Steel Frame Structure Performance in Blast Environments

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INTRODUCTION

The current practice in blast resistant design of building structures for conventional blast loads is, in general, to design the building envelope (infill and curtain walls) to resist the design blast load and dissipate energy through inelastic response while keeping inhabitants safe from the blast overpressures and debris. These façade components are usually anchored to the building structure at the perimeter beams or floors and roof diaphragms. The dynamic reactions of the façade components are transferred to the building lateral load resisting frames by the floor diaphragms. Ultimately, the lateral load resisting frames must transfer the dynamic reactions from the curtain and infill walls through the connections to the foundation while satisfying performance requirements. For conventional design blast loads, standard practice usually is to assume that the dynamic reactions, transferred to the building structural frame by the façade components, will produce only a moderate lateral response of the building frame. This is due to the relatively large natural period of the structure (compared to the façade components) and inertial mass. However, for larger than conventional threats, specially reinforced façade systems are required which may result in transfer of large lateral forces into the building frame structure. This may result in large force and deformation demands for the lateral load resisting frames.

The study presented herein is part of the "Steel Frame Structure Performance in Blast Environments" research program being carried out by Protection Engineering Consultants for the U.S. Department of State (DOS). The objective of this study is to investigate the implications of the blast resistant design of façade systems on the lateral response of steel frame structures under a specified design blast load environment. One of the guiding principles of this study was to make use of conventional structural analysis tools and methodologies that line up with established consensus criteria so that they can be widely usable and sustainable. The lateral response of different types of mid-rise typical steel frame structures, including moment frames and braced frames, was investigated through nonlinear dynamic analysis using the SAP2000 structural analysis software. Plastic hinge definitions and analysis procedures followed FEMA and UFC guidelines for nonlinear dynamic analysis of structures. The performance of these building frames was evaluated and compared globally and locally through assessment of overall frame response, and strength and plastic deformation demands frame members and connections.

ANALYSIS METHODOLOGY

The purpose of this study is to investigate the response of steel frame structures to the dynamic reactions from façade systems under blast loads. The general analysis approach for this study was based on an uncoupled analysis approach. The use of an un-coupled approach, where reactions from the façade systems are generated independently and then applied to a separate model of the supporting frame structure, was assumed to be a valid approach mainly because of the expected difference in natural periods of response between the components of the façade system and the supporting frame structure as a whole. Typically, exterior components of buildings (such as windows, curtain walls and infill walls) are analyzed for blast response as decoupled from the supporting structure and having rigid boundary (supports) conditions. This approach does not directly account for the interaction between a supported component, which is directly loaded by blast, and the supporting structure, which is loaded by the dynamic reaction forces from the supported component. However, the effects from this interaction are usually greatest when the natural periods of the two components are within a factor of two. The supported components (curtain and infill walls) tend to have relatively short natural periods compared to the longer natural period of the supporting frame structure (generally due to the mass of the building). Therefore, even if the façade components and supporting structure may be deflecting at the same time due to blast loads, the expected supporting frame sway response will be "slow" compared to the response of the supported components and the assumption that the building provides rigid boundary conditions and the response of the frame and façade system are out-of-phase (for the frame structure analysis purposes) is adequate.

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In order to validate this assumption, a preliminary study was conducted for determining the adequacy of the uncoupled approach for the analysis of steel frames subjected to blast loads. Uncoupled and coupled multi-degree-of-freedom (MDOF) nonlinear dynamic analyses, using SAP2000, were conducted on 1-story, 3-story and 12-story steel frames. For the coupled approach, the vertical components (studs) of the façade system were included in the model of the frame structure, spanning between floors diaphragms, and the blast load pressures were applied directly to the vertical studs. In the uncoupled approach, the vertical studs were analyzed under the blast load, independently from the frame structure, using single-degree-of-freedom (SDOF) analysis and the dynamic reactions were determined. Then, the calculated dynamic reactions were applied to the frame structure model at the floor diaphragms at each floor level. The results of the analyses showed that the results obtained with each approach are very similar. In addition, the peak deflections calculated using uncoupled analysis were slightly larger (conservative) than the deflections calculated using coupled analysis. It should be noted, however, that this simplified uncoupled approach does not consider the interaction between the façade system and the framing system, does not consider other blast effects on the structure such as roof and sidewall pressures, clearing effects, and dynamic drag on the back wall of the building, nor does it consider local blast effects on frame components or connections. Nevertheless, it was determined that the uncoupled analysis approach was adequate for this study.

