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Influence of Concrete Jacketing on the Performance of Steel Columns under Blast induced Progressive Collapse

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Abstract- **This study investigates the influence of concrete jacketing on the performance of steel columns subjected to blast loading, which can lead to progressive collapse. Using Finite Element Method (FEM) simulations, the research evaluates concrete-encased steel columns with different concrete cover thicknesses to measure their resistance to lateral displacements induced by blasts. Findings reveal that increasing the thickness of the concrete cover markedly improves the column's ability to withstand such loads. The study also highlights that steel has better energy dissipation properties than concrete. It examines how the combined challenges of progressive collapse and blast loading influence the overall structural response of a building, identifying potential failure points in structural members. This research emphasizes the need to integrate considerations of both blast resistance and progressive collapse potential into structural design, aiming to enhance the performance and resilience of steel columns and contribute to the development of more robust structural systems capable of surviving extreme events and reducing the risk of catastrophic failure.**

1. INTRODUCTION

The phenomenon of progressive collapse occurs when the localized damage of a primary structural component results in the complete or partial failure of the structural system (Elsanadedy et al. 2014) .Concerns about progressive collapse phenomena were initially raised after the incident at Ronan Point, a 22-story building in London, UK, which caused the partial collapse of one corner of the building (Leyendecker and Ellingwood n.d.). The level of concern in this area of research increased following the subsequent collapse of Skyline Plaza in Virginia, US, where one apartment in the building and its adjoining parking garage collapsed (Carino, Leyendecker, and Fattal 1983).

To evaluate and mitigate the risk of progressive collapse effectively, the progressive collapse analysis may be performed through three key methods in CSI SAP2000: Linear Static (LS), Nonlinear Static (NLS), and Nonlinear Dynamic (NLD). Linear Static analysis is limited to structures with a maximum of 10 stories that meet specific irregularity criteria mentioned in *(General Services Administration 2016)*, and all Demand Capacity Ratios (DCRs) are ≤ 2.0 as used in (Bhavana and Anand Baldota 2018),(JalaliLarijani et al. 2013). Nonlinear static analysis accounts for both material and geometric nonlinearities, making it suitable for structures with irregularities or when linear static analysis is inadequate due to high Demand-Capacity Ratios (DCRs), as noted in (Mahmoud et al. 2018). The Nonlinear Dynamic method, simulates the dynamic response to sudden loading events, offering comprehensive insights into structural behavior under extreme conditions *(General Services Administration 2016)*.

Numerous studies have explored the factors affecting progressive collapse, one such study examining the resilience of steel frames after sudden column loss. This research by Li et al. emphasized that various properties, such as damping and material properties, play crucial roles in enhancing collapse resistance (Li et al. 2018). The study highlighted the necessity of accurately representing these properties during modeling to understand their influence on the robustness of steel frames effectively. These insights are foundational for the current research as concrete jacketing was examined to enhance the robustness of steel columns under similar conditions.

Additionally, a study by Dadkhah and Mohebbi investigated the effect of the distance between a bomb and the building (stand-off distance) on its vulnerability (Dadkhah and Mohebbi 2023). This study found that increased stand-off distance with perimeter protection can improve blast resistance for low-rise buildings but may worsen the situation for high-rise structures. The findings emphasize the importance of considering both building characteristics and specific blast threats when implementing blast protection measures.

To numerically simulate the blast load, its parameters can be determined using different methods including TM5-1300 graphs and empirical equations to obtain reflected pressure on the structure as detailed by Zhou and Hao (Zhou and Hao 2008). While these methods provide valuable data for individual scenarios, they do not address the combined impact of progressive collapse and blast loads.

To improve the performance of steel columns, other researchers investigated the concrete jacketing of steel sections. For instance, Zerfu and Yadeta found that the thickness of the concrete cover surrounding the steel section improves the column's performance(Zerfu and Yadeta 2023). A thicker concrete cover results in less damage to the steel core and a greater

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ability to withstand loads. This study discovered that increasing the concrete cover thickness leads to a noticeable improvement in load capacity. The finite element analysis (FEA) used in their research closely matched real-world experimental results, validating the accuracy of the FEA method.

Elsanadedy et al. investigated the progressive collapse resulting from blast loading on a building (Elsanadedy et al. 2014). The primary findings indicated that progressive collapse can occur even with a relatively small charge weight, such as 500 kg, which can be easily transported in a vehicle. To mitigate this potential, the study recommends increasing the stand-off distance or strengthening ground story columns with concrete encasement or steel plates.

Kiakojouri et al. critiqued the limitations of the traditional code-based Alternate Load Path (ALP) method in evaluating the blast-induced progressive collapse of steel frames (Kiakojouri et al. 2021). They suggested a refined version of the ALP method that integrated threat-specific parameters for enhanced collapse predictions. Although their proposed method offered the potential for broader application, including other triggering events and more detailed modeling, further research is necessary to explore how increasing concrete cover thickness can improve resistance to progressive collapse under blast conditions.

Previous research has explored the effects of progressive collapse and blast loads individually, in addition to their combined impact, often recommending the reinforcement of steel columns through concrete encasement. However, the specific interaction between different concrete cover thicknesses and their effect on resistance to combined blast-induced progressive collapse has not been comprehensively studied. This research aims to fill this gap by providing a comprehensive analysis of how different thicknesses of concrete cover affect the structural resilience of steel columns under both blast and progressive collapse scenarios.

