# Framing Strategies for Enhanced Robustness in Steel Buildings

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## ABSTRACT

This paper describes the use of stiff-story framing to increase robustness in steel buildings subjected to column loss. Two case study buildings were designed; building A features a perimeter moment frame structure, while building B utilizes chevron-braced frames. The lateral-force-resisting systems (LFRSs) and stiff stories of these two prototype buildings were then modified to create different configurations in order to study which configurations were more robust. A linear static alternate path analysis was performed for each of the configurations, and the effectiveness in resisting column loss was determined. Based on the results from the analyses, two indices that quantify vulnerability and framing efficiency were developed that can be used by designers to evaluate framing alternatives. Cost and aesthetic considerations were explored as well. Evaluation of alternative configurations shows that tying the stiff story into the LFRS and supporting all exterior columns with the stiff-story framing will increase system robustness.

**Keywords:** stiff story, disproportionate collapse, robustness, column loss, alternate path analysis.

## INTRODUCTION

Typical beam-column connections and composite floor deck details in steel gravity frames may not have sufficient robustness to support design loads in the event of a column loss (Main and Sadek, 2012; Francisco, 2014; Johnson et al., 2015; Weigand and Berman, 2014). These gravity frame connections are exclusively designed to resist shear forces; however, in the event of a column loss, very large axial loads develop in these connections, potentially inducing failure. This problem has motivated researchers to investigate “best practices” for reducing risk of disproportionate collapse, including using fully restrained connections for all beam-column connections and sizing all connections to resist beam axial tension (Ellingwood et al., 2007). Other researchers have created enhanced connections (Weigand, 2014; Liu, 2010) capable of resisting the large axial loads. In addition to research on ways to improve robustness locally, by increasing capacity of beam-column connections or in floor slab details, some have suggested a more global approach by means of alternative framing solutions. The use of a strong floor or stiff story, intended to provide distributed stiffness, has been suggested as an effective means to avoid limit states such as connection fracture and, subsequently, disproportionate collapse in a column loss scenario (Carter, 2011).

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This paper studies the use of lateral framing at strategic locations combined with stiff stories to create alternate load paths in the case of column loss. A stiff story is a stiff and strong system, usually a full story high, capable of redistributing loads from a lost column below. Figure 1 shows a three-story, three-bay frame with a stiff story at the roof. The stiff story limits beam-column connection deformations and transforms the compressive force in the columns above the missing column to a tension, or hanger, force. Thus, the gravity load is redistributed through the stiff story to the adjacent columns, as shown by the arrows in Figure 1.

This paper presents an investigation of the feasibility of using stiff-story solutions integrated with typical lateral-force-resisting systems (LFRSs) to increase robustness in steel buildings. The use of stiff stories is not a novel concept because it has been used in industry in the past. For instance, Weidlinger Associates renovated an existing office building through use of hangers and ring beams at the roof to support columns below in the event of a column loss scenario (Naderi et al., 2015). However, this paper will explore various configurations and different measures for identifying effective framing strategies that can be utilized by the designer. The emphasis of the paper is exterior column loss, and therefore, stiff stories are not employed at interior gravity frames. The paper begins with a discussion of the case study buildings and the corresponding configurations derived from these buildings. A brief summary of the alternate path method used to analyze the buildings is then presented, followed by the analysis results. These results were used to develop indices that quantify the vulnerability, robustness and efficiency of these configurations.

## CASE STUDY BUILDINGS

Three categories of stiff-story systems in existing buildings were studied: cantilever/hanging perimeter, large central span, and cantilever truss, as shown schematically in Figure 2. These buildings rely on stiff stories above to distribute gravity loads due to fewer gravity columns at the first floor. This concept inspired the two stiff-story buildings designed for this case study, referred to as building A and building B, which incorporate both traditional and stiff-story framing.

The American Zinc building in St. Louis, Missouri, influenced the design of case study building A. American Zinc features an exterior Vierendeel truss system designed to transfer forces due to a large center span, similar to Figure 2b. For building A, this center span was eliminated to create a more traditionally framed steel building by incorporating additional columns at the ground level; however, the geometry and dimensions of the Vierendeel truss were replicated in building A to create stiff stories. The case study structure is a four-story office building. All perimeter framing is moment connected. A typical floor plan and exterior elevations are shown in Figures 3, 4 and 5, with triangles representing moment connections.

The second building studied, known as building B, was inspired by the Lamar Construction Corporate Headquarters, located in Hudsonville, Michigan, which features a large cantilever and Pratt truss system similar to Figure 2c. Again, the large span was removed by adding columns at each column line; however, the stiff-story truss of the Lamar Construction Corporate Headquarters building was incorporated into the design of building B. This case study building is a four-story office building with perimeter chevron-braced frames. Refer to Figures 6, 7 and 8 for the building layout and member sizes.

