Comparative Progressive Collapse Resistant Design Of A Midrise Steel Building Frame: Mrfs, Trusses, And Diagrids

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Progressive collapse requirements are often prescribed for the design of facilities occupied by U.S. federal agencies or providers of critical services. Notional removal of load-bearing elements throughout the perimeter frame is a common approach that is used to implicitly strengthen the structure in order to resist collapsing when damaged by blast or impact threats from outside the building. Many of these facilities are midrise buildings and are commonly designed for progressive collapse resistance using structural steel due to high ductility and favorable strength-to-weight ratio. This study considers several framing designs for a prototype midrise steel building that are designed first for conventional gravity, wind, and seismic loads and then enhanced to meet progressive collapse requirements. Two iterations of the MRF, which is commonly used for progressive collapse resistant steel buildings, are designed to different levels of allowable plastic deformation per current seismic design criteria. Two systems utilizing continuous trusses are considered: one using a hat truss above the roof and another with a belt truss located within the top story. The diagrid is designed using a multi-story inclined module which was selected to increase load-bearing efficiency in a progressive collapse scenario. The results of this study compare the structural performance and cost-benefit (measured in terms of steel weight and connection requirements) of each design when resisting a nominal removal of load-bearing elements per current progressive collapse resistant design criteria. The MRFs typically experience larger levels of plastic deformation, but the truss systems and diagrid structure demonstrate the ability to redistribute loads over a wider region of the perimeter structure. The truss and diagrid structures also tend to offer
similar performance to the MRFs with less weight though potentially with greater quantity and complexity of connections.
INTRODUCTION

Progressive collapse occurs when relatively localized damage to load-bearing structural elements causes a chain reaction of failures that eventually lead to the collapse of a disproportionately large part of the structure. Several design concepts for progressive collapse resistant buildings have been developed in recent decades in response to events such as the 1968 collapse an apartment building at Ronan Point in the UK (Pearson and Delatte 2005) and the progressive collapse of the Murrah Federal Building due to the 1995 Oklahoma City bombing (Corley et al. 1998). These design concepts include (1) “tying” elements together via added reinforcement or stronger connections to improve structural continuity, (2) providing alternate load paths around locations of likely damage, and (3) designing structural elements to fail in a ductile (i.e., gradual) rather than brittle (i.e., sudden) manner.

Based on methodologies first proposed by Ellingwood and Leyendecker (1978) among others, the most common method for achieving progressive collapse resistance is a direct design approach called the Alternate Path Method (APM), which serves as the centerpiece of the current progressive collapse resistance standards (GSA 2013; DoD 2009). APM is performed via multi-deterministic analyses to selectively enhance structural continuity to prevent progressive collapse. When using current APM for building frames, the structure is subjected to the instantaneous removal of single one-story columns (or other vertical load-bearing elements such as walls or braces) one at a time at several critical locations (both in plan and elevation). The current APM approach is threat-independent, meaning that the column is cleanly removed without any correlation to a potential hazard that caused its removal. As shown in Figure 1, current
APM approaches typically assume that the horizontal framing over the removed column is intact. When evaluating each column removal, the gravity loads supported by that column are dynamically amplified due to a sudden increase in downward acceleration, leading to an additional temporary inertial force which must also be resisted by the structure. The subsequent response to each removal indicates whether the system is able to “bridge” across the removed column and avoid disproportionate collapse (see Figure 1). The current APM approaches are performed on a “pass/fail” design basis—a structure is deemed adequate for collapse resistance if it is able to bridge over the removed column by meeting specified performance criteria. The frame is iteratively analyzed for each removal case until a configuration of member sizes and connection types can withstand “collapse” in accordance with the current design criteria. Any collapse due to the notional removal of a single column is considered to be a disproportionate response (GSA 2013; DoD 2013).

Steel frames are commonly used in progressive collapse resistant building design because of their ductility, strength-to-weight ratio, and design flexibility. In current
practice, flexural bridging by the horizontal framing spanning over the removed (i.e. damaged) columns is typically the primary load path mechanism to resist dynamic load amplification and redistribute the in-situ gravity loads from the removed column to the remaining columns. Moment resisting frames (MRFs), as shown in the simple schematic in Figure 2a, are often the structural system of choice for progressive collapse resistant steel buildings (Kim and Kim 2009) due to the significant preexisting experience within the industry, both in design and research, for these systems. Following the investigation of the Murrah Building collapse, Corley et al (1998) recommended the use of MRFs as a potentially effective means of providing increased ductility and continuity in resistance to local damage. It is worth noting that the only steel frame APM design example provided in the DoD progressive collapse design criteria (DoD 2013) utilizes a special MRF which leverages seismic design concepts from ASCE/SEI 41-06 (ASCE 2007) to enable plastic hinging, and thus energy absorption, in moment-connected horizontal framing over the removed column. The DoD design example reflects current practice by neglecting the potential contributions of the composite slab (Alashker, El-Tawil, and Sadek 2010) and catenary action (Kim and An 2009; Sadek et al. 2010), though both have received attention from recent research studies. In this paper, all designs of progressive collapse resistant steel building frames consider only the contribution of the primary framing and allow the potential for catenary action via plastic hinge models that include axial and moment (P-M) interaction. The contribution of composite action and slab membrane behavior to resisting progressive collapse has not yet reached adequate consensus in the structural design community and is therefore not included in this study.

Implementation of the APM provisions to achieve progressive collapse resistance
in steel buildings with MRFs invariably leads to an increase in the cost of the structural system due to increases in member sizes and moment connections. As a result, there has been an increasing interest in exploring alternative structural systems to MRFs, such as hat, belt, and outrigger trusses, which may be less sensitive to local damage scenarios and serve as collector assemblies for load redistribution. Rather than providing continuous MRFs at every story, a hat truss which extends above the top floor of the building, thus creating a structural parapet (Figure 2b), can be used to provide structural continuity over a removed column at stories below. If architectural requirements permit the placement of perimeter framing between stories, a belt truss can be placed in the perimeter frame at strategic story heights as an alternative to the hat truss without adding to the structural height of the building. Figure 2c shows a belt truss located within the top floor. Often times, belt trusses are used in conjunction with outrigger trusses, which connect the perimeter framing system to the core of the building as part of the lateral resisting system. These truss-based systems have the ability to improve the progressive collapse performance and potentially remain in the elastic range, increasing the structural robustness because they redistribute the loads away from the removal (Melchiorre 2008). A number of studies have been conducted on the effectiveness of using belt and outrigger trusses in high-rise construction to mitigate progressive collapse (Eltobgy 2013; Eltobgy, H. H., Nabil, A., Bakhoum 2013; Kim and Park 2012; Mirhom et al. 2012). The studies concluded that the preferred system considering cost effectiveness and progressive collapse resistance is one that employs a belt truss at the top of the building (Eltobgy, H. H., Nabil, A., Bakhoum 2013).