ANALYSIS CASES

Building Description

Steel frames can be designed to resist lateral loads as Moment Frames (MF), Braced Frames (BF), Shear Wall (SW) systems or a combination of these systems. In non-seismic regions, braced frames are the most popular for low and mid-rise buildings because they are often the most economical method to resist conventional wind loads. Different configurations of bracing systems can be used depending on the specifics of each building. The most commonly used brace frame types are: Chevron, Diagonal, and X-braces. Steel Moment Frames and Shear Walls systems are less popular for conventional design but are particularly common for seismic and progressive collapse applications. Seven different steel frame systems, representative of the most commonly used Lateral Force Resisting Systems (LFRS) for steel buildings, were selected to perform 3-dimensional non-linear dynamic analysis. All systems were designed as 6-story buildings. The Chevron and X-Brace systems were also designed for 3 and 12-story configurations to investigate the effects of building height. The selected systems are listed in Table 1.

Table 1. Steel frame cases.

Lateral Force Resisting System (LFRS)	Controlling Lateral Load	Building Heights Investigated
BF: Chevron	Wind	3 story, 6 story, 12 story
BF: X - Bracing	Wind	3 story, 6 story, 12 story
BF: 2 Story X - Bracing	Wind	6 story
BF: Diagonal Bracing (inbound brace in tension)	Wind	6 story
BF: Diagonal Bracing (inbound brace in compression)	Wind	6 story
BF: Eccentrically Braced (EBF)	Seismic	6 story
MF: Moment Frame	Wind	6 story

There are several parameters in addition to the type LFRS that can affect the final configuration, and consequently the response, of steel frame structures subjected to dynamic lateral loads. Some of these parameters are: building layout, story height, shape, location and number of bracing lines, and design criteria. Extensive research would be required to investigate the effects of each these parameters on the lateral response of frame structures. In order to compare the performance of each of the LFRS to each other, the effects caused by the other parameters must be controlled. Therefore, the frame structures for all analysis cases were designed based on the same plan geometry. The baseline floor plan used for the design of all analyzed buildings was based on geometry and characteristics of typical office buildings. The baseline floor plan was designed with 30-foot exterior bays all the way around the building and 20-foot and 22.5-foot interior bays, as shown in Figure 1. The typical floor to floor height was 13.5 ft. The typical floor system was designed as composite concrete deck over steel I-beams. At the roof level, a metal roof deck was used, which is typical for modern construction. The bays are framed by steel girders that span from column to column. Floor joists span between the girders. All Braced Frame (BF) buildings, except for the 12-story building, were designed using two lines of bracing in both axis of the building. Additional bracing lines were required for the 12-story building to satisfy drift requirements. Figure 1 illustrates the baseline floor plan and LFRS location for all the braced frame buildings. The moment frame building has similar floor layout with the LFRS consisting of moment frames all around the perimeter of the building.