Each building was designed to comply with ASCE 7-10 and the AISC *Specification,* based on the locations and loads of the building for which it was inspired (ASCE, 2010; AISC, 2010). The dead loads for buildings A and B are 61 and 75 psf, respectively. Building A was conservatively designed for 100-psf live load due to the open layout. For building B, an 80-psf live load was applied per the original

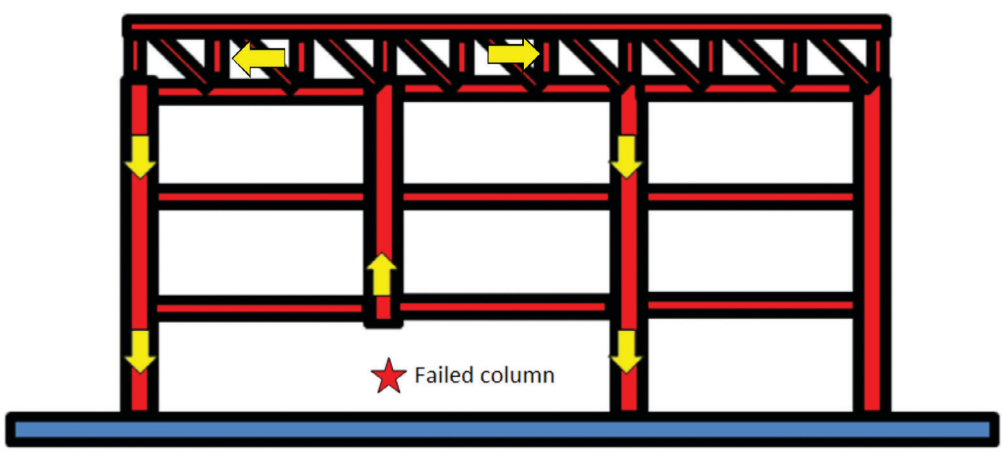
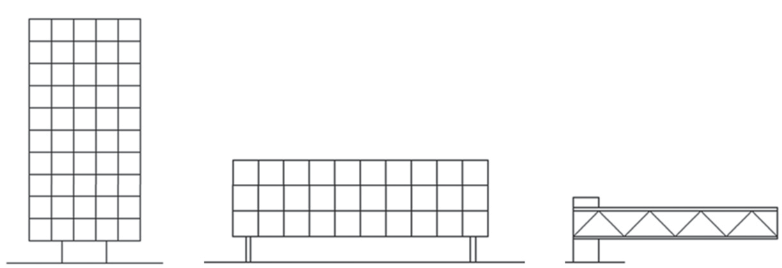



*Fig. 1. Column loss in a building with a stiff story.*



(a) (b) (c)

*Fig. 2. Structural system: (a) cantilever/hanging perimeter; (b) large center span; (c) cantilever.*