2. VERIFICATION OF FEM WITH GSA2003 ANALYTICAL EXAMPLE

2.1 DIMENSIONS AND CHARACTERISTICS

According to the GSA 2003 guidelines *(General Services Administration 2003)* and as referenced in (Bhavana and Anand Baldota 2018) and (JalaliLarijani et al. 2013), linear static analysis (LSA) is applicable for buildings that are 10 stories or less, provided they do not exhibit significant irregularities as defined in Paragraph 3.2.11.1.1 and as used in (Bhavana and Anand Baldota 2018), (JalaliLarijani et al. 2013). The studied structure meets these criteria, as it does not present significant discontinuities in the gravity-load-carrying or lateral-force-resisting systems. Additionally, the stiffness and/or strength ratios between adjacent sides of columns, beams, and intersecting walls exceed 50%, ensuring regularity. Therefore, the linear static approach is appropriate and compliant with the GSA 2003 standards.

The analysis was conducted using frame elements and a linear static analysis method, modelling was performed using CSI ETABS software. The building is located in Atlanta, GA, on Site Class D, leading to its classification as Seismic Design Category (SDC) C. Steel Intermediate Moment Frames (IMFs), as detailed in *(American Institute of Steel Construction 2002)*, serves as the lateral-force-resisting system for this structure. The building comprises a 10-story office building with a footprint of 150 feet by 100 feet as shown in Figure 1. Its lateral-force-resisting system consists of 3 bays of steel moment-resisting frames, while the remaining framing is interconnected with simple shear connections. The building plan layout is organized into 5 × 5 bays as depicted in Figure 1.

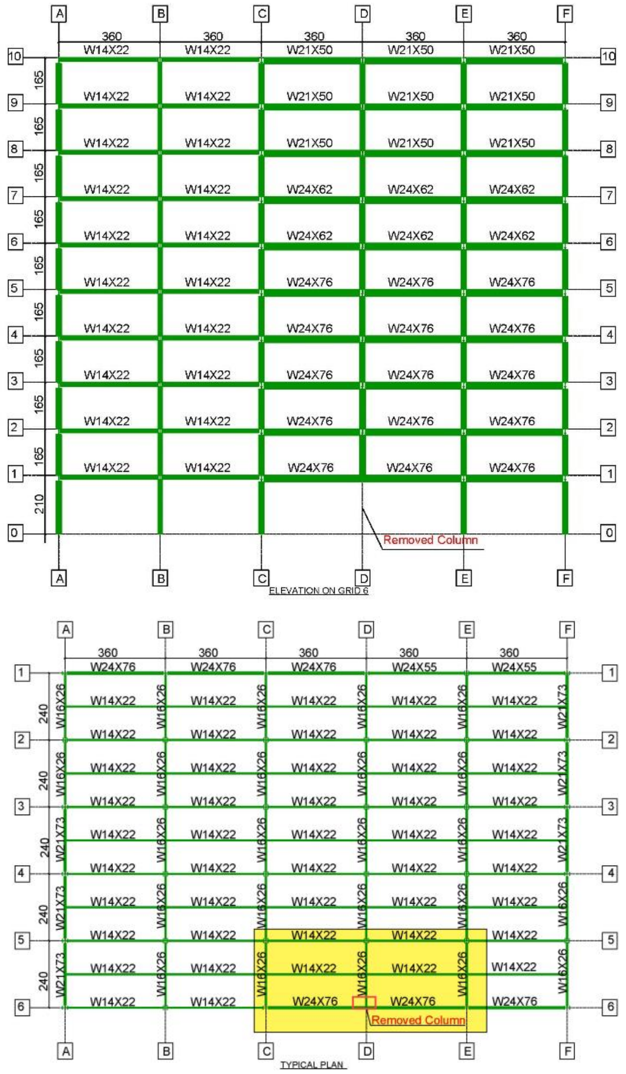

FIGURE 1. Building Elevation and Floor Plan (Dimensions in inches) (Dimensions in inches)(Ghosh, 2006)

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Structural steel members, including beams and columns, are primarily made from ASTM A992 steel with a yield strength of F_y = 50 ksi. The stress-strain curve for ASTM A992 steel, illustrated in Figure 2, demonstrates its strain hardening and indicates the yielding criteria used. The floor system comprises steel beams supporting a 3-in. metal deck topped with 3 ¼ in. lightweight (110 pcf) concrete and headed shear studs to facilitate composite action. Steel deck attachment to supporting beams is achieved through puddle welds, while shear studs are welded through the deck to the beam flange below *(American Institute of Steel Construction 2002)*. Members are represented by centerline elements, with zero ends offset to accommodate joint flexibility. All moment connections, both in the East-West and North-South directions, are welded unreinforced flange (WUF) connections, encompassing the design of double plates, shear tabs, and bolts. Gravity framing connections are assumed to be pinned, except for secondary member checks where they are considered partially restrained (PR) moment connections *(American Institute of Steel Construction 2002)*. Column-to-foundation connections are treated as pinned, and each floor is assumed to act as a rigid diaphragm *(American Institute of Steel Construction 2002), (General Services Administration 2003)*. Nonstructural components such as exterior cladding and interior walls are not factored into the designs, as their impact on structural behavior and performance is deemed negligible *(American Institute of Steel Construction 2002), (General Services Administration 2003)*. Only the loads resulting from these components are considered. The investigation will involve the removal of one column at axes D6, F6, and F3 on the first floor, respectively.