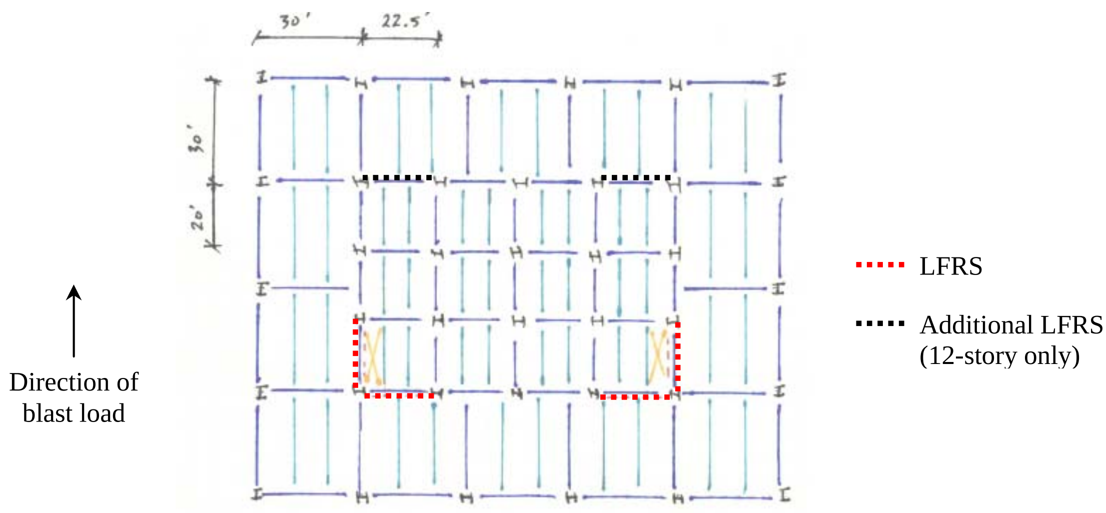

Figure 1. Typical building layout for braced frame systems

The building systems were designed to satisfy all strength and serviceability limits per IBC 2006 [1] and ASCE 7-10 [2], based on typical dead and live loads for office buildings. In addition, to ensure that the baseline lateral stiffness and strength of all the systems were comparable, all buildings were designed based on a basic wind speed of 115 mph for strength and 90 mph (50-year wind) for serviceability, per ASCE 7-10 [2], and a total and interstory drift limit of $h/400$. All steel frame components were designed following conventional design approaches per the AISC Steel Construction Manual [3].

Blast Resistant and Conventional Façade Systems Design

Two types of façade systems were considered in this study: conventional (non-hardened) and blast resistant (hardened). The conventional façade consisted of a typical cold formed steel (CFS) 8-inch deep stud wall system. The hardened façade, on the other hand, was designed to resist the design blast load using the single-degree-of-freedom (SDOF) methodology [4]. The hardened façade consisted of HSS 6x4x½ tubes, spaced at 4 ft on center, with brick veneer wall. For all the 3 and 6-story buildings, the selected façade system design was used uniformly at all floor levels. For the 12-story building, the variation in standoff distance and angle of incidence along the height was considered and, consequently, three different sets of blast resistant façade designs were used along the height of the building.

NONLINEAR DYNAMIC ANALYSIS METHODOLOGY

Determination of Dynamic Reactions from Façade Components

As the first step of the uncoupled analysis approach, the dynamic reactions from the façade components due to the design blast load were determined by SDOF analysis, using the SDOF Blast Effect Design Spreadsheet (SBEDS) [4]. The CFS and HSS studs of the non-hardened and hardened façade systems, respectively, were analyzed as simply supported and uniformly loaded. The conventional non-hardened façade system was expected to fail catastrophically when subjected to the design blast load specified for this study. To simulate the effect of the failure of the CFS studs on the frame structure, the calculated dynamic reaction was truncated at the time when the studs reached a deflection corresponding to blow out, per the response limits specified in PDC-TR-06-08 [5]. Thus, after the studs had reached such level of deflection, the studs were considered to have flown into the building and would no longer transfer load into the structure. The hardened façade system, however, was designed to resist the design blast load without failure or rupture and, therefore, was assumed to be able to transfer the entire dynamic reactions into the supporting frame structure. The calculated dynamic reaction for the hardened façade studs is shown in Figure 2.

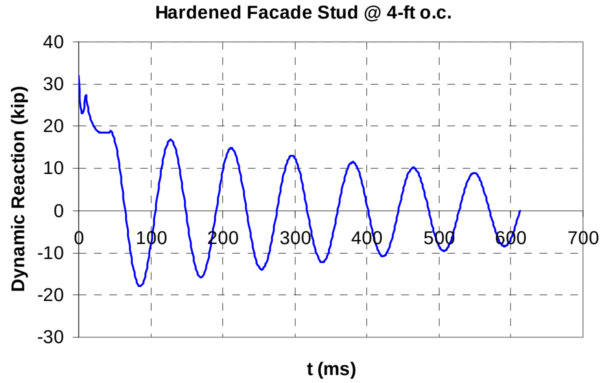

Figure 2. Façade dynamic reaction applied to frame structure models

Dynamic Analysis Methodology

The analysis of all frame structures was performed with the general purpose structural design and analysis software SAP2000, which has nonlinear dynamic (NLD) analysis capabilities. The dynamic reaction shown in Figure 2 was imported into SAP2000 and placed as load time history functions into the structural models for each of the analysis cases shown in Table 1. The dynamic loads were applied to the models as point loads placed every 4 ft O.C. on the floor diaphragms at the blast loaded side of the building, as depicted in Figure 3.