Another structural system which has been considered in recent studies of
progressive collapse resistant buildings is a diagrid. Diagrids utilize diagonal members to create a diamond grid which wraps around the perimeter of the building (Boake 2014). The diagonal perimeter framing can be designed to resist both gravity and lateral loads (Mele et al. 2014), effectively eliminating the need for perimeter columns and typically requiring less steel than conventional systems (Boake 2014). The diamond-shaped diagrid framing can also offer alternate load paths and load redistribution in the event of a local damage to its perimeter framing. Milana et al (2015) compared different diagrid configurations in both intact and damaged states to evaluate the loss of residual capacity. Kwon et al (2012) evaluated and compared the progressive collapse potential of an existing reinforced concrete moment frame structure with that of a steel diagrid structure. That study showed that after a removal scenario to simulate local damage, the loads carried by the removed elements experienced more widespread redistribution around the building perimeter in the diagrid than in the moment frame.

Though previous studies have individually examined the progressive collapse resistance of MRFs, trusses, and diagrids for a variety of steel building frames, there is very little current guidance on the comparative performance and construction cost of these systems. This study addresses this need by comparing the progressive collapse resistance of the following systems for a 10-story (i.e. “midrise”) prototype frame (see Figure 2):

- Special MRFs designed to two different levels of allowable plastic deformation (Figure 2a)
- Ordinary MRF with a hat truss located above the roofline (Figure 2b)
- Ordinary MRF with a belt truss placed at the top floor (Figure 2c)
Diagrid framing units arranged with a conventional incline pattern (Figure 2d)

Figure 2: Perimeter Framing Systems Examined in this Study

Prior to examining progressive collapse resistance, an iteration of the prototype frame implementing each system is designed for gravity and lateral loads (wind and seismic) corresponding to a fictitious site in southern Maryland near the Washington, DC metro area. Midrise structures, typically defined as having 4 to 12 stories, in this general location represent a significant portion of the building inventory for which progressive collapse resistant design is required in the U.S. Each prototype is designed such that the lateral and progressive collapse resisting systems are co-located in the structural system to provide performance efficiency and direct cost comparison. Each structure is then designed for progressive collapse resistance via the APM procedure in accordance with current DoD criteria (DoD 2013). The results demonstrate the ability of each system to withstand local damage as represented by single-story removal of vertical load-bearing elements. Comparisons of cost-benefit and relative robustness of each system provides valuable perspective on a wider array of design options for resisting progressive collapse in steel-framed buildings.
The prototype midrise building design is based on a structure designed for a previous research effort conducted at the National Institute of Standards and Technology (NIST) on the progressive collapse performance of midrise steel-framed buildings designed for seismic resistance (Sadek et al. 2010). The structure is a 10-story office building, rectangular in plan with dimensions of 30.5 m by 45.7 m (100 ft by 150 ft) with five equal bays in both directions. The first floor height is 5.83 m (17.5 ft), and remaining floor heights are 4.191 m (13.75 ft). The floor plan and elevation of the building are shown in Figure 3. The prototype system was designed for seismic effects without consideration of progressive collapse by providing full-height moment frames on all four sides of the building perimeter. To resist progressive collapse due to selective removal of perimeter framing, structural continuity is needed throughout the perimeter framing. The moment frame configuration used as the baseline structural system for this study, which varies slightly from the NIST design, is shown in Figure 3. Framing consists entirely of steel W-shape sections, and the floor framing is designed as composite with the slab.
For a typical floor, the total design dead load was 3.46 kN/m$^2$ (76 psf), and the design live load was 4.79 kN/m$^2$ (100 psf). For the roof, the design dead load was 2.68 kN/m$^2$ (56 psf), and the design live load was 0.96 kN/m$^2$ (20 psf). The lateral system of the baseline structure consists of perimeter MRFs on all four sides over the height of the frame. A continuous MRF envelope is used on the long face and is terminated at the corner of the short face such that a removal of any single-story column along the building perimeter on the long face at any height location would be supported by MRF framing. Removals on the longer spanning bays pose the greater risk of progressive collapse – removals in the shorter bays can be more easily accommodated by the corner-terminated moment frame.

As previously mentioned the building is located near the Washington, D.C. area in Maryland and was designed for Seismic Category B using a static pushover analysis in accordance with ASCE 7-10 (ASCE/SEI 2010). The wind loads were determined in accordance with ASCE 7-10 (ASCE/SEI 2010) with a risk category III, exposure
category C, and resulting 115 mph (185 kph) wind velocity. The relevant coefficients
selected to determine the wind load profiles are shown in Table 1. The vertical load profile
for both wind and seismic loads are provided in

Figure 4. Two wind profiles were calculated: one for the hat truss prototype (due
to the increased height provided by the hat truss above the roofline) and another for all
other prototypes.

Table 1: Relevant Coefficients for Wind Design

<table>
<thead>
<tr>
<th>Factor</th>
<th>Variable</th>
<th>Value</th>
<th>Local Reference (ASCE 7-10)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wind directionality factor</td>
<td>$k_d$</td>
<td>.85</td>
<td>Table 26.6-1</td>
</tr>
<tr>
<td>Exposure factor</td>
<td>$C_e$</td>
<td>1</td>
<td>Table 7-2</td>
</tr>
<tr>
<td>Topographic factor</td>
<td>$k_{zt}$</td>
<td>1</td>
<td>Section 26.8.2</td>
</tr>
<tr>
<td>Gust effect factor</td>
<td>$G$</td>
<td>.85</td>
<td>Section 26.9</td>
</tr>
</tbody>
</table>

Seismic load profiles were calculated for each of the prototype buildings by accounting
for their pertinent characteristics. The response modification factor (R value) for the MRF
and truss frames were determined using ASCE 7-10 Table 12.2-1 (ASCE/SEI 2010). The
R values were selected to be 8 and 3.5 for the MRF and trusses, respectively. The R value
for the diagrid was selected as 3.64 based on a study by Baker et al (2010).
The focus of this paper is the development of progressive collapse resistance structural systems via the notional removal of one-story columns (or equivalent diagonal framing in the diagrid structure) per the APM procedures outlined in UFC 4-023-03 (DoD 2009). UFC allows the removal of interior columns to be neglected under the assumption that screening is provided at its entrances or other operational security is available, which is common for most facilities that require progressive collapse resistance design. Perimeter columns, which may be susceptible to damage from blast or impact hazards exterior to the building, are therefore the focus for potential removal.