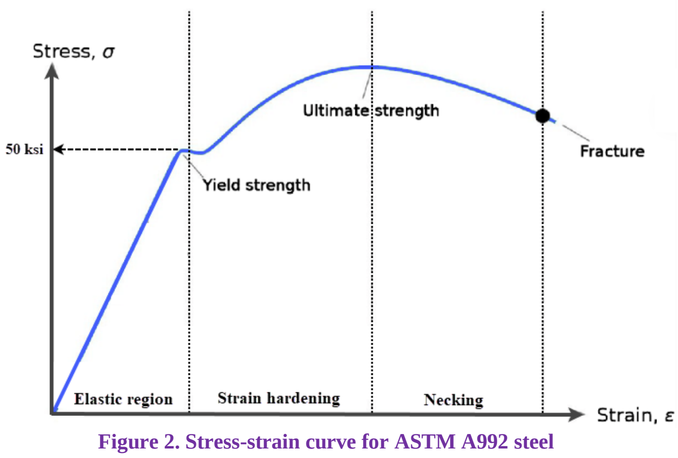

Figure 2. Stress-strain curve for ASTM A992 steel

2.2 LOADING

A standardized set of loads was applied to the prototype structures. This included a total dead load (DL) of 46 psf for typical floors, comprising a 3-inch metal deck and a 3 1/4-inch lightweight composite slab, with an additional allowance for the deck. The roof floor had the same composite slab weight, with a superimposed load (SDL) of 30 psf for ceilings, mechanical loads on typical floors, and 10 psf for the roof. The self-weight of steel framing elements was also included in the dead loads. Since all structural elements except the slab were explicitly modeled, the slab and deck weight were distributed as loads on the beams. Consequently, the final superimposed dead load was 46+30 psf for occupied floors and 46+10 psf for the roof. For partition loading, a live load (LL) of 100 psf was assumed for typical floors, while a Live Roof (Lr) load of 20 psf was considered for the roof.

2.3 ANALYSIS AND RESULTS

The column removed from the structure is situated along the longitudinal side of the building at axis D6, as shown in Figure 1. To simulate progressive collapse, an increased load is applied directly above the collapsed column using the formula: Increased Load = 2(DL + 0.25LL) (Carino, Leyendecker, and Fattal 1983). This method is employed to validate the findings reported in (Ghosh, 2006).

Table 1 presents the bending and shear stresses for D/6-E/6 beams resulting from the removal of Column D6. These data are used to validate the analysis results reported by (Ghosh, 2006). Additionally, the state of D/6-E/6 beams is evaluated through the calculation of the flexure Demand-Capacity Ratio (DCR), calculated as the ratio of the applied moment (M_u) to the resistance moment capacity (ΦM_n) which is calculated according to provisions of chapter F in ANSI/AISC 360-22 *(American Institute of Steel Construction 2022)*.

The results of this analysis demonstrate that the FEM outcomes closely match (Ghosh, 2006) results, with a standard deviation of 1.581% and 1.6% for bending and shear stresses in the beams, respectively. DCR values are less than the limiting value of 2.0 for all of the D/6-E/6 beams along the height of the building. This indicates that the building is considered to have a low potential for progressive collapse, as outlined in *(General Services Administration 2003)*. Additionally, following the removal of the column, the beam directly above the removed column exhibited greater deformation compared to the adjacent beams, as illustrated in Figure 3.

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Table 1. Bending and Shear Stress on Beams Due to the Removal of Column.

Story	Beam D/6-E/6	Xuemei		FEM Model		Div. M	Div. V	DCR M	DCR V
		M_u (ft-kips)	V_u (Kips)	M_u (ft-kips)	V_u (Kips)				
Roof	W21X50	259	26	245	24	5.4%	7.7%	0.51	0.10
9	W21X50	330	33	307	30	7.0%	9.1%	0.64	0.12
8	W21X50	326	33	305	30	6.4%	9.1%	0.63	0.12
7	W24X62	474	44	443	41	6.5%	6.8%	0.66	0.13
6	W24X62	506	46	474	42	6.3%	8.7%	0.71	0.13
5	W24X76	626	56	599	52	4.3%	7.1%	0.68	0.16
4	W24X76	645	58	623	54	3.4%	6.9%	0.71	0.16
3	W24X76	685	61	656	56	4.2%	8.2%	0.75	0.17
2	W24X76	729	64	697	59	4.4%	7.8%	0.80	0.18
1	W24X76	700	61	688	59	1.7%	3.3%	0.79	0.18
Mean						4.960%	7.470%		
Standard Deviation						1.581%	1.607%		

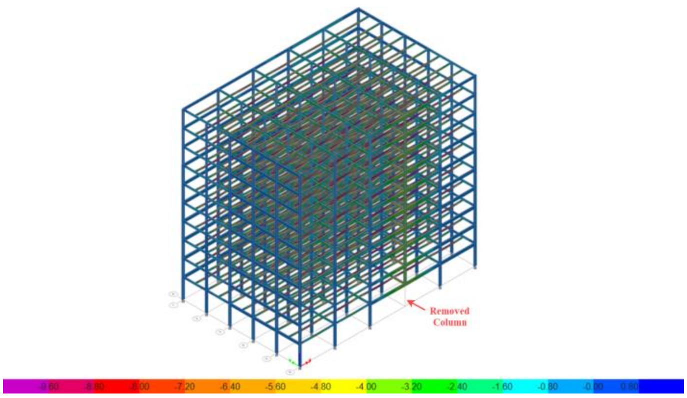

Figure 3. Deformed shape: After column removal (inches)

3. THE AISC FULL-SCALE COLUMN BLAST LOAD VERIFICATION

After verifying the results of the PC, it is necessary to ensure the blast load on the building is accurately represented through the time history function. This involves verifying the results of the AISC full-scale test with the numerical model results.

An experimental test was performed for a column in a building subject to blast load at ground level. The analysis was conducted using the linear static analysis method, with CSI SAP2000 software used for modeling. The column is modeled as the actual case with a shell element, fixed-hinged end supports, and blast load as time history load. The verified data is the deflection of the column at points of east flange, web, and west flange along the height of the column.

3.1 DIMENSIONS AND PROPERTIES:

The columns were modelled using shell elements, the flange plate was shell layered element with dimension 0.40*0.043 m (15.865*1.688 inch), web plate was shell layered element with dimension 0.41*.027 m (16.000*1.045 inch),
and column length is 5.70 m (18.75 ft).
Column end supports are simulated through fixed-hinged end conditions to present FEM results close to the experimental results. FEM is a column with hinged-end supports at top and fixed-end supports at the bottom.

