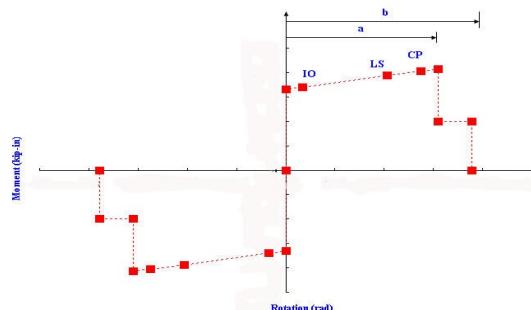


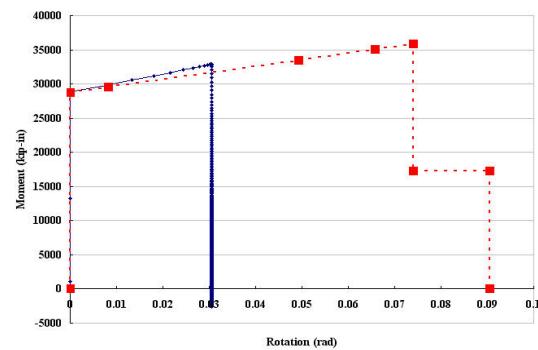
Fig. 11 Criteria of Plastic Hinge Behavior

for steel moment hinges.

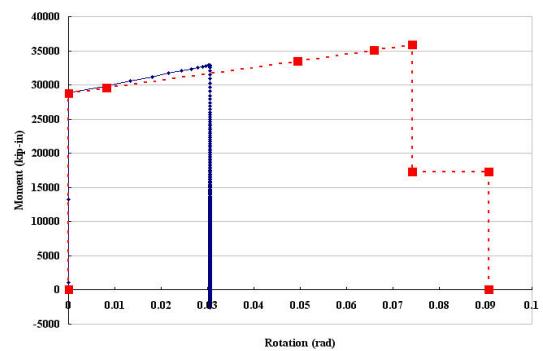
The moment-rotation following yield is shown in Fig. 10. It should be noted that point B represents yielding and this rotation is subtracted from the deformations at point C, D and E. Therefore, only the plastic deformation is indicated by the hinge. The hinge parameters are summarized in Table 2 along with the FEMA condition assessment.

To calculate the yield rotation,  $\theta_y$ , is used from FEMA 356 equation 5-1 and 5-2. The moment-rotation curve that gives the yield value and the plastic deformation following yield is shown in Fig. 11. It should be noted that point IO represents immediate occupancy, LS indicates life safety and CP menas collapse prevention.

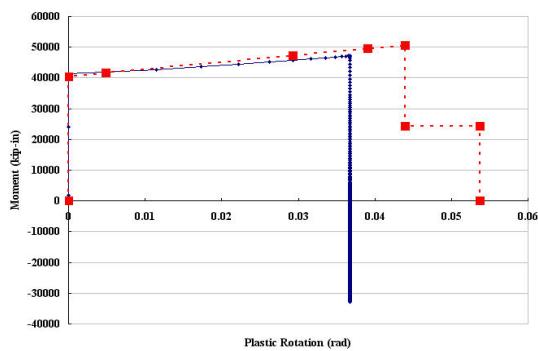
The maximum member demands for the condition of 4,000lb(1814kg) @ 20 feet(6m) are summarized in Fig. 12. Here it can be seen that there is yielding in the beams over the height of the frame with plastic rotation demands ranging from 0.031 radians at the first floor to 0.007 radians at the roof level. These high plastic rotation demands cause the structure to be critical for life safety(LS). Columns at the first floor level also have a high plastic rotation demand that makes them also critical for LS. The roof columns are only weakly nonlinear and are therefore



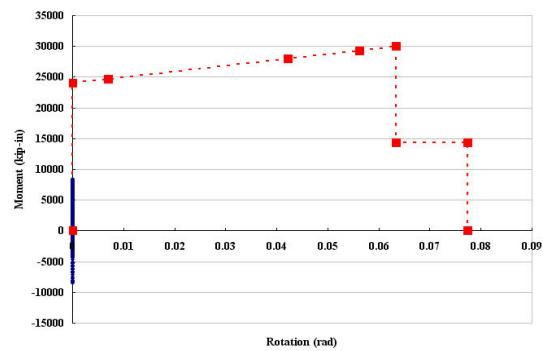
(a) Moment-Rotation(Col.61)



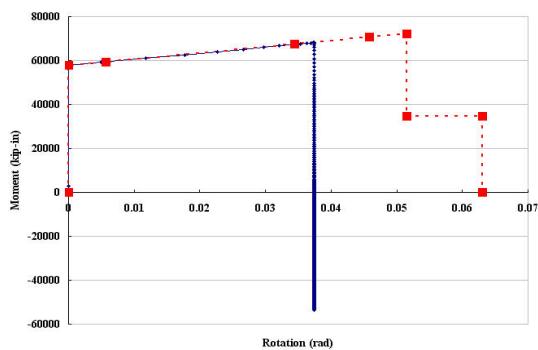
(b) Moment-Rotation(Col.121)



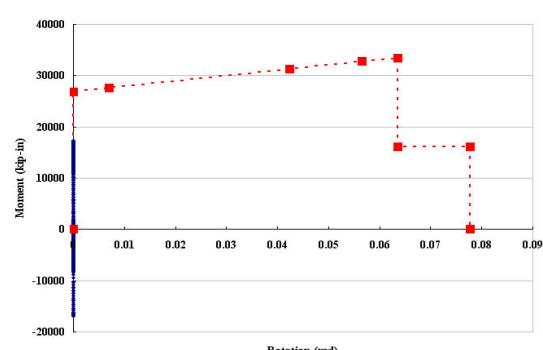
(c) Moment-Rotation(Col.1)