For gravity and lateral design, a standard 2D perimeter frame seismic and wind load evaluation was conducted. This study examines the performance of the longer perimeter face of the building. The short direction frame has smaller bays, which will result in a more robust framing design to resist the same lateral loads. This added robustness combined with shorter bay spans will provide greater progressive collapse resistance than the longer direction frame. Focusing on the longer direction frame will
therefore result in a conservative evaluation of progressive collapse resistance if the ensuing percent increase in framing and connections is assumed for the short direction frames.

Analysis and design of the structures for all load cases in this study was performed using SAP2000 version 17.2.0 (Wilson and Habibullah 1997). As a default, all frame elements were modeled as linear beam elements. For conservative simplification, the perimeter framing was analyzed for gravity and lateral loads as a 2D plane frame model assuming symmetry for lateral loading. All supports at the base of each column are modeled as pinned. The dead and live loads on the frame were determined by using the direct tributary width. Point loads were applied at the locations of incoming girders, and distributed line loads were applied on the beams in the frame.

As is common practice, the prototype building relies on its perimeter framing for both lateral and progressive collapse resistance. This study focuses the performance of the longer North and South perimeter faces of the building for gravity and lateral design followed by progressive collapse evaluation. The short direction frame has smaller bays, which will result in a more robust framing design to resist the same lateral loads. This added robustness combined with shorter bay spans will provide greater progressive collapse resistance than the longer direction frame. Focusing on the longer direction frame will therefore result in a conservative evaluation of progressive collapse resistance if the ensuing percent increase in framing and connections is assumed for the short direction frames.

Using the 2D frame approach with the prototype layout, the four perimeter framing systems were designed to resist both gravity and lateral loads and were then re-
designed for progressive collapse resistance. The design of these systems for gravity and lateral loading will be discussed in detail in the following subsections – the design for progressive collapse resistance will be discussed later in Section 0Error! Reference source not found.. Table 2 summarizes the number of connections and frame weight of the frame systems prior to progressive collapse design. The connections include shear, splice, ordinary moment frame (OMF), special moment frame (SMF), seats, and diagrid nodes (Boake 2014), which are illustrated in c) Column Splice
d) Shear Connection
e) Diagrid Seat Connection
f) Diagrid Node Connection (Boake 2014)

Figure 5. The framing weight is the steel weight of the E-W planar frame for each lateral framing system.

a) Bolted Moment Connection  
b) Welded Moment Connection
c) Column Splice
d) Shear Connection

e) Diagrid Seat Connection

Elevation

Section
f) Diagrid Node Connection (Boake 2014)

Figure 5: Representative Schematic of Connections Considered

Table 2: Connections in Gravity and Lateral Frame Systems

<table>
<thead>
<tr>
<th>Frame Type</th>
<th>Number of Shear Connections</th>
<th>Number of Splice Connections</th>
<th>Number of OMF Connections</th>
<th>Number of SMF Connections</th>
<th>Number of Nodes</th>
<th>Number of Seat Connections</th>
<th>Framing Weight (kN)</th>
</tr>
</thead>
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<tr>
<td>MRF</td>
<td>0</td>
<td>24</td>
<td>100</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>732.12</td>
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<tr>
<td>Hat Truss</td>
<td>38</td>
<td>30</td>
<td>110</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>622.94</td>
</tr>
<tr>
<td>Belt Truss</td>
<td>20</td>
<td>18</td>
<td>100</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>600.95</td>
</tr>
<tr>
<td>Diagrid</td>
<td>12</td>
<td>33</td>
<td>12</td>
<td>0</td>
<td>26</td>
<td>48</td>
<td>706.02</td>
</tr>
</tbody>
</table>

Moment Resistant Frame Gravity and Lateral Design

The special moment resisting frame was developed from the NIST prototype system with reductions in size to meet the lower wind velocity and Seismic Categories of the targeted region. This system uses special moment connections. The final gravity and
lateral resisting moment resisting frame design is shown in Figure 6. The total weight of the frame is 732 kN (165 kips), and 100 special moment connections 24 splices are used.

Figure 6: MRF Gravity and Lateral Design

Truss Systems Gravity and Lateral Designs

Two truss systems, a hat truss and a belt truss, were evaluated. By using a truss system, the structure more naturally/easily redistributes loads and improvements in robustness are expected. The incorporation of a hat truss or belt truss increases the lateral resistance and overall stiffness of the structure.

The hat truss system consists of an ordinary moment frame with a 3.048 m (10 ft) tall flat truss above the roof in the plane of the frame. The belt truss system features diagonal members in the top floor to create the truss. For both truss systems, two
configurations were considered. In the first, the diagonal members pointed upward, like an upside down “V.” In the second, the diagonals pointed downward like a “V.” After an initial gravity design, the weight of the upside down “V” configuration for the belt truss was 9% less than the upwards “V” configuration. Similarly, the configuration with the upside down “V” was lighter for hat truss system was lighter than the upward “V” configuration by approximately 15%. The upside down “V” provides top bracing to the top chord, which reduces the sizes of the members of in the top chord of the hat truss. For this reason the configuration with diagonals pointed upward to an upside down “V” was used in this study. The configurations for each system are shown in Figures 2b and 2c.

In both systems, all of the beams are continuous and the diagonal members in the top floor are pin connected, meaning that the chords of the trusses are continuous while the diagonals are axial members. Both truss systems rely on the floor diaphragm to connect to the core. The increased stiffness of the structure from the trusses allows for the use of an ordinary moment connections in the rest of the system. The top chord of the hat truss is braced every 4.57 m (15 ft) with a 45-degree kicker to the roof diaphragm. The final gravity and lateral resisting design of the hat truss and the belt truss systems are shown in Figure 7. The total weights of the truss systems are comparable. The hat truss system weighs 623 kN (140 kips), and the belt truss system weighs 601 kN (135 kips).
Figure 7: Hat and belt truss gravity and lateral design
Diagonalized grid structures, or diagrids, are structural systems that eliminate the need for columns by introducing diagonal members along the exterior perimeter of the building. Diagrids are a transformation of traditional, orthogonal building systems. The connections used in orthogonal building systems are also modified into nodal connections. Diagrids are able to efficiently resist both gravity and lateral loads due to the diagonalization of the members. As a result, often times, the need for additional lateral resisting systems elsewhere in the building is less for diagrids than traditional building frames.

Boake has explored diagrids extensively in (Boake 2013) (Boake 2016) and (Boake 2014) and provides a platform for understanding and designing the structures. Diagrids are adaptable and innovative structures that have become a more common building system since the early 2000’s. Not only are diagrids structurally efficient, but they are also architecturally significant and sustainable. The diagonal members and beams come together in a way that creates a triangulation on the surface of the building. The modules created by the triangulation are visually dominant and have aesthetic importance. Typically, the size of the module is determined by the number of floors between nodes, or tip to tip of the diagonal formed by two triangles. The efficiency of the diagrid system has been proven to reduce the weight of these structures in comparison to conventional systems (Boake 2014). This increases the sustainability of the building since less steel is required for construction.