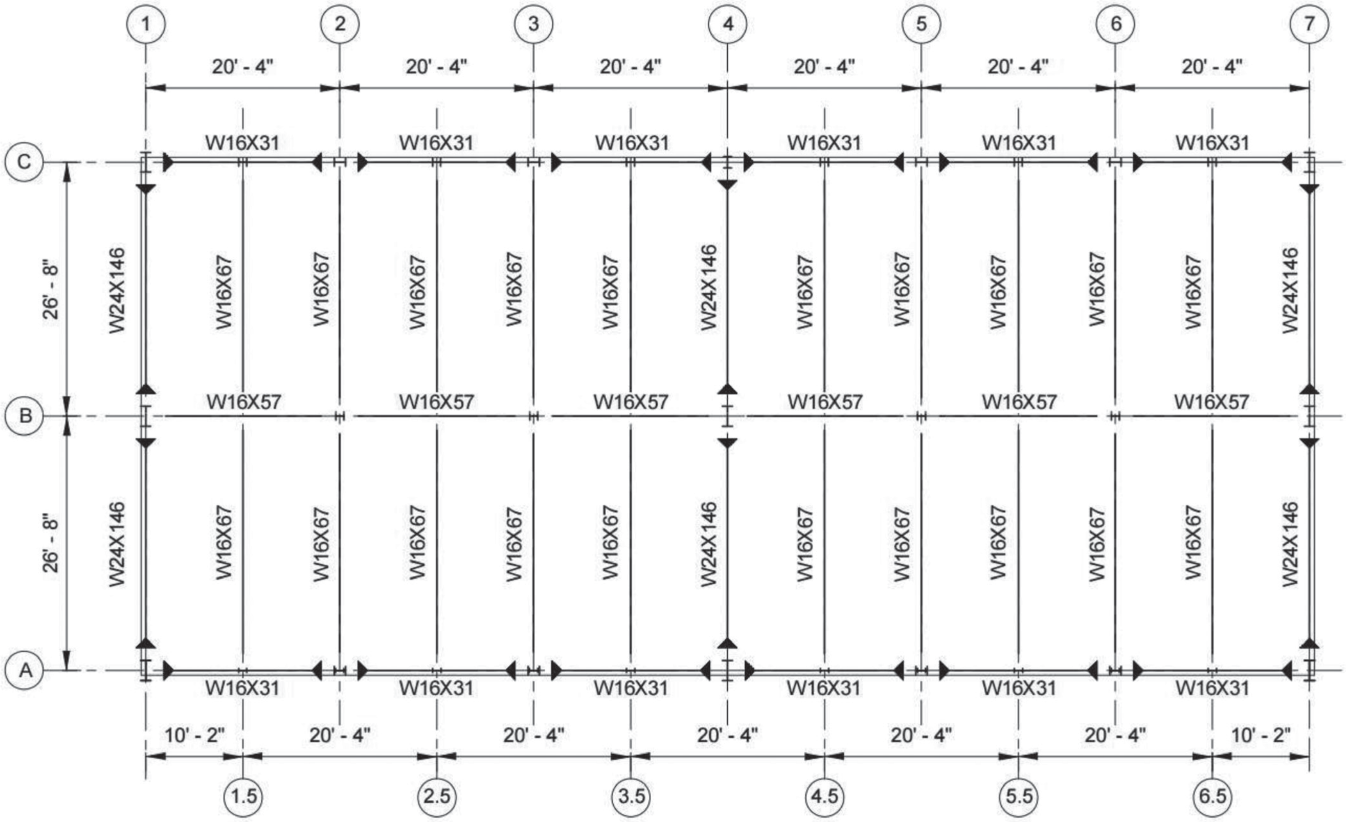



*Fig. 3. Plan view (level 2) of building A.*

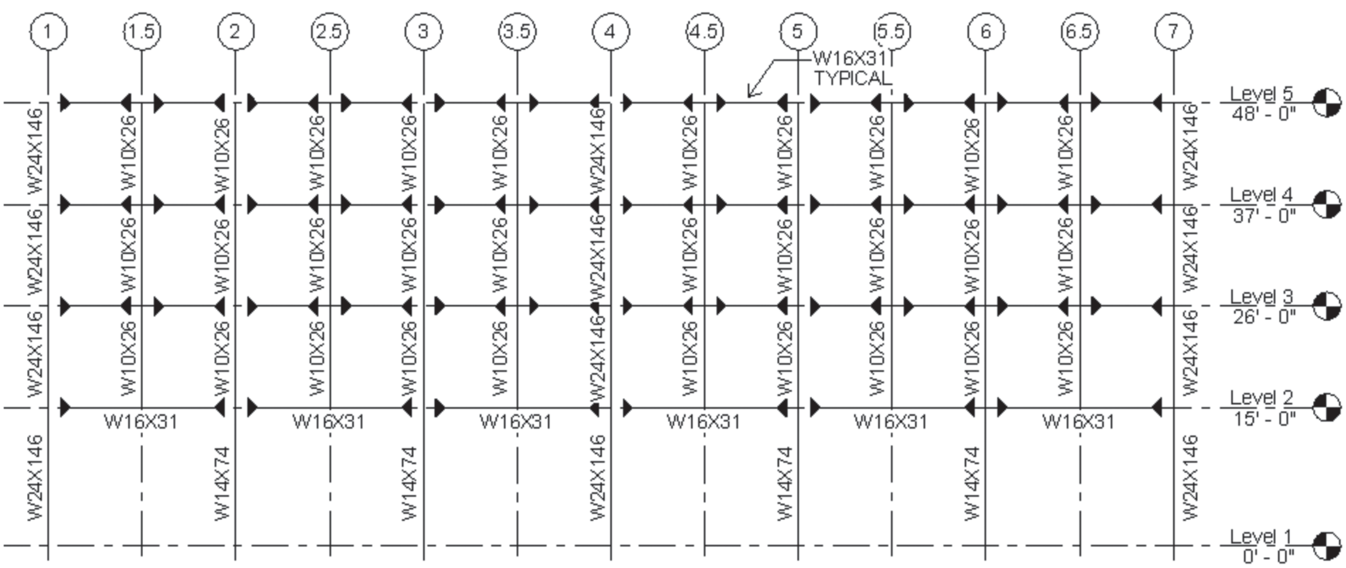



*Fig 4. Elevation view (gridline A) of building A.*