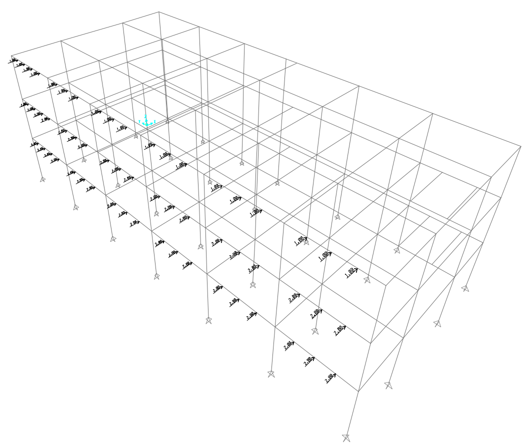

Figure 3. Façade dynamic reactions applied to the frame structure

For the 3 and 6-story buildings, a uniform façade system was used through the building and the maximum blast load at ground level was applied uniformly throughout the face of the building. For the 12-story building, however, the dynamic reactions were determined considering the variations in the standoff and the angle of incidence of the blast load along the height of the building, which results in decreasing blast pressures at the higher stories. This resulted in three sets of blast resistant façade designs and corresponding dynamic reactions at the 1 to 6, 7 to 9 and 9 to 12 stories. Likewise, for the 12 story model, the calculated dynamic reactions were modified to account for variations in the arrival time.

The NLD analysis was performed in SAP2000 using the Newmark time integration method and the default values for the Gamma and Beta parameters, which generally provide good results and convergence times. The mass source of the building was determined in SAP2000 from the applied loads using the 1.2 DL (including self-weight) + 0.5 LL load combination, which corresponds to the extreme event load case per UFC 4-023-03 [8]. A gravity load static analysis, using this load case combination, was performed by SAP2000 to determine the initial condition of the building prior to the dynamic analysis under blast loads. The analysis time-step for each analysis was empirically selected to provide converging results at one percent of the natural period of the building. The natural period was determined by performing a modal analysis and selecting the natural period of the dominating mode of vibration. The dominating mode of vibration was selected visually based on the motion of the structure. P-delta effects were considered for all analyses.

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Nonlinear Model

Material nonlinearity was incorporated into the analyses by using the plastic hinge approach with the requirements in Chapter 5 of the ASCE 41-06 [8] for NLD analysis of steel frames. ASCE 41-06 [8] provides modeling parameters in the form of "Hinge Definitions" which are resistance deflection curves (see Figure 4) that dictate the nonlinear response of numerous types of components such as connections, beams, columns, slabs, and braces. The modeling parameters (a, b and c in Figure 4) provided in ASCE 41 have been developed through extensive research and analysis, and have been vetted by a panel of experts. In addition to modeling parameters for plastic hinges, ASCE 41 also provides modeling guidance for material strength and elastic stiffness of beam, columns and slab elements. Additional information regarding the modeling parameter for NLD analysis of steel frame systems can be found in Chapter 5 of ASCE 41-06 [8] and Chapter 5 in the UFC 4-023-03 [8].