The Hearst Tower in New York City, shown in Figure 8, is an example of a diagrid. This building has a module size of 6 stories. The triangulation on the façade of
the Hearst Tower is apparent due to the exposure of the beams at mid height of the module. Other diagrid buildings have a diamond pattern on the surface instead to conceal the beams at mid height of the module. As evident in the photograph, the Hearst Tower features a unique corner treatment. Because there are no columns at the corners of the buildings, the beams at the corners are cantilevered. In this case, the longest cantilever would have been 6.1 m (20 ft). The designers of Hearst Tower solved the issue by creating additional triangulation at the corners for a “bird mouth” effect (Boake 2014).

Figure 8: Hearst Tower, New York, New York [Credit: P. Trasborg]

Nodes occur at the point in the diagrid at which the diagonals come together and meet the floor beams. They are fabricated as separate elements to which the other members connect. In this way, nodes promote axial load paths through the diagonals. Most diagrid structures have been built in Asia and the Middle East (Boake 2016), although they have also been built in the US and Europe. The selection of member types is dependent on the location, so the structures vary worldwide. In the US, wide flange sections are most commonly used. In Asia and the Middle East, concrete filled steel tubes
are preferred (Boake 2016). The selection of the member type and sizes impacts the
design of the nodes.

In recent years, many buildings that have unusual geometries or curves take
advantage of diagrid systems. The triangulation of the surface allows curves and other
forms to be approximated without complicated construction methods. Diagrids are also
used in high rise construction due to the increased stiffness, stability, and subsequent
reduction in drift.

As the diagrid has been emerging as an efficient and versatile system, the amount
of research and exploration done on diagrids has increased. Moon has conducted a
number of studies investigating the optimal angle of inclination of the diagonals as well
as provided design guidelines for diagrids (Moon, Connor, and Fernandez 2007) (Moon
2008a) (Moon 2008b) (Moon 2009). A methodology used to determine preliminary
member sizes for 20 and 60 story buildings is presented. The optimal angles for each
model is investigated by comparing lateral and vertical displacement when different
angles are used (Moon, Connor, and Fernandez 2007). A study exploring the optimal
angle for buildings with 40, 50, 60, 70 and 80 stories and systems with uniform angles as
well as varying angles was conducted. For buildings with aspect ratios of 4 to 9 that use
uniform angles, the optimal angle was determined to be 60 to 70 degrees by comparing
the structure weight. For tall buildings, a system with varying angles that gradually get
less steep towards the top of the structure is most efficient (Moon 2008a) (Moon 2008b)
(Moon 2009).

The diagrid structural system was applied to the 10-story building previously
described. Since the building site is in the U.S., wide flange sections were used. Because
it is a midrise building, only uniform angles were evaluated. Design procedures presented by Boake and Moon were considered in the design (Boake 2014) (Moon 2009).

Unlike the MRF and truss systems which had discontinuities in the moment resisting frame systems, the diagrid systems have a full wrap-around effect. For this reason, the penalty for the full tributary width was not accepted in the design as the faces perpendicular to the frame analyzed will help to resist the loads. The connection of the diagonal members into the nodes is assumed to be pinned to preserve the axial load through the diagonals.

Generally, the module is selected by dividing the height of the frame into equal sections (Boake 2016). Since the building analyzed in this study is only 10 stories, the module was selected by dividing the width of the building into equal segments instead. A module width of 7.62 m (25 ft) was selected. Additionally, the first floor has a taller story height than typical floors. If the diagrid pattern extended down to the first floor with consistent geometry, nodes would occur in the middle of the story rather than at the base. To solve this issue, the first floor was treated with vertical columns rather than diagonal members. Although not typical, this is something that has been done in practice and can be seen in Figure 8.

In previous studies, building heights ranging from 20 to 80 stories were evaluated so there is no data for shorter buildings with smaller aspect ratios; therefore, a small angle optimization study is conducted for the 10 story building. Three systems with 48, 66, and 73 degree angles were considered. The heights of the modules were 2 stories, 3 stories, and 4 stories, respectively. The three systems are shown in Figure 9.
Three systems with three different angles of inclination were designed for gravity and lateral resistance and the total weights were compared. The two story module system (48-degree incline) was the heaviest and was eliminated. A preliminary progressive collapse study was done with the four story module (66 degree incline) and six story module (73 degree incline) systems. A nonlinear analysis with five selective removal scenarios was conducted to evaluate the load redistribution in the systems. Based on this preliminary analysis, the four story module system (66 degree incline) was selected to be used in this study because it was predicted to have the best progressive collapse resisting capacity.

In order to maintain moment continuity in the beams, seat connections are used where a beam meets a single axial member (see Figure 10). At these intersection points, the beam was assumed to pass behind the axial member and be connected to it with a bracket. The splices occur in the beams at the prescribed locations (see Figure 10) to maintain moment continuity throughout the beams and ensure that length limitations are not exceeded. Additionally, for architectural considerations and ease of construction, the axial members are rotated so that the face of the flange faces the exterior for the frame. The final gravity and lateral resisting design of the diagrid frame system is shown in

Figure 9: Diagrid Systems with Various Angles
Figure 10. The final gravity and lateral resisting diagrid system weighed 706 kN (159 kips).

PROGRESSIVE COLLAPSE DESIGN

Following the completion of the gravity+lateral design phase, each framing system was evaluated for resistance to progressive collapse. A comprehensive set of column removal scenarios was performed in accordance with UFC 4-023-03 (DoD 2009) using the alternate path direct design approach. For each scenario, one column at a time is removed, assuming continuity in the beams and their connections above the removal at critical locations throughout the building frame. The UFC requires corner columns and interior columns to be removed. Removal of the column adjacent to the corner column
was also included in this study. The most critical elements are commonly first floor
columns since, in addition to supporting the weight of all stories above, they are usually
the closest in proximity to a surface level blast threat. Columns in the story directly below
the roof, at mid-height of the building, and above a column splice or change in column
size must also be removed to fully comply with the requirements of the alternate path
method (DoD 2009). The primary causes for removing elements near mid-height and the
roof of the building are the potential for small aircraft impact and to prevent any
substantial collapse of the roof, respectively. In cases where diagonal framing members
are present, all elements located “within a distance of 30% of the largest dimension of the
associated bay from the column removal location” must be removed (DoD 2009). If any
component of the framing system fails under a removal scenario, it must be strengthened
or resized to ensure sufficient structural integrity is maintained throughout the frame.