The material used for the steel column is ASTM A992. The stress-strain curve for ASTM A992 steel, illustrated in Figure 2, demonstrates its strain hardening and indicates the yielding criteria used. The yield strength is 50 ksi, the ultimate strength is 65 ksi, the damping ratio was 5%, and the strain hardening increased yield strength by 24%.

3.2 LOADING:

Blast load was modelled as a time history function. The charge weight is 4000 lbs; the effective standoff distance is 188 inches. The charge wave parameters can be determined from the CONWEP Program as shown in Figure 4, the reflected pressure is 9500 psi and arrival time and blast load duration are 1.08 msec and 2.82 msec, respectively. The charge was applied at an angle of 27°, resulting in the force being analysed in two directions. The flange force corresponds to the cosine component, while the web force corresponds to the sine component. These forces vary along the height of the column, with the maximum reflected pressure occurring at the bottom of the column. This pressure decreases with height until it reaches zero at a height of 5.72 meters. This load distribution is illustrated in Figure 5.

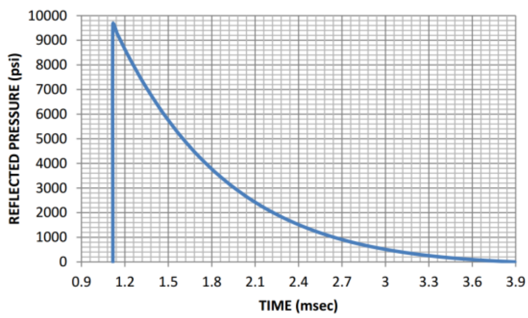

Figure 4. Reflected Pressure on the structure Vs. Time

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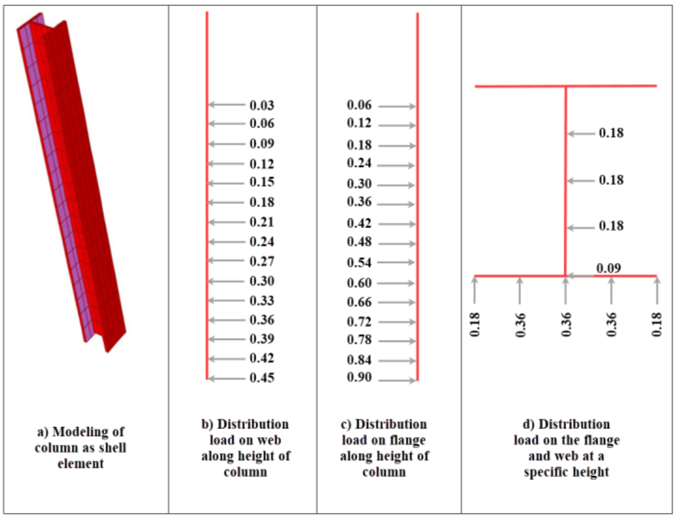

Figure 5. Distribution of blast Load on the Examined Column

3.3 ANALYSIS AND RESULTS:

The numerical results are generally higher than the measured deflection data as shown in Figure 6. The average ratio between the predicted deflection and the measured one is 1.25. The following reasons are given for such difference in results (Magallanes, Martinez, and Koenig 2006):

1. The analysis assumed conservative loads with a fully reflected surface, whereas the actual cladding had a small finite surface, reducing the total impulse.
2. The explosive was idealized as a hemispherical-shaped charge on the ground, increasing the blast pressure and impulse.
3. The debris was assumed to be uniform and not connected before shockwave impingement, not accounting for the energy absorbed during cladding fracture.

Similar findings on the impact of perimeter protection on blast performance are discussed in studies where increasing the standoff distance significantly influences the structural response under blast loads. For instance, the study on the effect of perimeter protection on steel moment-resisting buildings highlighted that increased standoff distance reduces peak story drift, showing the necessity of accurate blast load assumptions (Dadkhah and Mohebbi 2023).

After validating the results for both the progressive collapse (PC) and blast load scenarios, the next step will be to examine the impact of concrete jacketing on momentum-resistant frames (MRF) when subjected to the combined effects of blast loading and progressive collapse.

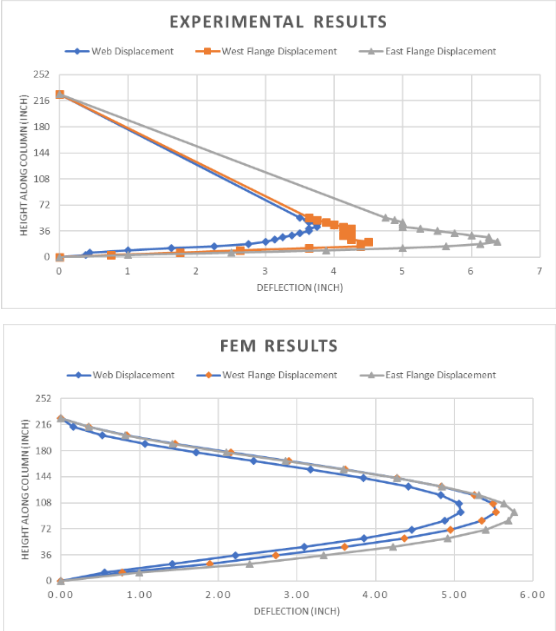

Figure 6. Deflection of the column as a function of height along the column for FEM Vs. experimental results

4. PARAMETRIC STUDY

In this study, the linear static analysis (LSA) method and CSI SAP2000 software were used to evaluate the structural response of a building subjected to blast loads and to assess potential retrofitting strategies for enhancing its capacity to resist lateral displacement and absorb energy from such loads. According to the GSA 2003 guidelines, the LSA method applies to this building according to Paragraph 3.2.11.1.1 in GSA 2003 *(General Services Administration 2003)*.