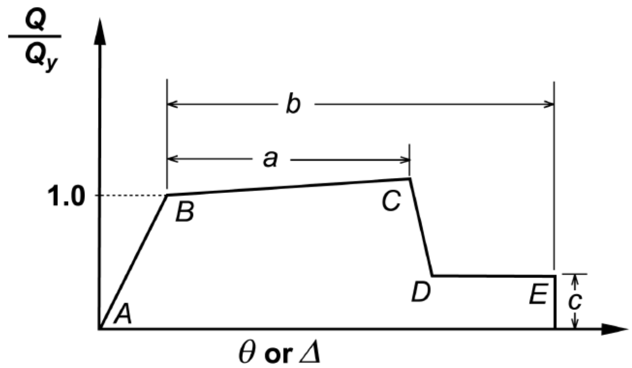

Figure 4. Typical hinge definition (ASCE 41-06 [8])

The beam hinges are moment rotation type hinges and fall into 2 different categories: beam hinges and connection hinges. The beam hinges were placed at the ends and mid-span of all beam elements. Maximum allowable hinge rotation depended on whether the member was a beam or a girder: girders were considered primary members and allowed less rotation, while beams were considered secondary members and allowed more rotation. Connection hinges are based on rotational behavior of connections studied through research and testing. Two types of connection hinges were used: simple shear tabs and Welded Unreinforced Flange (WUF) moment connections. The simple shear tables were used for all beam-column connections in the BF systems, and for the interior beam-column connections of the steel MF system. The WUF moment connection was used for the perimeter connections of the steel MF system. The modeling parameters for the beam hinges were taken from Table 5-6 in ASCE 41-06 [8] for all components except for shear tabs which used the response parameters from Table 5-2 in the UFC 4-23-03 [8]. Note that the shear tabs were modeled as Partially-Restrained (PR) hinges and, therefore, have some moment capacity prior to yielding. Based on tests results observed during the DTRA DISCRETE/LETO test series [8], the shear tabs were defined to yield when the connection reached a moment capacity equal to 30% of the moment capacity of the connecting beam element.

All columns in the model had hinges that considered axial forces and moments (P-M hinges). These P-M hinges were placed at the end of all column elements and were activated whenever the column had any combination of axial loads and moments that exceeded the P-M envelope. Brace hinges were placed mid-span in all brace elements and were activated when the brace reached its axial capacity. Column and brace hinges were automatically generated in SAP2000 based on ASCE 41-06 [8] requirements. For the Eccentric Braced Frames (EBF), shear hinges were placed at the beginning and end of all link beams in the model. The shear hinges are based on the expected shear capacity of the member per ASCE 41-06 [8].

NLD analysis using ASCE 41-06/ UFC 4-023-03 require that the expected material strengths be used. For A500 Gr. 50 steel the yield strength 55 ksi was used (instead 50 ksi). All steel material strength values were also automatically generated in SAP200 based on values from Table 5-3 in ASCE 41-06 [8]. Furthermore, the stiffness for beams with Partially-Restrained (PR) connections were manually adjusted in SAP200 per ASCE 41-07 [8] stiffness modifications to account for the reduced stiffness of the shear tab (compared to the stiffness of the beam). Similarly, the stiffness of the link beam for EBF was also modified. The link beam is designed to deform in shear and therefore ASCE 41-07 requires the stiffness of the material be modified to capture both shear and flexural deformations.

Finally, the limit for assessing failure of a given hinge was based on the Collapse Prevention (CP) limits given in Table 5-6 in ASCE 41-06 [8]. For shear tabs connection failure was based on the plastic rotation limits for primary components in Table 5-2 of the UFC 4-023-03 [8]. The floor joists, roof joists, floor deck and roof deck were considered secondary components and were not included explicitly in the model. The floor system was assumed to have adequate shear strength to transfer the dynamic reactions from the diaphragm into the LFRS.

ANALYSIS RESULTS

Blast Resistant vs. Conventional Façade systems

All 3 and 6-story buildings shown in Table 1 were analyzed under dynamic loads from blast resistant and conventional facade systems.

The results of the SDOF analyses of the façade systems showed that blast loaded conventional façade systems fail very early in the response. As a result, the dynamic reactions transferred to the supporting structure are minimal and did not cause significant response of the LFRS. The catastrophic failure of the conventional façade system would produce blowout of the façade components into the interior space of the building resulting in high level of injury and lethality for the building inhabitants and, therefore, it is unacceptable for providing a minimum level of protection. The hardened façade system, on the other hand, was designed to resist the design blast load without failure or rupture. Therefore, the hardened façade system was able to absorb a significant amount of energy from the directly applied blast loads and transfer it into the supporting frame structure. Consequently, the LFRS was subjected to high dynamic reactions that induced significant lateral response. Therefore, the analysis cases for conventional façade systems are ignored in the rest of this study and only results for hardened façade analysis cases are presented in the following sections.