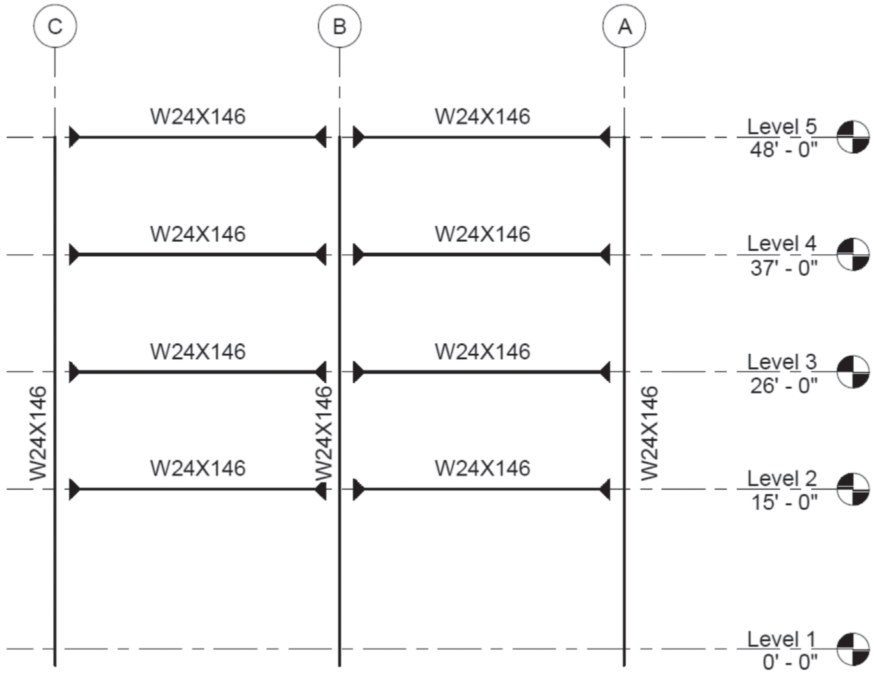



*Fig. 5. Elevation view (gridline 1) of building A.*

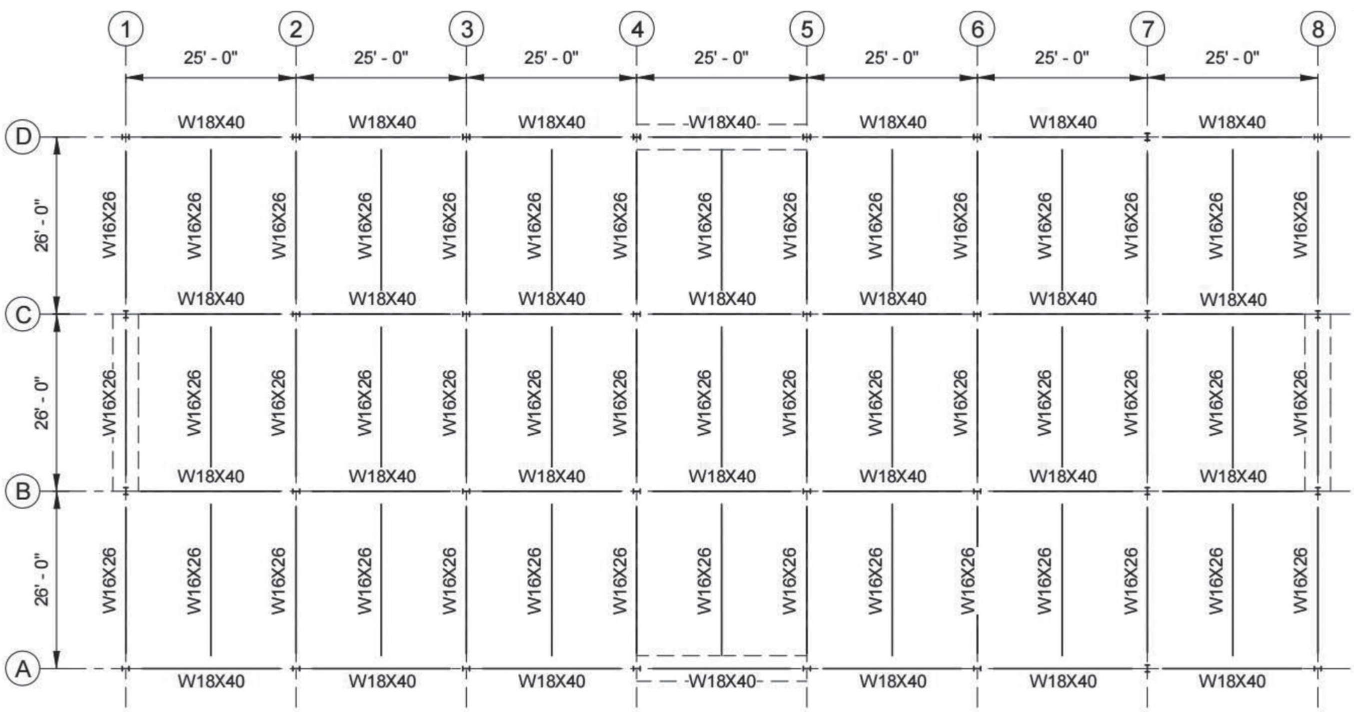



*Fig. 6. Plan view (level 2) of building B (dotted lines indicate braced frames).*