A blast equivalent to 5 tons of TNT was initially simulated at a standoff distance of 40.4 feet to evaluate its impact on the building's height. Following this, the formation of plastic hinges in various structural elements will guide the decision on which vertical load-bearing element should be removed. The building's structural integrity will then be assessed based on the GSA acceptance criteria, focusing on determining the Demand-to-Capacity Ratios (DCRs) and evaluating the overall structure condition.

Furthermore, the study investigates how to enhance the building capacity through blast energy absorption and distinct lateral displacement. Various scenarios were explored to understand the effectiveness of concrete jacketing on steel columns, with 4000 psi concrete covers ranging from B/6, B/5, to B/4, where B represents the column dimension. This investigation aimed to identify the optimal concrete cover thickness necessary, thereby contributing to the development of efficient retrofitting strategies for blast-resistant structures.

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4.1 DIMENSIONS AND PROPERTIES

The building, consisting of a three-by-two bay layout with each bay measuring 20.0 feet in length, features a three-story steel frame structure with a total height of 40.4 feet. It employs a moment-resisting frame as its lateral force-resisting system (LFRS), designed primarily to handle gravity and wind loads, but not explicitly for explosion effects. The beam-to-column connections are rigid to improve the columns' resistance in both directions, while the girder-to-beam connections are pinned.
The structure was modeled using frame elements to precisely represent the columns, beams, and bracing system.

This approach allowed for precise simulation of the building's behavior under various loads. Gravity loads were applied to the structural shells, effectively transferring these loads to the frame.

The material used is ASTM A992 steel, characterized by a yield strength (Fy) of 50 ksi and an ultimate tensile strength (Fu) of 65 ksi. The steel's strength is further enhanced by considering strain hardening, as illustrated in Figure 2, and by applying a Dynamic Impact Factor (DIF) of 4%, as specified in the guidelines of *(TM5 1300 1990)*.

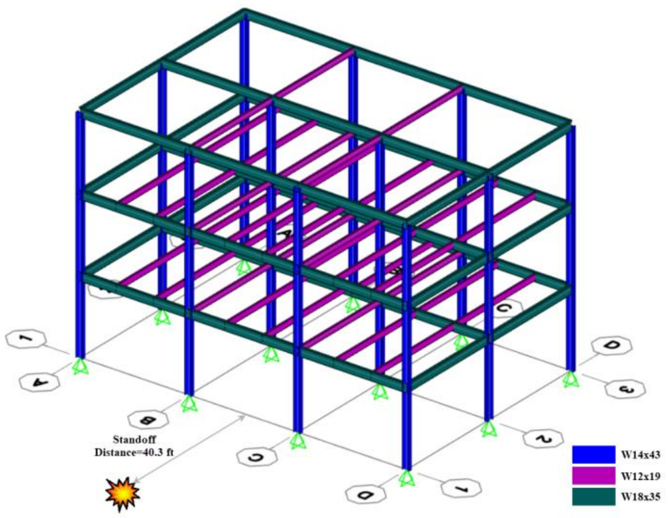

FIGURE 7. MRF Model Geometry and Cross Sections

4.2 LOADING

The gravity loads acting upon the building are meticulously considered and quantified to ensure structural integrity. The floor systems are designed to withstand a dead load of 48 psf and superimposed dead loading of 10 psf, accounting for the weight of permanent fixtures and materials. The design floor live load was 100 psf. Moreover, the roof was designed for a live load of 21 psf. In terms of wind loads, the building is designed to resist typical gravity and wind loads with a basic wind speed of 167 feet/sec (Hadjioannou, McKay, and Benshoof 2021).

The blast load was modeled as time history function and the selected explosive material is solid explosive material 11023 lbs of TNT, with a standoff distance of 40.4 feet, to affect the height of the building. The charge wave parameters can be determined from the CONWEP Program as shown in Figure 8, the reflected pressure is 3000 psi and the arrival time and blast load duration are 4.005 msec and 7.789 msec, respectively.

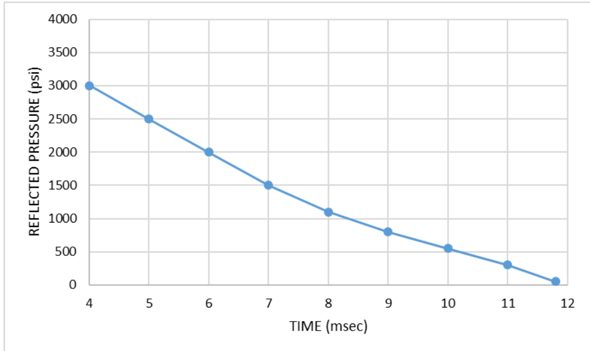

Figure 8. Reflected Pressure on the structure Vs. Time

4.3 PLASTIC HINGE DEFINITION

For MRF beams, Figure 9 shows the generalized force versus deformation curves used throughout this study to specify component modeling and acceptance criteria for deformation-controlled actions. Component modeling involves acceptance criteria for deformation-controlled actions across different material types.

The relationship between unloaded and yield points is linear (A to B), with point C indicates component strength and point D marks significant strength degradation (line CD). Beyond point E, strength is substantially reduced, and is nearly zero beyond E. Acceptance criteria can be expressed in translational deformation Δ and rotational deformation θ or deformation ratios. The parameters A, B, C, D, and E, were obtained from *(ASCE 2000)*, providing force versus deformation curves for component modeling and acceptance criteria as shown in Table 2.