Direct Comparison of Different 6-Story LFRS

As previously discussed, seven different configurations of 6-story steel frame buildings representative of the most commonly used systems were analyzed against the design blast load specified for this study. All LFRS were designed using identical building layout, geometry and conventional design criteria (lateral drift limited to h/400). All buildings were compared based on the performance of the 6-story height configurations. Therefore, the performance of the 3-story and 12-story Chevron and X-Brace systems are not included in this comparison.

In order to be consistent in the evaluation of the different LRFS and to be able to relate their performance to existing consensus documents, the Structural Performance Levels given in ASCE 41-06 [8] were used to rank the performance of the systems based on interpretation of the results obtained from the analyses. The discussion of structural performance in ASCE 41 is based on structural levels defined in terms of damage description of the components of the LFRS along with recommended drift limits for seismic response of buildings. S-1 is the best possible performance and represents a structural condition after the event where Immediate Occupancy (IO) is possible. S-5 is the least acceptable performance and represents a structural condition after the event where collapse is prevented and the structural system is sufficiently robust to allow evacuation, but occupancy and re-use is unlikely. The performance levels in ASCE 41 are based on seismic loads which are longer duration cyclical loads in nature. Blast loads, as shown in the analysis, will induce cyclical response but the duration is shorter than a seismic event. However, the structural performance levels are likely appropriate because of the cyclical nature of the response. The comparisons provided in this study are all based on a maximum allowable Collapse Prevention (CP) value from ASCE 41, implying no occupancy or re-use of the structures. Should a higher level of performance be desired, the IO or an intermediate level of response (Life Safety) should be considered. For all cases, careful consideration must be given to determining the level at which individual structural components are considered to reach their failure point. For cases that did not clearly fit a performance level, engineering judgment was used to assign a performance level to the system.

Table 2 shows the structural performance levels assigned to each system in order with best performance at the top. The peak total drift was calculated by dividing the maximum peak deflection, measured at the top story of the building, by the building height. As mentioned above, the direct comparison between the different types of LFRS was made using only the 6-story buildings. This was necessary in order ensure no additional parameters affected the response of the systems. The performance levels assigned to the analyzed 3-story and 12-story buildings are provided in Table 3.

Table 2. Structural Performance Level for 6-story frame structures with hardened façade systems

Rank	Lateral Force Resisting System (LFRS)	ASCE 41-06 Structural Performance	Peak Total Drift (Inbound)	Peak Total Drift (Rebound)	LFRS Damage from NLD Analysis
1	MF - Steel Moment Frame (WUF)	S-1	0.60%	0.45%	No hinges observed in moment connections, several hinges yield for PR (shear tabs) but do not fail, low demands in columns
2	BF - Two Story X Bracing	S-2	0.41%	0.40%	No failure of braces, axial demand / capacity for braces in the range of 50%
3	EBF - Eccentric Braced Frame (Link Beam)	S-3	0.45%	0.40%	Link beam shear failures at 3rd and 4th stories, no compression braces failure but high axial demand / capacity rations in the range of 80%
4	BF - Diagonal (Tension)	S-4	0.48%	0.43%	Compression buckling of braces in upper three stories. Axial demand/capacity rations of 66% in remaining bracing components at lower stories
5	BF - Diagonal (Compression)	S-4	0.47%	0.63%	Compression failure of one brace in 5th story and axial demand / capacity rations of 60% to 95% in remaining bracing components
6	BF - X Brace	S-4	0.44%	0.39%	Compression failure of top stories braces and axial demand / capacity ratios of 70% to 86% in lower 3 stories braces
7	BF - Chevron	S-4	0.40%	0.53%	Compression failure of top stories braces and axial demand / capacity ratios of 70% to 80% in lower 3 stories braces