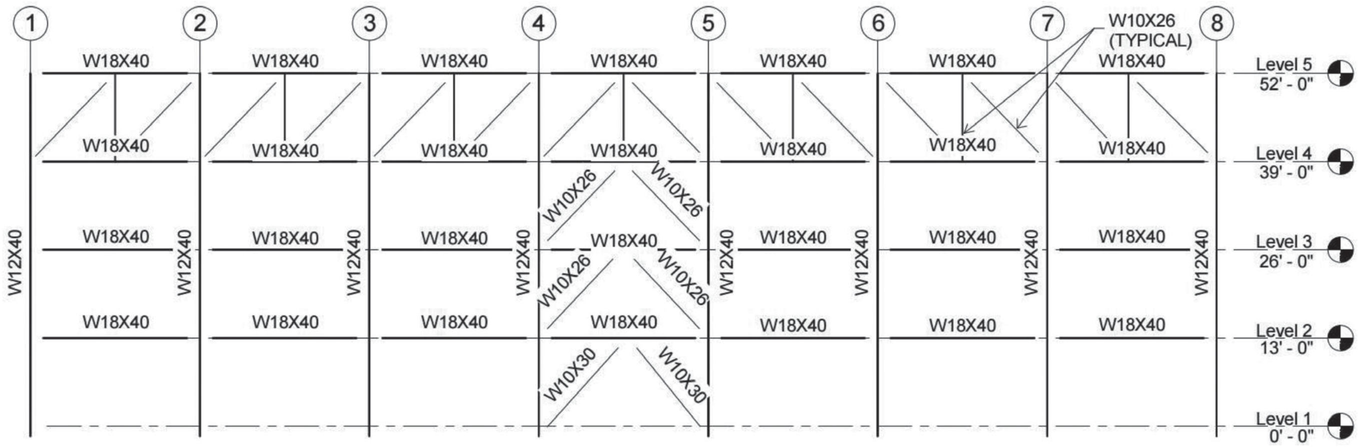



*Fig. 7. Elevation view (gridline A) of building B.*

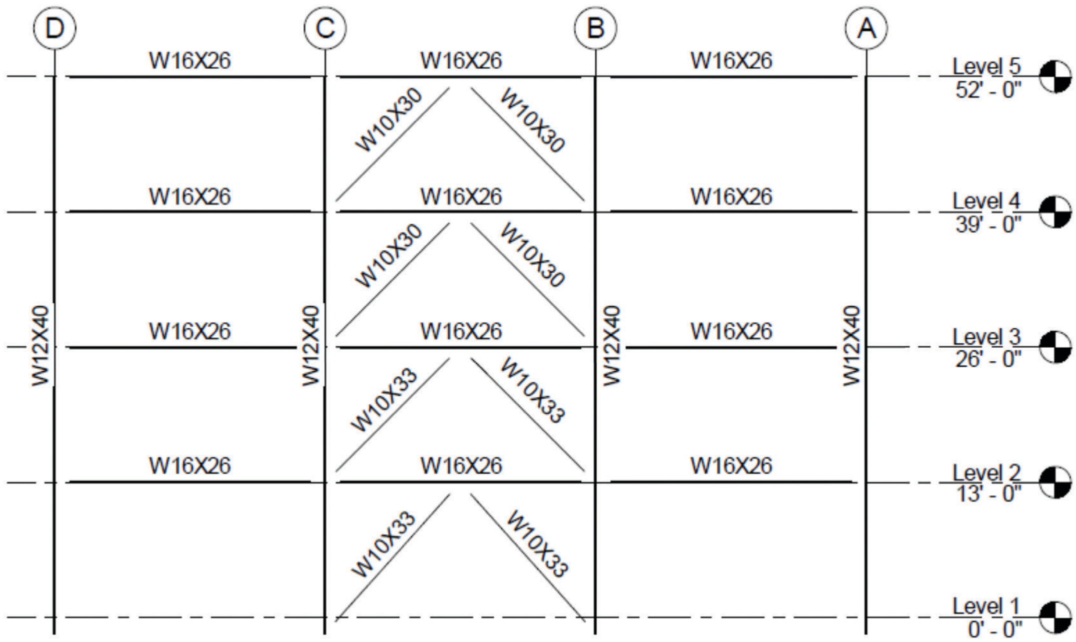



*Fig. 8. Elevation view (gridline 1) of building B.*

building design. Both buildings were designed as Seismic Design Category C structures. Building A was designed with ordinary steel moment frames, while building B incorporated an ordinary concentrically braced frame system. More details about each structure, as well as the existing buildings used for inspiration, can be found in the AISC report (Cortes and Liu, 2015).

Both buildings employ lateral-force-resisting systems along the perimeter of the buildings; hence, only exterior column loss scenarios were evaluated. For the purposes of this study, it was assumed that the threat to column loss exists at exterior columns only due to controlled public access and no underground parking. In the future, the proposed methodology can be applied to interior column loss scenarios as well.

## BUILDING CONFIGURATIONS

Alternative building configurations for both case study buildings were developed in order to evaluate the effectiveness of different framing strategies on the robustness of a steel structure. The configurations for each building have the same overall geometry and differ only in the locations of LFRSs and the presence and locations of the stiff stories. Each configuration is code compliant for gravity and lateral loads but was not designed for the column-loss scenario.

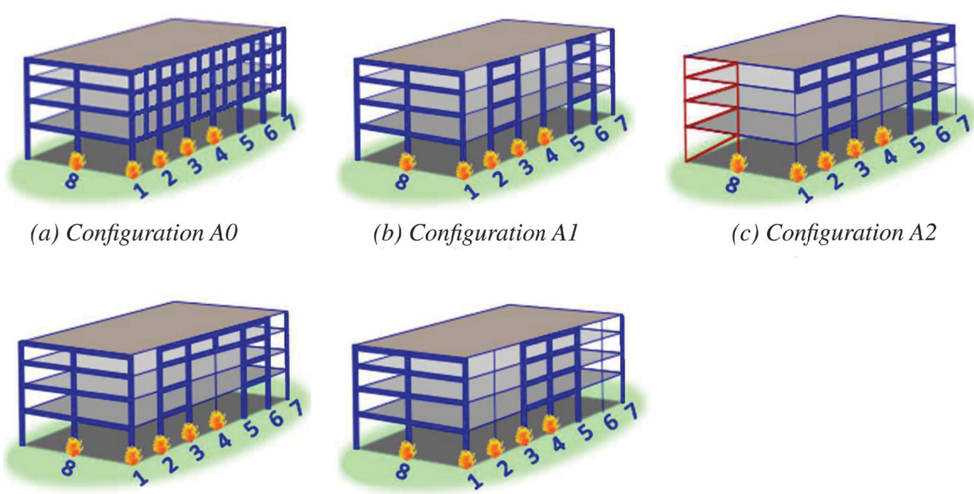
While many different potential configurations could have been developed, the configurations used were chosen because they vary both LFRS locations and stiff-story locations, as well as the level of robustness. Additional framing configurations can be found in Cortes and Liu (2015) and Cortes et al. (2015).

### Building A Configurations

Four additional configurations for building A were analyzed to explore the influence of stiff stories and integration of the LFRS on system robustness (Figure 9). In the development of the alternative configurations, some were developed using the stiff-story concept, while others were intentionally designed to rely on the LFRS for system robustness in the event of column loss. The intention was to consider the relative cost effectiveness of the different framing configurations. In Figure 9, thick lines designate moment frame components, and fire symbols represent locations where column removal was evaluated. Using the alternate path procedure, which will be explained in the next section, one column was removed at a time in order to analyze each of the configurations. Configuration A0 is the initial configuration with full Vierendeel trusses in the long direction of the building and moment frames in the short direction (column lines 1, 4 and 7 in Figure 9). A1 does not have a stiff-story element; however, each external column is part of a moment frame, with column 4 contributing to a moment frame in the transverse direction. For building A2, a braced frame was employed in the transverse direction (at gridlines 1 and 7) to provide another alternative should the engineer wish to avoid having corner columns at the upper story with moment connections in both orthogonal directions. Configurations A2 and A3 include stiff stories of varying length. Configuration A4 employs traditional moment frames, with some external gravity frames but no stiff story.