In progressive collapse analysis, components with low axial load ($P/P_{CL} \leq 0.5$) are
classified as deformation-controlled. These components are capable of plastic rotation
according to the models and criteria specified in UFC 4-023-03 and ASCE 41-06. Limits
of plastic rotation for deformation-controlled steel members are provided in Chapter 5 of
ASCE 41-06. Three thresholds of plastic rotation are defined by the criteria in the
following order of severity: Immediate Occupancy (IO), Life Safety (LS), and Collapse
Prevention (CP). For beams subjected to flexure or a combination of flexure plus axial
tension, Collapse Prevention (CP) rotation limits are used. For all other members, Life
Safety (LS) rotation limits are used. Plastic moment capacity and rotational limits of
deformation-controlled elements account for expected strength of the material (in this
case, as defined by Chapter 5 in ASCE 41-06). Components under high axial load
(P/P_{CL} > 0.5, where P_{CL} is the lower-bound axial load capacity) are classified as force-controlled. These components must have a demand-to-capacity ratio less than unity for both combined (1) axial load and bi-axial bending and (2) shear. Capacity for force-controlled elements accounts for lower-bound strength of the material (in this case, as defined by Chapter 5 in ASCE 41-06) as well as all appropriate strength reduction factors according to the material specific design code (in this case, the AISC Steel Construction Manual (AISC 2011)).

A recent study by Marjanishvili and Agnew (Marjanishvili et al. 2006) compared the currently available analysis approaches for conducting progressive collapse analysis via APM, which are listed here in the order of increasing computational complexity: Linear Static (LS), Nonlinear Static (NS), Linear Dynamic (LD), and Nonlinear Dynamic (ND). While each approach has tradeoffs in terms of efficiency and accuracy, the study highlighted the effectiveness of ND procedures (which require the most computational effort) because they more realistically account for the dynamic amplification of the gravity loads as well as the nonlinear, plastic response of the structure to the damage scenario (Marjanishvili et al. 2006). The ND approach typically results in lighter progressive collapse resistant design for most building frames and is therefore used for all APM analyses in this study. Additionally, a nonlinear dynamic analysis avoids the need for prediction of structural behavior. Due to the complexity of the systems in this study, it would be difficult to determine where the dynamic increase occurs for a nonlinear static analysis.

Using the ND analysis method, the vertical force due to conventional loads in the column to be removed is first determined using a static analysis. The column of interest is
then removed and temporarily replaced by a vertical reaction at the top node equal in magnitude to that of the expected vertical force in the removed column. The reaction is removed quickly over a time interval equal to 0.1Tn, where Tn is the natural period of the vertical vibration mode over the removed column. The structure is subsequently allowed to respond dynamically to the removal. SAP2000 (Wilson and Habibullah 1997) was used to calculate the nonlinear time history response which is then used to determine the maximum deformation of each component of the framing system.

Two versions of the Special MRF structure were designed: one using the CP plastic hinge limit as prescribed in the DoD criteria, and another using a more stringent IO limit. Together with the truss and diagrid systems, a total of five progressive collapse resistant design solutions were developed. Table 3 outlines the connection inventory as well as the final framing weight of the progressive collapse resistant design.

Table 3: Connections in Progressive Collapse Frame Designs

<table>
<thead>
<tr>
<th>Frame Type</th>
<th>Number of Shear Connections</th>
<th>Number of Splice Connections</th>
<th>Number of OMF Connections</th>
<th>Number of SMF Connections</th>
<th>Number of Nodes</th>
<th>Number of Seat Connections</th>
<th>Framing Weight (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>MRF CP</td>
<td>0</td>
<td>18</td>
<td>0</td>
<td>100</td>
<td>0</td>
<td>0</td>
<td>972.60</td>
</tr>
<tr>
<td>MRF IO</td>
<td>0</td>
<td>18</td>
<td>0</td>
<td>100</td>
<td>0</td>
<td>0</td>
<td>1121.04</td>
</tr>
<tr>
<td>Hat Truss</td>
<td>38</td>
<td>24</td>
<td>110</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>1073.99</td>
</tr>
<tr>
<td>Belt Truss</td>
<td>20</td>
<td>22</td>
<td>100</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>898.99</td>
</tr>
<tr>
<td>Diagrid</td>
<td>12</td>
<td>32</td>
<td>12</td>
<td>0</td>
<td>26</td>
<td>48</td>
<td>926.07</td>
</tr>
</tbody>
</table>

Moment Resistant Frame Progressive Collapse Design

The MRF system was designed for progressive collapse resistance in accordance with UFC 04-023-03 (DoD 2009). The system was designed so that the plastic hinges do not exceed the Collapse Prevention (CP) limit state as well as the Immediate Occupancy (IO) limit state. Both analyses required 15 removal cases at different locations in the...
frame for UFC compliance. The systems were designed based on the structural response to the removal scenarios. The column removals are shown in Figure 11 and Figure 12. Beam sizes were increased if the plastic hinge rotation exceeded the limit state for any removal case. Column sizes were increased if the demand to capacity ratio was greater than one for any removal case. The final designs for the MRF frame for CP limit state and the MRF frame for the IO limit state are shown in Figure 11 and Figure 12, respectively.

Figure 11: Removal Scenarios and Final Progressive Collapse Design of MRF CP
Plastic hinges formed in the majority of the beams above the removed column for all removal cases. The removals toward the bottom caused the hinge rotation in beams toward the bottom to exceed the limit state and required redesign. Additionally, the removal of the corner column below the roof caused hinging that surpassed the limit state, so the roof beam was also increased significantly. The MRF designed to remain below the CP limit state weighs 973 kN (219 kip) and the MRF designed to remain below the IO limit state weighs 1121 kN (252 kip).

**Truss Progressive Collapse Design**

A progressive collapse analysis compliant with UFC 04-023-03 was performed on the hat truss frame and the belt truss frame (DoD 2009). The analysis involved 15 and 17 removal cases at different locations in the frame for the hat truss and belt truss,
respectively. The UFC states “if any other column is within a distance of 30% of the largest dimension of the associated bay from the column removal location, it must be simultaneously removed as well” (DoD 2009). As such, at the story below the roof in the belt truss system, it was required to remove the truss members that frame into the columns in addition to the columns. The frame was designed based on the response of the system to the removal scenarios. Beam members needed to be larger if the hinge deformation was above the CP range for any removal case. Column and truss members needed to be increased if the demand to capacity ratio exceeded a value of one for any individual removal case. The removal cases and final design of the hat truss is shown in Figure 14. The removal cases and final design of the belt truss is shown in Figure 13.

Figure 13: Removal Scenarios and Final Progressive Collapse Design of Hat Truss
Both of the truss systems performed well for progressive collapse resistance. No IO or CP hinges formed under any of the removal cases for either system. For this reason, ordinary moment connections can be used in the frames. The weight of the progressive collapse resistant designs for the hat truss system and belt truss system are 1074 kN (241 kips) and 900 kN (202 kips), respectively.