In general all systems resulted in relatively low peak drift values. The drift value for the steel MF system was under the ASCE 41-06 recommended value for Immediate Occupancy (IO) of 0.7%. All Braced Frames, except for the diagonal brace system when loaded in compression, resulted in drift values close or lower than the ASCE 41-06 recommended value for IO of 0.5%. The two-story X-frame provided the best performance of the Brace Frame systems. The two-story X-frame system showed no failure of the bracing components. The maximum axial demands developed on the bracing components reached approximately 50% of the capacity. A demand over capacity ratio of 50% indicates that the bracing components did not approach the failure limit; however, 50% could be considered moderate for components with brittle failure modes (buckling). Therefore, special consideration must be given to the strength of the bracing elements and its connections when considering the use of this type of system.

The Eccentric Braced Frame (EBF) resulted in no failure of bracing components. However, high shear demands and failure were observed in the link beam at the middle floors. The maximum demand recorded in any of the braces was 80% which indicates that the components approached the failure limit. This system showed significantly less failure of bracing elements than the concentric braced frames (BF). Nevertheless, special consideration must be given to the shear capacity of the link beam. The rest of the Braced Frame systems (Diagonal, X and Chevron) showed all similar response modes. All showed compression failures of the braces in the upper floors relatively early in the response before the first peak deflection during inbound. However, the systems were able to resist the following cycles of response without additional formation of hinges. The analyses showed that the bracing elements that reach their failure limits in compression were able to develop tension forces during rebound and, therefore, contribute to the lateral strength of LFRS. Thus, these systems have adequate reserve capacity to resist the design blast load without a high risk for collapse.

The Effects of Building Height on Lateral Response

In addition to the type of LFRS, the effect of building height on the lateral response of LFRS was also investigated for two of the most typical Braced Frame systems: X-Brace and Chevron. These selected systems were analyzed for 3, 6 and 12-story configurations. Table 3 shows the performance levels assigned to the 3-story and 12-story X-Brace and Chevron systems. The performance levels assigned to the 6-story X-Brace and Chevron systems are provided in Table 2 (ranks 6 and 7, among the 6-story systems, respectively).

Table 3. Structural Performance Level for 3 and 12-story frame structures with hardened façade systems

Lateral Force Resisting System (LFRS)	ASCE 41-06 Structural Performance	Peak Total Drift (Inbound)	Peak Total Drift (Rebound)	LFRS Damage from NLD Analysis
12- story BF - Chevron	S-1	0.25%	0.24%	No Damage
12- story BF – X Brace	S-2	0.29%	0.24%	Little Damage, failure of braces in upper two stories, moderate to low axial demands on adjacent columns
3- story BF - Chevron	> S-5	0.50%	1.20%	Complete damage, failure of braces and beam connections at bracing locations
3- story BF – X Brace	> S-5	0.44%	0.90%	Complete damage, failure of braces and beam, high axial load demands in adjacent columns

The results of the analyses indicated that buildings 6-story and higher, designed to meet inter-story and total drift limits of h/400 against typical wind loads, are very likely to resist the design blast load specified for this study with minimal damage. The results obtained from the analysis with blast resistant facades for both Braced Frame systems consistently showed that the performance of the building improved when the building height was increased. For both Braced Frame types the results were similar:

- The 3-story buildings resulted in compression failure of the majority of bracing elements
- The 6-story buildings resulted in failure of some of the bracing components at the upper levels and no significant damage of the braces in the lower 3 stories
- The 12-story buildings resulted in failure of the braces only at the upper two stories for the X-brace system and no significant damage for the Chevron system

Although this might be counter-intuitive to some extent, there are clear reasons for the better performance of taller buildings when subjected to the design blast load. First, under typical circumstances, blast loads decrease in magnitude with building height, where as conventional wind design loads increase with building height. Therefore, the lateral load demands on the LFRS due to conventional wind loads get higher with building height which results in stronger bracing components needed to meet serviceability and strength requirements. In addition to the increase in strength of the bracing components, the added number of stories represents an increase in the gravity load demands on the columns of the lower portion of the building. This results in larger gravity load resisting components such as columns in the bottom portion of the building. As a result, the combined effect of increased lateral wind loads with increased gravity loads results in larger structural components of the LFRS in the bottom portion of the building. Since blast loads are most effective at the bottom of the building, they are directly resisted by the stronger bottom portion of the building and a better performance against blast loads is obtained.