### Building B Configurations

Six additional configurations were analyzed for building B, as shown in Figure 10. B0 is the original base configuration




(a) *Configuration A0* (b) *Configuration A1* (c) *Configuration A2*

(d) *Configuration A3* (e) *Configuration A4*

*Fig. 9. Building A configurations (thick lines represent moment frame components).*

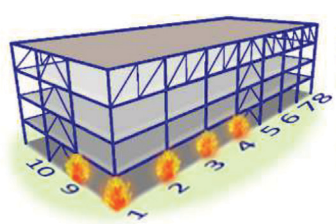
inspired by the Lamar Headquarters. As mentioned previously, the other configurations are variations of the base building, with changes in braced frame locations and variations of framing with and without a stiff story. Again, in order to study the relative cost effectiveness of different framing strategies, some configurations were designed to rely on the LFRS as much as possible for system robustness in the event of column loss. For B1 and B4, the stiff story has been removed, creating a more traditional building. Many of the columns in these configurations are not adjacent to braced frames or supported by a stiff story, making B1 and B4 more vulnerable than other configurations. Configurations B2 and B5 are similar to B1 and B4, respectively, with a stiff story added in the longitudinal direction of the building. B3 and B6 examine the effect of extending a stiff story around the entire perimeter of the building. Chevron braces were selected as the stiff-story elements in lieu of diagonal braces in order to extrapolate the framing of the braced frames in the LFRS, and B1 replaced B0 as the base building for comparisons of robustness.

## ALTERNATE PATH ANALYSIS

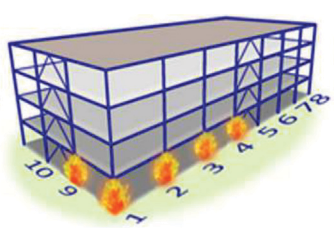
The Alternate Path (AP) Linear Static Procedure (LSP) provided in UFC 4-023-03, *Design of Buildings to Resist Progressive Collapse* (DoD, 2009), was implemented in order to study each of the configurations presented in the previous sections. ASCE 41-06, *Seismic Rehabilitation of Existing Buildings* (ASCE, 2007), was also used extensively because the UFC guidelines reference ASCE 41-06 often. UFC requires that at least one column near the middle of the structure on the long side, one column near the middle of the structure on the short side, and one corner column be removed. In this study, however, every unique exterior column removal scenario was conducted for each configuration. Because the buildings are symmetrical, only one of each type of uniquely supported column was selected for removal (identified by the fire symbols in Figures 9 and 10). Columns not supported by moment connections or braces were assumed to be incapable of meeting acceptance criteria in column loss scenarios because the beam-column connections are modeled as pinned connections and do not have rotational resistance. Column removal scenarios at each of these locations are classified as a collapse mechanism.

The structural analysis software, SAP 2000 (CSI, 2014), was used to analyze each column removal scenario through a staged construction process. For each building configuration, two models were created: one for deformation-controlled actions and the other for force-controlled actions. The deformation-controlled model is used when verifying ductile limit states (e.g., column tension, beam flexure), while the force-controlled model is used for verifying brittle limit states (e.g., column compression, beam shear). All members were analyzed according to the AISC *Specification* (AISC, 2010) and using the load combination shown in Equation 1. The load factor $\Omega_L$ is 1.0 for areas away from the removed column in both force-controlled and deformation-controlled actions. For areas immediately adjacent to the removed elements, $\Omega_L$ is a function of the *m*-factors for deformation-controlled actions and 2.0 for force-controlled actions. In Equation 1, *D*, *L* and *S* are the dead, live and snow loads, respectively.

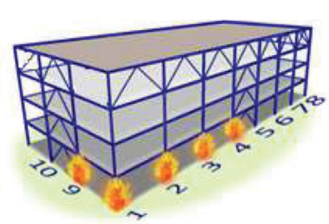
$$G_L = \Omega_L\left[1.2D + (0.5L \text{ or } 0.2S)\right] \tag{1}$$