Table 2. Moment (M) VS. Rotation (θ)

Rotation (θ)	Moment (M)
-6.16	-0.20
-5.03	-0.20
-5.03	-1
0	-1
0	0
0	1
5.03	1
5.03	0.20

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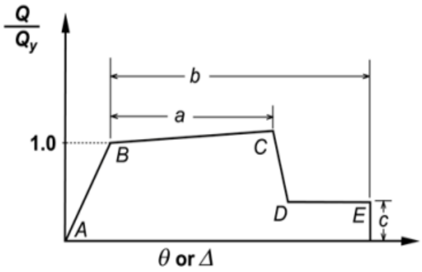
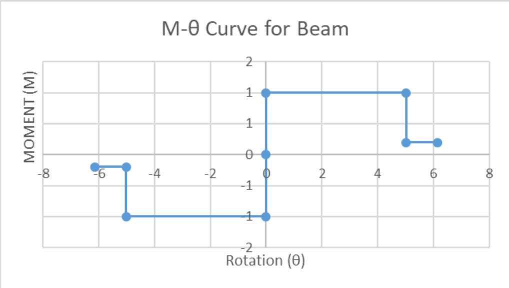
6.16	0.20



(1)



(2)

FIGURE 9. Generalized Component Force-Deformation and Acceptance Criteria (1) (ASCE 2000) and Moment–Rotation Curve for Beam (2).

From the analysis results of the column cross-section and according to FEMA 356 guidelines *(ASCE 2000)* the ratio of the axial force in the member (P) to the lower-bound compression strength of the column (PCL) has been calculated and found to be greater than 0.5 as shown in Table 3. Based on this finding, the formation of plastic hinges in the column was considered force-controlled, indicating a brittle behavior.

Table 3. Axial force-lower bound compression strength ratio for columns

P (kips)	P_{CL} (kips)	P/P_{cl}
295	551.67	0.53

4.4 CONCRETE JACKETING

The analysis was conducted using frame elements and a linear static analysis method, with CSI SAP2000 software used for modeling.

The concrete encasement examined in this study adhered to the specifications outlined in ACI318 following the stress-strain curve shown in Figure 10, which states a minimum concrete cover of 1-1/2 inches according to table 20.5.1.3.1 *(American Concrete Institute 2019)*. Therefore, column encasement thicknesses were determined as follows: B/6 resulted in a thickness of approximately 2.27 inches, B/5 equated to around 2.73 inches, and B/4 necessitated approximately 3.42 inches of concrete cover.

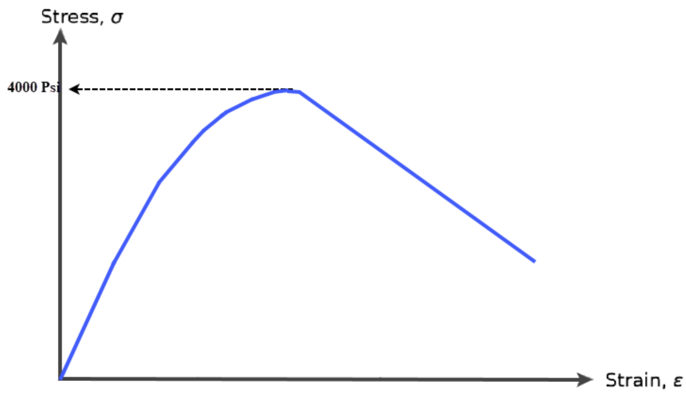


Figure 10. Stress-strain curve for concrete

The concrete jacketing involved adding 6-3-#5 reinforced steel bars and #8@8 stirrups, as illustrated in Figure 11. The dimensions, area, reinforcement (RFT) ratio and steel core ratio for columns were specified in the analysis, as shown in Table 4. The reinforcement ratio and steel core ratio, as detailed in Table 4, adhere to the minimum and maximum ratios specified in Chapter I of AISC 360-22, which range from 1% to 8%. This ensures compliance with the relevant standards *(American Institute of Steel Construction 2022)*. Similarly, the steel core ratio meets the minimum requirement of 1% *(American Institute of Steel Construction 2022)*, demonstrating adequate steel core presence in the composite column design.

Connections between beams and jacketed columns are maintained as rigid to enhance the column's resistance in both directions, whereas connections between girders and beams are designed as pinned.

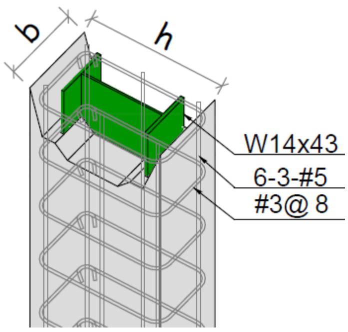


Figure 11. Column Concrete Jacketing RFT

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Table 4. Column Jacketing Specifications and Ratios

Column Jacket-ing	h (inch)	b (inch)	Area (inch^2)	RFT Ratio	Steel Core Ratio
(B/6)	18.21	10.66	194.16	7.45%	6.49%
(B/5)	19.12	11.19	214.06	6.76%	5.89%
(B/4)	20.49	11.99	245.73	5.89%	5.13%

4.5 ANALYSIS AND RESULTS

During the study, the blast load was applied to the building façade as joint forces and loads directly transferred to the structure as shown in Figure 12. Plastic hinges formed in the columns aided in determining which column would eventually fail under the effect of blast load. Once the collapsed column, was identified, it was removed from the model, as illustrated in Figure 13, Figure 14 and Figure 15. After the column's removal, the structural changes are checked. Subsequently, column (D-1) Demand-Capacity Ratios (DCR) values have been computed.