Due to the lack of bracing or diaphragm contribution above the roof, the members in the top cord of the hat truss needed to be increased significantly to prevent bucking. The largest member used in this system was W40x324. In practice, kickers extending from the roof to the top cord can be used. In this study, kickers extending from the top chord to the roof at a 45 degree angle were included and conservatively considered to be the same section as the diagonal members.
The diagrid structure was designed for progressive collapse resistance as prescribed in UFC 04-302-03 (DoD 2009). For this system, 36 column removal scenarios were required. According to UFC, “if any other column is within a distance of 30% of the largest dimension of the associated bay from the column removal location, it must be simultaneously removed as well” (DoD 2009). Because the members above the first floor that resist the gravity load are diagonal and meet at nodes, most removal scenarios require the removal of two members. This is shown in Figure 15. The removals occur on floors above the nodes because the diagonal members change size at the node, so it is the location of a column splice. The UFC states that column removals must occur at the “story above the location of a column splice or change in column size” (DoD 2009). The system was designed based on the structural performance of the removal scenarios. Beams were increased if the plastic hinge rotation exceeded the limit state. Columns and diagonal members were increased if the demand to capacity ratio exceeded a value of one. The final design of the progressive collapse resistant diagrid is provided in Figure 15.
Because the system was simplified as a 2D frame, the worst response occurred for the removal scenarios closest to the corner. In reality, the frame in the orthogonal direction would contribute to the resistance of the additional load due to the removed column. In most cases, rather than causing the hinge rotation to exceed the limit states, these removals overstressed the diagonal members.

The weight of the progressive collapse resistant diagrid frame was 926 kN (208 kips).

**COMPARISON OF COST AND PERFORMANCE**

A comparative analysis was done on the three framing system types that were considered in this study. Four gravity and lateral resistant designs and five progressive

collapse resistant frames resulted from this study. Table 4 summarizes the weights of the frames. The first column contains the weights after the gravity and lateral resistant design. The second column contains the weights after the progressive collapse resistant design. The third column contains the percent increase of each system from the gravity and lateral design to the progressive collapse design. The fourth column contains the percent increase of each progressive collapse system weight from the baseline MRF designed for gravity+lateral. The fifth column contains the percent increase of each progressive collapse system relative to the MRF progressive collapse system designed for the collapse prevention limit state.

Table 4: Frame Weights

<table>
<thead>
<tr>
<th>Frame Type</th>
<th>Gravity and Lateral (kN)</th>
<th>Progressive Collapse (kN)</th>
<th>Percent Increase Gravity+Lateral to PC</th>
<th>Percent Increase Relative to MRF Gravity+Lateral</th>
<th>Percent Increase Relative to MRF CP PC</th>
</tr>
</thead>
<tbody>
<tr>
<td>MRF (CP)</td>
<td>732</td>
<td>973</td>
<td>32.85%</td>
<td>32.85%</td>
<td>0.00%</td>
</tr>
<tr>
<td>MRF (IO)</td>
<td>732</td>
<td>1121</td>
<td>53.12%</td>
<td>53.12%</td>
<td>13.24%</td>
</tr>
<tr>
<td>Hat Truss</td>
<td>613</td>
<td>1012</td>
<td>64.96%</td>
<td>45.59%</td>
<td>3.87%</td>
</tr>
<tr>
<td>Belt Truss</td>
<td>601</td>
<td>899</td>
<td>49.59%</td>
<td>27.77%</td>
<td>-8.19%</td>
</tr>
<tr>
<td>Diagrid</td>
<td>706</td>
<td>926</td>
<td>31.17%</td>
<td>27.47%</td>
<td>-5.02%</td>
</tr>
</tbody>
</table>

The belt truss system was the lightest for gravity and lateral resistance. By incorporating a belt truss in the top story, the beams elsewhere in the system could be decreased, since the belt truss takes on and redistributes load throughout the frame. The heaviest gravity and lateral design was the MRF. As expected, the diagrid was lighter than the MRF. The hat truss required the most additional steel above its gravity+lateral design to achieve progressive collapse resistance, and the diagrid and MRF CP required the least. The belt truss and MRF IO required a similar amount of additional steel above their gravity+lateral designs to achieve progressive collapse resistance. For progressive
collapse resistance, the MRF IO required 13.24% more steel than the MRF CP, and the
hat truss required 3.87% more steel than the MRF CP. Both the belt truss and the diagrid
required less steel than the MRF CP with 8.19% and 5.02% less steel, respectively.

For progressive collapse resistant design, the belt truss system was the lightest, but it was not much lighter than the diagrid. While the hat truss and belt truss systems were relatively close in weight for the gravity and lateral design, there is a substantial difference in the weights of the systems for progressive collapse design. The MRF designed for immediate occupancy was the heaviest, but the hat truss system was similar in weight.

The belt truss system and diagrid are both 6 to 9 percent lighter than the MRF designed for CP. According to UFC, the systems must remain below the CP limit states for all removal cases. However, both the belt truss system and diagrid prevented the formation of CP hinges for every removal case due to their ability to redistribute loads. In order to achieve the similar performance, the MRF would have a weight increase of 13.12 percent. This suggests that the truss and diagrid systems are more efficient than the MRF system that is used commonly in design.

While the truss and diagrid systems may be more efficient than the MRF frames, these systems require more connections. Each connection has a different fabrication time because of the different amount of welding or bolting required. The time required for a shear or splice connection is one hour, a seat connection is two hours, an ordinary or special moment frame connection is four hours, and a node is 32 hours (Troutman, 2017).
Table 5: Connections in Progressive Collapse Frame Systems

<table>
<thead>
<tr>
<th>Frame Type</th>
<th>Number of Shear Connections</th>
<th>Number of Splice Connections</th>
<th>Number of OMF Connections</th>
<th>Number of SMF Connections</th>
<th>Number of Nodes</th>
<th>Number of Seat Connections</th>
<th>Frame Fabrication Time</th>
</tr>
</thead>
<tbody>
<tr>
<td>MRF CP</td>
<td>0</td>
<td>18</td>
<td>0</td>
<td>100</td>
<td>0</td>
<td>0</td>
<td>420</td>
</tr>
<tr>
<td>MRF IO</td>
<td>0</td>
<td>18</td>
<td>0</td>
<td>100</td>
<td>0</td>
<td>0</td>
<td>420</td>
</tr>
<tr>
<td>Hat Truss</td>
<td>20</td>
<td>24</td>
<td>110</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>486</td>
</tr>
<tr>
<td>Belt Truss</td>
<td>20</td>
<td>22</td>
<td>100</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>444</td>
</tr>
<tr>
<td>Diagrid</td>
<td>12</td>
<td>32</td>
<td>12</td>
<td>0</td>
<td>26</td>
<td>48</td>
<td>926</td>
</tr>
</tbody>
</table>

The fabrication time for a node is considerably larger than the rest of the connections due to the complexity of this type of connection. Nodes require significantly more welding and bolting because six members come together at one point, four of which are at an angle. This is demonstrated in Figure 5. Because of this, the total fabrication time of a diagrid frame greatly exceeds that of the other systems. The MRF systems have the least number of connections and have the fastest total connection fabrication times. The connection fabrication times for the truss systems are comparable to the MRF systems.