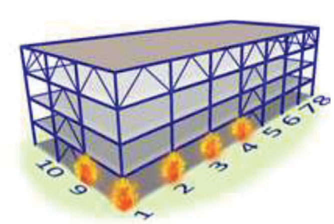

*(a) Configuration B0*



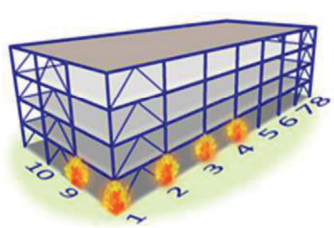

*(b) Configuration B1*



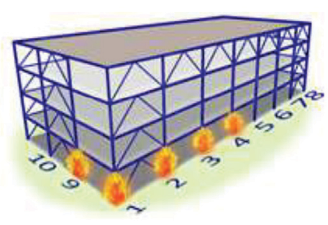

*(c) Configuration B2*



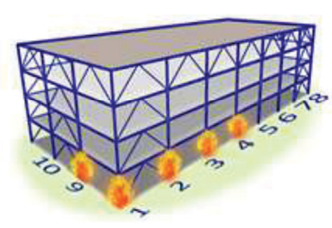

*(d) Configuration B3*




*(e) Configuration B4*



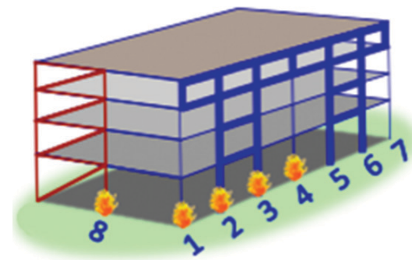
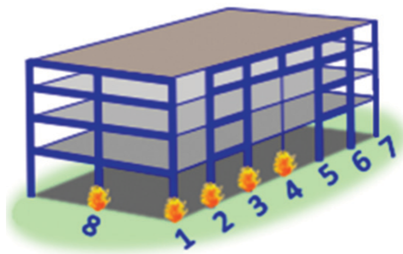

*(f) Configuration B5*




*(g) Configuration B6*

*Fig. 10. Building B configurations.*

**Table 1. Building A Configuration Results Summary**

| | |
|---|---|
|  A1 CM = 0 RE = 0 |  A2 CM = 0 RE = 10 |
|  A3 CM = 0 RE = 0 |  A4 CM = 4 RE = 0 |

CM = collapse mechanism
RE = elements requiring redesign

Here, *m*-factors are numerical values that are part of the acceptance criteria for structural members and elements. These were derived for seismic loads and are included in ASCE 41 for linear procedures. ASCE 41 and the UFC both make use of *m*-factors in determining load increases and for the final evaluation of member suitability.

For each scenario, the first-story column (and adjoining brace, if applicable) was removed and loads were applied. Results from the column removals of each model were then analyzed to determine which specific frame members are inadequate according to UFC and ASCE 41 criteria. Figure 11 shows results of configuration A0 analyses. The "x" indicates which columns were removed. The circled members are those that did not meet criteria and required redesign. In this scenario, all of the inadequate elements are Vierendeel truss columns adjacent to a missing column, which are carrying a portion of the load originally carried by the missing column. These results are logical because the building was designed without consideration of column removal, and thus, the adjacent columns were not designed for the redistributed loads.

**Building A Results**

The preceding process was followed to analyze every configuration. The results for building A configurations are summarized in Table 1, which shows the configuration name, number of collapse mechanisms (CMs) resulting from column removals, and the number of elements requiring redesign (RE). The column removal analysis results are based on 16 exterior columns.

Although a stiff story was not directly employed in A1, strategic location of the moment frames allowed the elimination of collapse mechanisms under the elastic analysis. This strategy was sufficient to eliminate column loss risk. For configuration A2, removal of column 8 (and the adjoining brace) caused building torsion, which resulted in five members requiring redesign; therefore, removal of the same column on the opposite face of the building was assumed to also require five members to be redesigned. This building torsion is due to a rigid-body rotation of the braced frame caused by the vertical displacement at the missing column (and brace) location. No redesign was required when a

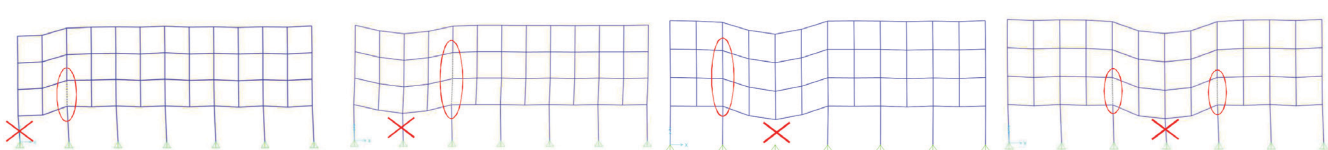
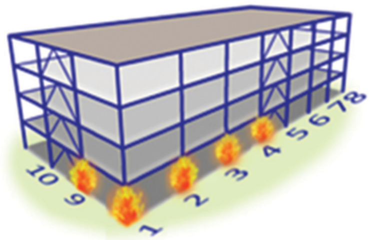
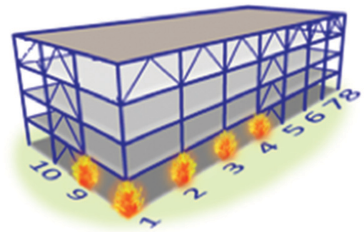


*Fig. 11. Column removal deflected shapes for configuration A0. Unsatisfactory members identified with oval.*

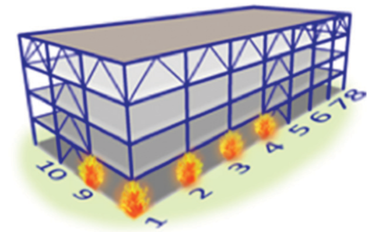
**Table 2. Building B Configuration Results Summary**