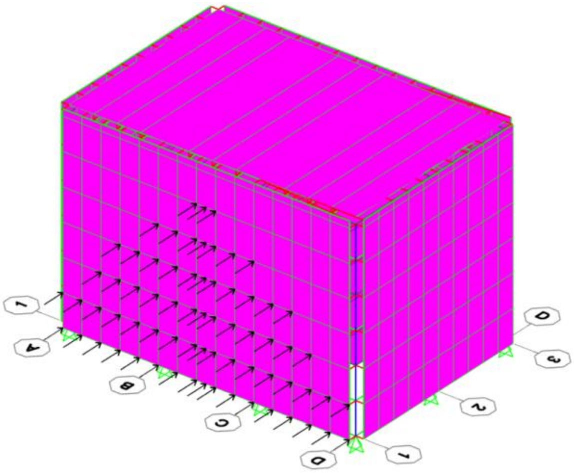

Figure 12. Blast Loading Scenario

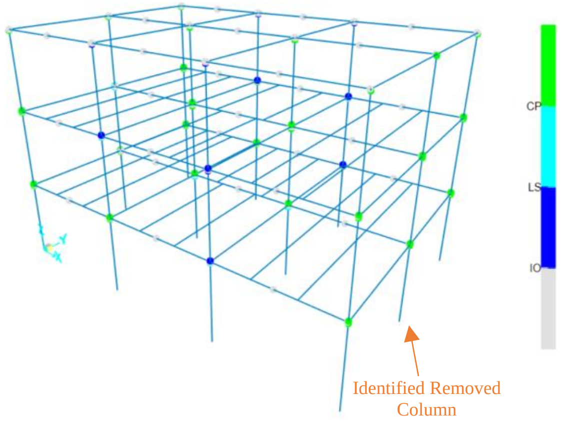

Figure 13. MRF Plastic Hinge Formation: Before Column Removal

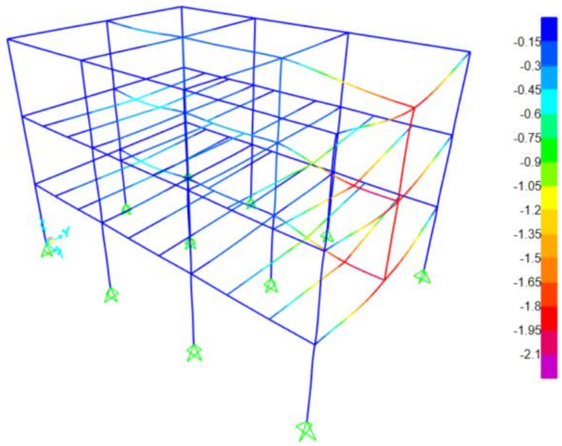

Figure 14. Deformed shape: After column removal (inches)

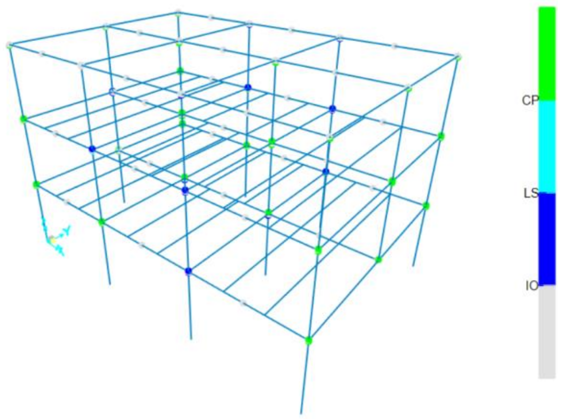

Figure 15. MRF Plastic Hinge Formation: After Column Removal

Steel columns were strengthened with concrete covers at ratios of B/6, B/4, and B/5; B is the dimension of the cross-section, to investigate the impact of this jacketing on resistance

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to progressive collapse.

The results demonstrate that concrete jacketing significantly improves column performance in resisting bending moments across all building stories, consistent with the findings in (Elsanadedy et al. 2014). In Story 1, steel columns without jacketing experience a bending moment (M2) of 45 Kips.ft, whereas columns with B/4 concrete jacketing can achieve up to 76 Kips.ft. In Story 2, concrete jacketing increases the moment from -30 Kips.ft to -85 Kips.ft, and in Story 3, from -31 Kips.ft to -65 Kips.ft, as shown in Table 5.

Despite the increased bending moments with more extensive concrete jacketing, the Demand-Capacity Ratio (DCR) values are lower when accounting for the combined effects of axial force, shear force, and bending moment, as illustrated in Figure 16. The emphasis on M2 is due to the column orientation, as the blast load primarily generates M2 moments.

Table 5. Straining Actions on Column Resulting from Column Removal

Story	Column	Steel Columns			Concrete Jacketing (B/6)			Concrete Jacketing (B/5)			Concrete Jacketing (B/4)		
		P (Kips)	M2 (Kips.ft)	M3 (Kips.ft)	P (Kips)	M2 (Kips.ft)	M3 (Kips.ft)	P (Kips)	M2 (Kips.ft)	M3 (Kips.ft)	P (Kips)	M2 (Kips.ft)	M3 (Kips.ft)
Story 1	W14x43	-102	45	-4	-104	74	-1	-102	75	-1	-101	76	0
Story 2	W14x43	-53	-30	9	-90	-59	15	-85	-62	15	0	-85	-65
Story 3	W14x43	-15	-31	5	-35	-60	12	-32	-62	12	-33	-65	12

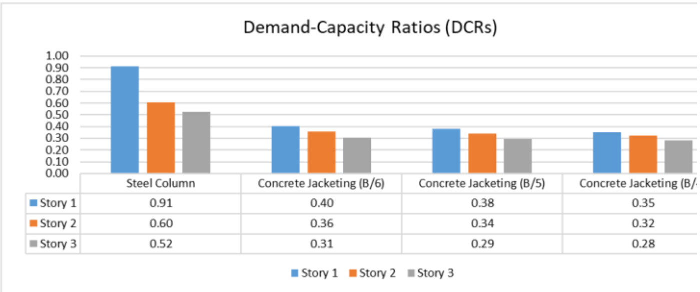

Figure 16. DCR Values Due to Column Removal

During the analysis, the structure dissipated the energy generated by the blast load for all types of columns as shown in Figure 17. Figure 17 illustrates the relationship between base shear and lateral displacement. The integration area of the base shear-displacement curve provides insights into the structure's energy dissipation effectiveness. Steel columns exhibit superior performance in energy dissipation, highlighting their capacity to withstand external forces and minimize potential damage. This aligns with studies on UHPFRC jacketing, which showed improved performance in resisting blast loads and enhancing structural robustness (Li and Aoude 2023).