In order to assess the performance of each system and compare them, a robustness analysis was conducted. The analysis was carried out using a proposed robustness analysis for progressive collapse resistance systems, which compares the load at which the undamaged structure collapses to the load at which the damaged structure collapses, as a guide. The equation for the proposed relative robustness index (RRI) is shown below:

\[
RRI = \frac{L_{\text{damaged}} - L_{\text{design}}}{L_{\text{intact}} - L_{\text{design}}} = \frac{L_{\text{damaged}}}{L_{\text{design}}} - 1 = \frac{\lambda_{\text{damaged}} - 1}{\lambda_{\text{undamaged}} - 1}
\] (1)
where $L_{design}$ is the total design load, $L_{damaged}$ is the total load that causes the damaged structure to fail and $L_{intact}$ is the total load that causes the undamaged structure to fail (Fallon, Quiel, and Naito 2016). In this study, the design load was the expected design load for a progressive collapse event. $\lambda_{damaged}$ indicates the percentage of additional load above the expected design load that can be applied to the damaged structure before it collapses. The RRI is a normalized ratio comparing the damaged structure to the undamaged structure. The robustness analysis was performed for three column removals on the first floor: (1) corner, (2) adjacent to corner, (3) interior. The dynamic reaction factor (DRF) for each column removal was also determined. The DRFs were determined by comparing the total load, which includes the dynamic load increase, on the damaged structure at failure, to the expected design load on the undamaged structure. A DRF close to one indicates that the system has high plasticity but experiences more damage. A higher DRF means the system is stiffer but experiences less damage. The results are provided in Table 6.

Table 6: Robustness Analysis Results of Progressive Collapse Systems

<table>
<thead>
<tr>
<th></th>
<th>$\lambda_{undamaged}$</th>
<th>Column Removal</th>
<th>$\lambda_{damaged}$</th>
<th>RRI</th>
<th>DRF</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>MRF CP</strong></td>
<td>2.90</td>
<td>1</td>
<td>1.20</td>
<td>10.5%</td>
<td>1.25</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>1.15</td>
<td>7.9%</td>
<td>1.28</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>1.15</td>
<td>7.9%</td>
<td>1.27</td>
</tr>
<tr>
<td><strong>MRF IO</strong></td>
<td>3.50</td>
<td>1</td>
<td>1.30</td>
<td>12.0%</td>
<td>1.49</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>1.25</td>
<td>10.0%</td>
<td>1.51</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>1.25</td>
<td>10.0%</td>
<td>1.47</td>
</tr>
<tr>
<td><strong>Hat Truss</strong></td>
<td>2.95</td>
<td>1</td>
<td>1.10</td>
<td>5.1%</td>
<td>1.44</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>1.10</td>
<td>5.1%</td>
<td>1.64</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>1.13</td>
<td>6.7%</td>
<td>1.71</td>
</tr>
<tr>
<td><strong>Belt Truss</strong></td>
<td>3.00</td>
<td>1</td>
<td>1.14</td>
<td>7.0%</td>
<td>1.62</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>1.15</td>
<td>7.5%</td>
<td>1.71</td>
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<tr>
<td></td>
<td></td>
<td>3</td>
<td>1.18</td>
<td>9.0%</td>
<td>1.80</td>
</tr>
<tr>
<td><strong>Diagrid</strong></td>
<td>2.75</td>
<td>1</td>
<td>1.35</td>
<td>20.0%</td>
<td>1.62</td>
</tr>
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<td></td>
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<td>2</td>
<td>1.30</td>
<td>17.1%</td>
<td>1.58</td>
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<td></td>
<td></td>
<td>3</td>
<td>1.65</td>
<td>37.1%</td>
<td>1.90</td>
</tr>
</tbody>
</table>
The $\lambda_{\text{undamaged}}$ and $\lambda_{\text{damaged}}$ values show how much more load the undamaged and damaged frames, respectively, can carry above the design load. The $\lambda_{\text{undamaged}}$ values indicate that the MRF IO system can resist 3.50 more weight than it was designed for, the most of all the systems, and the diagrid can withstand 2.75 more weight than it was designed for, the lowest of the systems. However, the $\lambda_{\text{damaged}}$ values indicate that the diagrid can withstand the most additional load in the damage state, suggesting that it is the most robust system considered.

The MRF CP does not have a high RRI, but the DRF is low, meaning that there is less load amplification from the dynamic effects, and the system absorbs the energy through the hinging in the beams. While the MRF IO was designed to withstand a greater level of damage, the increase in load that can be resisted by the system is only greater than the MRF CP by 2.1%-2.5%. The DRFs for the MRF IO are also greater than the MRF CP, so the MRF IO is stiffer because it does not utilize as much of the moment-rotation ductility in the hinges. Both the belt and hat trusses are stiff systems as indicated by the high DRFs, and have RRIs of less than 10% for each column removal. These systems were not designed to withstand much more than the design loads. On the other hand, the diagrid has RRIs ranging from 17.1% to 37.1%, which is notably higher than all of the other systems. That being said, the diagrid has the highest DRF of all of the systems because of the increased stiffness. The diagrid has three to four times more robustness because of the load redistribution in the system.

The deflected shape due to the progressive collapse load supports the conclusions drawn from the robustness analysis. Figure 16 shows the deflected shape of the MRF CP, belt truss, and diagrid systems. The deflection of each system is shown at 8x
All of the beams above the removal in the MRF clearly hinge, indicating that the system absorbs energy and is damaged in the extreme load event. The belt truss and diagrid have significantly less deflection, indicating that these systems are stiffer than the MRFs and redistribute the energy across the frame. The elasticity in these systems results in greater deformations above the removal case than the truss and diagrid systems. No hinging occurs in either the belt truss or diagrid system, and the deflections are significantly lower than the MRF. As shown in Table 6, the truss and diagrid systems experience higher dynamic increase effects, and this is because the systems are stiff and redistribute the loads to the supports rather than absorb the energy by hinging. The stiffness in the truss and diagrid systems results in smaller deformations than the MRFs.