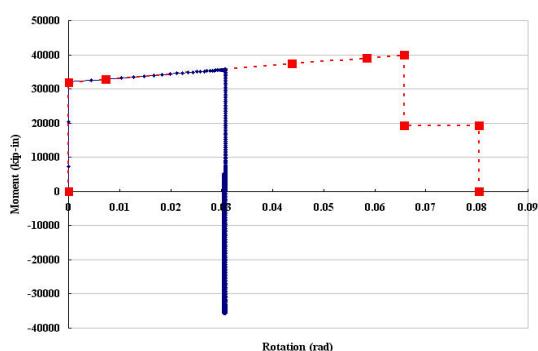
(d) Moment-Rotation(Col.10)



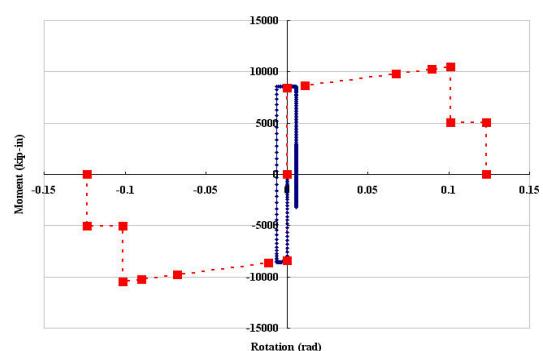
(e) Moment-Rotation(Col.11)



(f) Moment-Rotations(Col. 20)



(g) Moment-Rotation(Beam 661)



(h) Moment-Rotation(Beam 670)

Fig. 12 Moment-Rotation Relation in case of 4,000lb(1814kg) TNT Weight at 20ft(6m) Standoff Distance

Table 3 The Results of Nonlinear Analysis for 4,000lb (1814kg) TNT Weight at 20 feet(6m)

Case	Member	Section	$\theta_y$	$\theta_p$	Hinge State
4,000lb @ 20ft	Column (Longitudinal)	No.1 (1 <sup>st</sup> Floor)	0.006	0.037	LS-CP
		No.10 (Roof)	0.007	0	Linear
		No.11 (1 <sup>st</sup> Floor)	0.006	0.038	LS - CP
		No.20 (Roof)	0.007	0	Linear
	Column (Transverse)	No.61 (1 <sup>st</sup> Floor)	0.008	0.031	IO-LS
		No.121 (1 <sup>st</sup> Floor)	0.008	0.031	IO-LS
	Beam (Longitudinal)	No.661 (1 <sup>st</sup> Floor)	0.007	0.031	IO-LS
		No.670 (1 <sup>st</sup> Floor)	0.011	0.01	<IO

classified as IO. Using the FEMA criteria, the building would be critical under these blast loads.

#### 4. Interstory Drift(Nonlinear Analysis)

An important parameter in earthquake resistant design is the interstory drift index that is obtained by dividing the maximum relative story displacement by the story height. The UBC requires that for structures having a period greater than 0.7 seconds the interstory drift be limited to 0.02. The graph shown in Fig. 13 indicates that for the 1,000lb (454kg) TNT weight at 20 feet(6m) the drift is well below this limit, further indicating that the structure is suitable for immediate occupancy. However, for

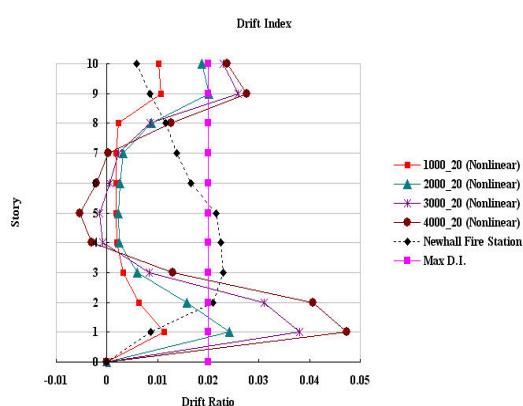


Fig. 13 Max. Drift Ratio Analyzed by Nonlinear Analysis

other cases, the interstory drift ratio is above the limiting value defined as UBC97.

#### 5. Conclusions

If the blast wave punctures the wall and enters the buildings, injury may occur to the occupants due to flying glass and nonstructural components but the force on the frame would be remained.

The results of this limited study indicate the following:

- ① For the blast loads used, the radii of the craters were all less than the standoff distances.
- ② 2,000lb(907kg) and 4,000lb(1814kg) TNT weight at 20 feet(6m) standoff distance, this structure is also needed adequate protection.
- ③ The demand/capacity ratios for blast based on an elastic analysis can be used to indicate areas of potential problems when nonlinear analyses are conducted.
- ④ The higher demand/capacity ratios and inelastic deformations occur in the longitudinal frames (strong axis column)
- ⑤ The drift ratio does not satisfies the UBC'97 code for the 2,000lb(907kg) TNT weight and 4,000lb(1814kg) TNT weight at 20 feet(6m).
- ⑥ The diaphragm is effective in distributing the blast loads from the front face to the other frames on the perimeter provided in remains intact.
- ⑦ The columns(10, 20) on roof level through longitudinal direction are still in linear ranges. According to the recommendations of FEMA356, this building under 4,000lb(1814kg) TNT weight at 20ft(6m) would be classified as suitable for collapse prevention(CP).

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- 논문접수일 2011년 10월 27일
- 논문심사일  
    1차 2011년 11월 8일  
    2차 2011년 11월 24일
- 게재확정일 2011년 12월 1일