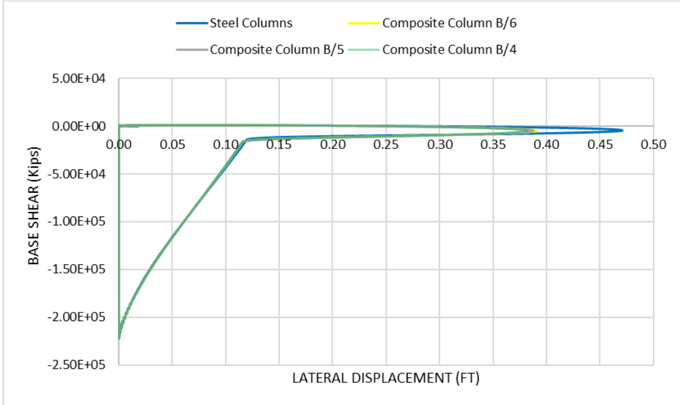

FIGURE 17. Comparison of Energy Dissipation for the Reference System Results (MRF with Steel Columns) and Steel Columns with Varying Concrete Covers

Additionally, lateral displacements were measured for all column types. The results indicated that steel columns with a concrete cover of B/4 exhibited superior performance in resisting lateral displacement, showing a lateral displacement of 0.375 feet at 54.4 milliseconds. In comparison, steel columns without concrete cover experienced a lateral displacement of 0.471 feet.

Comparatively, steel columns with cover B/6 achieved a lateral displacement of 0.386 feet, whereas steel columns with cover B/5 and steel columns achieved maximum lateral displacements of 0.382 feet as shown in Figure 18. Figure 19 presents the relative enhancement in lateral displacement for various Concrete Covers. Columns with a concrete cover of B/4 outperformed those with B/5 and B/6 by 20.31%, 18.82%, and 18.00%, respectively, as shown in Figure 19.

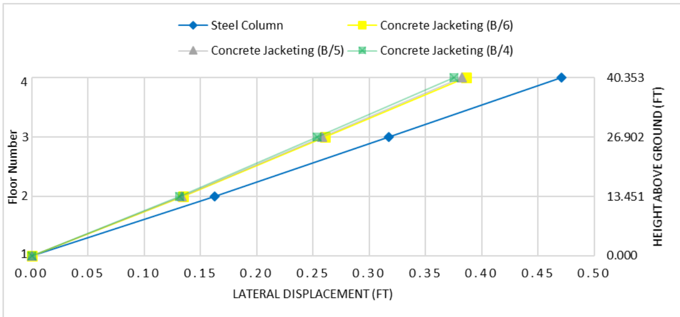

Figure 18. Relative Enhancement in Lateral Displacement for Different Concrete Covers

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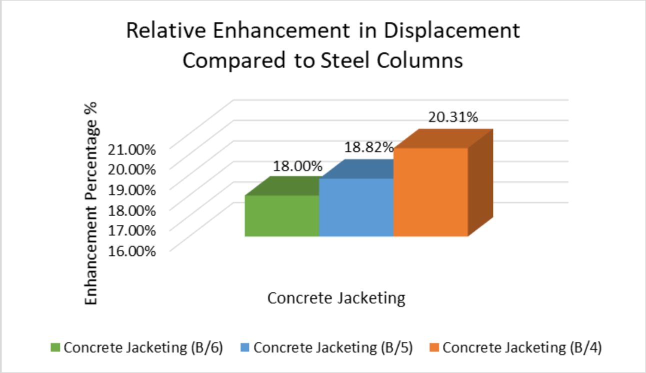

Figure 19. Relative Enhancement in Lateral Displacement for Various Concrete Covers

5. CONCLUSION AND FUTURE RESEARCH

Conclusions:

1. Moment Demand-Capacity Ratios (DCRs): The parametric study found that moment DCRs for columns decrease as the concrete jacketing increases, indicating a reduced potential for progressive collapse. This result supports the recommendations in (Elsanadedy et al. 2014), affirming the structural robustness of the system. Conversely, steel columns showed higher DCR ratios, suggesting the need to reinforce these columns with concrete covers to enhance their failure resistance.
2. Energy Absorption and Lateral Displacement: The analysis demonstrated that while steel columns are more effective at absorbing blast energy, columns with concrete covers are better at resisting lateral displacement. Specifically, columns with a concrete cover of B/4 outperformed those with B/5 and B/6 by 20.31%, 18.82%, and 18.00%, respectively, as shown in Figure 19. This underscores the importance of selecting an optimal concrete cover thickness to improve blast resistance and prevent progressive collapse. These findings align with (Elsanadedy et al. 2014) and (Li and Aoude 2023), which recommend increasing standoff distances and strengthening columns to reduce the risk of progressive collapse.
3. Practical Implications: The results are significant for the design and retrofitting of blast-resistant structures. Identifying the optimal concrete cover thickness allows engineers to implement more effective retrofitting strategies, enhancing building resilience against blasts and progressive collapse. This approach not only improves safety but also has the potential to reduce repair and maintenance costs related to structural failures.

Future Research Areas:

1. Long-Term Performance: Investigate the long-term performance of concrete-encased steel columns under various environmental conditions and repeated blast loads to evaluate their durability and maintenance requirements.
2. Advanced Materials: Investigate the potential of using advanced materials, such as ultra-high-performance concrete (UHPC) or fiber-reinforced polymers (FRP), in concrete jacketing. These materials could significantly improve blast resistance and energy absorption capabilities, offering enhanced protection for structural elements.
3. Finite Element Modeling: Develop comprehensive finite element models that include additional variables, such as varying blast intensities, different column configurations, and building heights, to generalize the findings across diverse structural scenarios.

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