For the MRF CP design in this study, the DRFs and RRIs were both low, meaning that there is limited robustness in the system but a high level of plasticity. The MRF IO offers a smaller level of damage and increased possibility of operability after an event with a 13.24% increase in the steel weight. The relative robustness of the MRF IO is only approximately 2% greater than that of the MRF CP, indicating that the additional weight
and lower level of damage is not worth the limited increase in performance. In order to withstand a larger load, the girders over the removal would have to be increased. Multiple column removals that are likely to occur during a threat dependent progressive collapse event would undoubtedly cause both the MRF CP and MRF IO to fail.

CONCLUSIONS AND FUTURE WORK

The current state of practice in progressive collapse resistance for steel buildings centers on the use of seismic-based moment resisting frame solutions to distribute loads due to the removal of a load bearing element. Typically, MRFs are designed for progressive collapse to allow plastic hinging up to a collapse prevention rotation limit in accordance with seismic design standards. This study explores alternative structural systems, which may offer improved robustness or economy, not only in response to the damage scenarios prescribed in the current design standards, but also to a wider range of more realistic damage as well. Mid-rise buildings constitute a large segment of the building inventory but have received relatively little research attention as compared to low or high rise buildings. This study uses a 10-story prototype building frame that is designed for a SDC-B and 115 mph maximum wind load, which correspond to a generic site in the Washington, D.C. area, where many mid-rise buildings are constructed with progressive collapse considerations. The lateral system and progressive collapse resistance are co-located at the perimeter of the building, with the primary damage scenarios occurring due to exterior blast or impact threats. A baseline design of the prototype was designed as an ordinary moment frame to resist gravity and lateral loads. This structure was then enhanced to resist progressive collapse. This study compares the
performance of the following structural framing systems to resist progressive collapse due to standard damage scenarios: (1) Special Moment Resisting Frame with two prototypes designed to the CP and IO seismic damage limits, (2) Ordinary Moment Resisting Frame with a 10-foot hat truss above the rooftop, (3) Ordinary Moment Resisting Frame with a belt truss at the top floor, and (4) a Diagrid framing system with a 4-story module (66 degree angle of inclination).

The MRFs offer a well-established design option for progressive collapse resistance. The MRF was designed to both the CP limit, as is allowed by current design standards, and the IO limit, to limit damage and increase the likelihood of preserving more of the building functionality after the damage event. The CP design required 32.85% more steel weight than the baseline gravity+lateral MRF design as well as an upgrade from ordinary to special moment connections. Achieving an IO limit requires a 13.24% increase in steel weight above the CP design, which is 53.12% above the baseline MRF, using comparable special moment connections. While the MRF IO frame limits the damage and promotes operability after an event, the additional steel weight required is significant. A steel MRF designed to a higher seismic design category would naturally incur a lower increase in steel percentage to achieve progressive collapse resistance. While the MRF IO frame is a heavier system and will have less damage than the MRF CP frame, its efficiency is almost as sensitive the MRF CP frame to a removal. The relative robustness of the MRF CP and MRF IO are comparable, as the robustness of the IO exceeds that of the CP by only a few percent. The additional weight required to achieve the IO limit state in an MRF is not worth the limited performance increase. If a higher level of damage can be accepted, the MRF CP system is more efficient and absorbs more
energy during a removal scenario.

The placement of a continuous truss across the top of the building can enable the designer to avoid the need for special moment framing while still providing adequate progressive collapse resistance. The truss systems provide an alternative design that remains below the IO damage threshold at a similar weight to the MRF CP. They also eliminate the need for special moment connections because the beams do not hinge, so ordinary moment connections are used instead. The hat truss would typically be used to either (1) avoid the placement of truss framing across or behind the façade at occupied floors or (2) offer a retrofit system that can be overlaid on top of an existing frame. In this case, the hat truss prototype was designed as a new design rather than a retrofit, meaning that the structure incorporates the hat truss into its gravity+lateral resistance. The hat and belt trusses provide progressive collapse resistance with IO hinge rotation in the ordinary moment frame connections; however the members in the top chord of the hat truss are massive in order to withstand the load incurred by the removal scenario, presenting challenges for constructability. To achieve a more efficient hat truss, the depth of the truss may have to be increased. The hat truss system is able to achieve this performance level at similar frame steel weights to that of the MRF IO design. The belt truss is able to provide this performance level at weights 8% lower than the MRF CP design, making it the lightest of the five systems considered in this study. Despite the lower level of damage and weight advantage of the belt truss, neither of the truss systems have a high level of robustness.

Three diagrids with different module sizes, two, four and six story with the same module width, were examined. The systems were first designed for gravity+lateral, and
the two story module was eliminated due to weight. Preliminary removal scenarios were
performed for the remaining two systems, four and six story modules. The diagrid with
the four story module was predicted to be the most efficient at redistributing loads and
was therefore chosen for the progressive collapse resistant design. The diagrid is able to
achieve an IO performance level with a steel frame weight 5% less than the MRF CP
design, although it requires twice the fabrication time due to the complexity of the
connections; however, the diagrid system simplifies the load transfer because the nodes
transfer only axial force and not moment, so there is a greater potential for load
redistribution. Compared to the other frames, the diagrid has three to four times more
robustness because of the load redistribution in the system. A diagrid also presents an
architectural investment due to the flexibility in the design and ability to approximate
complex shapes. This study only considered a simple frame, and a diagrid may be more
efficient than the MRF and truss systems in a more complicated system, even if the only
complication in the frame was a setback for a door. The diagrid provides increased
flexibility of design, and further research would need to be conducted in order to
investigate this as well as the constructability and economy of scale of these systems.

The MRF and truss systems have low RRLs because they are designed to just meet
the demand of one column removal. A load perturbation would greatly affect these
systems. In a threat dependent scenario, it is likely that more than one member would be
completely or partially damaged (Gombeda et al. 2016). From a threat dependent
perspective, the MRF and truss systems are unreliable because if even just a small
increase in the design load occurs, the system would fail. Under a multiple removal
scenario, both truss systems would fail, but it is unclear whether the failure would occur
in the truss members, the columns, or the girders. Further research would have to be conducted to evaluate the failure of belt truss and hat truss frame systems to evaluate the failure modes in such a case. The truss systems offer the option to engineer for a threat dependent scenario that may require an increase in the system capacity. In that case, a penalty in weight and increased DRF would have to be accepted. Conversely, the diagrid can withstand 20-40% of the expected design load. Because of the large relative robustness of this system, the diagrid may be able to easily withstand the load incurred due to multiple removals. In such a threat dependent scenario that could cause multiple column failures, the diagrid offers an increased ability to engineer the system above the rest of the systems considered. The diagrid system works like a web and allows for selective strengthening, inherently giving it progressive collapse resistant capabilities and offering more ability to engineer the system.

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