SUMMARY AND RECOMMENDATIONS

Summary and Conclusions

The lateral response of typical steel frame structures, with conventional and blast-resistant façade systems, subjected to a specified design blast load, was evaluated using conventional structural analysis tools and methodologies based on established consensus criteria. Non-linear Dynamic (NLD) analysis of the typical steel frame systems was performed using SAP2000, based on the plastic hinge non-linear approach per the requirements of ASCE 41-06 and UFC 4-23-03. Seven different steel frame systems representative of the most commonly used Lateral Force Resisting Systems (LFRS) were selected and all were analyzed under the dynamic reactions from blast resistant and conventional façade systems subjected to the specified design blast load.

The analyses of conventional (non-hardened) façade systems showed that these façade systems fail catastrophically very early in the response. As a result, the calculated dynamic reactions transferred to the supporting structure were minimal and did not cause significant response of the LFRS with conventional façade systems. However, this is an unacceptable performance since the catastrophic failure of the façade system would produce blowout of the façade components into the interior space of the building, resulting in high levels of injury and lethality for the building inhabitants. The hardened façade systems, on the other hand, were designed to resist the design blast load without failure or rupture and were able to absorb significant amount of energy from the applied blast load and transfer them into the frame structure. Consequently, this study focused on the effect of the dynamic reactions from blast-resistant façade systems on steel frame structures

Results showed that steel moment frames provided the best performance of all considered LFRS, with little or no damage in the components of the LFRS and small demands in the vertical load carrying components. This is due to the inherent ductility and strength capacity of moment frames for resisting lateral loads. Moment frames are particularly common for seismic regions and progressive collapse applications. However, for conventional loads, braced frames are most popular, particularly for low and mid-rise buildings, because they are often the most economical method to resist conventional wind loads. The results of the analyses (with blast resistant façade systems) showed that most steel braced frame types when used in typical office buildings of 6-story and higher, and designed for conventional loads, will be able to resist the blast load specified for this study without a significant risk for collapse of the structure. Furthermore, analysis results for two of the most typical Braced Frames systems (Chevron and X-Brace) configurations consistently showed that the performance of the building improved when the building height was increased from 3 to 6 and 12 stories. This is due to the increase of the gravity and wind design loads with building height, which results in larger structural components of the LFRS required to meet strength and serviceability requirements. The results also showed that the analyzed 3-story LFRS are not able to resist the specified design blast load without likely resulting in significant damage of the lateral and vertical force resisting system.

Recommendations and Future Work

The NLD analysis and the evaluation of the systems performance was based on the plastic hinge non-linear approach per ASCE 41-06. The response limits and performance levels in ASCE 41-06 were developed based on cyclical loads that can degrade and cause fatigue in components after several cycles of response. The results of the analyses performed in this study showed that blast loads will cause some level of cyclical response of the LFRS and, in some cases, the rebound deflection could even exceed the inbound peak deflection. However, the extent of the level of degradation that blast induced cyclical behavior can cause on the components of frame systems has not been investigated. Thus, the use of the Collapse Prevention (CP) limits in ASCE 41-06 is a conservative approach in lieu of more appropriate response limits. Additional research is needed to improve and adapt the plastic hinge modeling parameters and acceptance criteria for frames subjected to seismic loads in ASCE 41-06 to better represent the performance of steel frames under blast loaded environments.

Furthermore, the lateral strength, stiffness, mass and overall response of a frame system is directly dependent on the element sizes, connection types, floor system, floor plan, etc. These parameters could vary significantly on a case to case basis depending on the actual building configuration. This study focused on the effects of LFRS type and building height in the lateral response of blast loaded steel frames. Future research efforts have been recommended to the funding agency to investigate the effects of other parameters in lateral response of blast loaded frames. Protection Engineering Consultants is currently continuing this research work to address the identified research gaps through additional analysis and structural testing.

ACKNOWLEDGEMENT

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