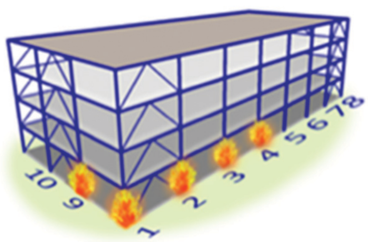
B1 CM = 12 RE = 68



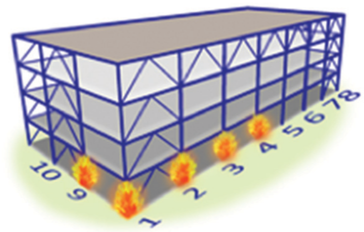
B2 CM = 0 RE = 36



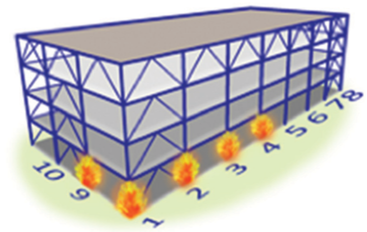
B3 CM = 0 RE = 4



B4 CM = 8 RE = 48



B5 CM = 0 RE = 28



B6 CM = 0 RE = 8

CM = collapse mechanism
RE = elements requiring redesign

column under the stiff story (columns 1–7) was removed. Through incorporation of a stiff story and strategic location of LFRSs, all members in configuration A3 passed acceptance criteria. For configuration A4, collapse mechanisms were assumed at columns 2 and 6. The vulnerability on these columns could be mitigated by adding a stiff story in the spans between columns 2–3 and 5–6.

The analysis of this building showed that stiff stories can effectively redistribute loads in column loss scenarios. This was evidenced in configurations A2 and A3. It is believed that because the moment frames were designed to limit interstory drifts, the members had excess load-carrying capacity, allowing the LRFS and stiff-story components to carry the distributed loads without requiring redesign.

### Building B Results

Building B configurations were also analyzed for column loss and the results are summarized in Table 2. The number of collapse mechanisms and members requiring redesign is based on the total number of exterior columns in the building (20).

Configuration B1 was used as the baseline and does not have a stiff story. It has 12 columns that, if removed, would result in a CM, making it very vulnerable and the least robust of all configurations. The braced frame configurations in the short direction of the building lacked redundancy, causing many members to require redesign when one of the braced frame columns was removed. For case B1, 60 of the 68 elements requiring redesign were due to removal of columns 9 and 10, as well as the other two identical columns due to symmetry. This is due to the previously mentioned building torsion that results from removal of the brace causing rigid-body rotation. Configuration B2 is similar to configuration B1, with a stiff story added in the longitudinal direction of the building. This stiff story eliminated the collapse mechanisms and reduced the number of members requiring redesign from 68 to 36. Once again, most of the members requiring redesign were due to the removal of column 9 (and identical columns) causing building torsion. Configuration B3 incorporates stiff-story elements in the short direction of the building, completing the stiff story around the full perimeter of the building. This minimized building torsion under column removal 9 (and identical columns) and resulted in only four members requiring redesign. B4 does not incorporate a stiff story and has eight vulnerable columns, resulting in CMs under static analysis, and 48 members requiring redesign. Forty-four of these member redesigns are due to building torsion effects caused by the loss of column 9 (and identical columns). Configuration B5 has no CMs; however, as for previous configurations, removing column 9 (and identical columns) caused a building torsion that was responsible for the 28 members requiring redesign. It should be noted that in this configuration, no elements required redesign when a column under the stiff story (columns 1–8) was removed. Configuration B6 is the same as B5, but with a stiff story in the transverse direction as well. This reduced the number of elements requiring redesign. This reduction is mainly due to the additional stiffness provided by the stiff story in the short direction, which helped reduce the building torsion.

The analysis results from the configurations that featured stiff stories show that the stiff-story concept can, when adequately designed, redistribute loads throughout the structure. Removal of a column supported by a stiff story often did not require member redesign for that specific column removal scenario. Some column removals resulted in members that did not meet acceptance criteria but could easily be remedied with a slightly larger section size. The most substantial improvements in performance (i.e., reduction in members requiring redesign) were achieved by continuing the stiff story around the perimeter of the building, as shown by configurations B3 and B6. While RE does not necessarily result in disproportionate collapse, it is an indication of the level of effort required beyond conventional analysis and design for gravity and lateral loads to achieve system robustness.

## SUPPORT AND BRACING FACTORS

In order to quantify the vulnerability, robustness and efficiency of each of these configurations, a support factor, $S_F$, and bracing factor, $B_F$, were developed. These two factors reflect the scope of the parametric study, which was based on relatively simple building configurations designed for typical gravity and lateral loads but not for column loss. The support factor provides some indication of the relative robustness of the system, while the bracing factor provides an indication of the relative effectiveness of a particular framing configuration in resisting collapse. These factors can be used by designers to evaluate the most efficient configuration for their own building designs.

The support factor is calculated as the percentage of exterior columns that are supported by a braced frame, moment frame, or stiff story. For these case study buildings, each column from ground to roof would count as one column. For a taller building, which might have stiff stories at intervals over its height, this accounting of columns may need to be revised. The stiff stories are effective at preventing CMs of columns they support. For building B1, the support factor, $S_F$, would then be based on eight columns supported out of 20 exterior columns, resulting in 40%. For configuration B4, the factor would be based on 12 columns supported (or 60%). As shown in Figure 12, there is a direct correlation of support factor, $S_F$, to number of collapse mechanisms (CM). CM and $S_F$ are essentially complements of one another. For example, 60% of columns supported translates into 40% of columns collapsing, while 100% of columns supported results in no collapse mechanisms.

Relative efficiency, or effectiveness, of different framing solutions would vary by designer. However, if following the same design approach used in this study (i.e., designing initially for gravity and lateral loads without consideration for column loss), and if defining higher effectiveness by lower

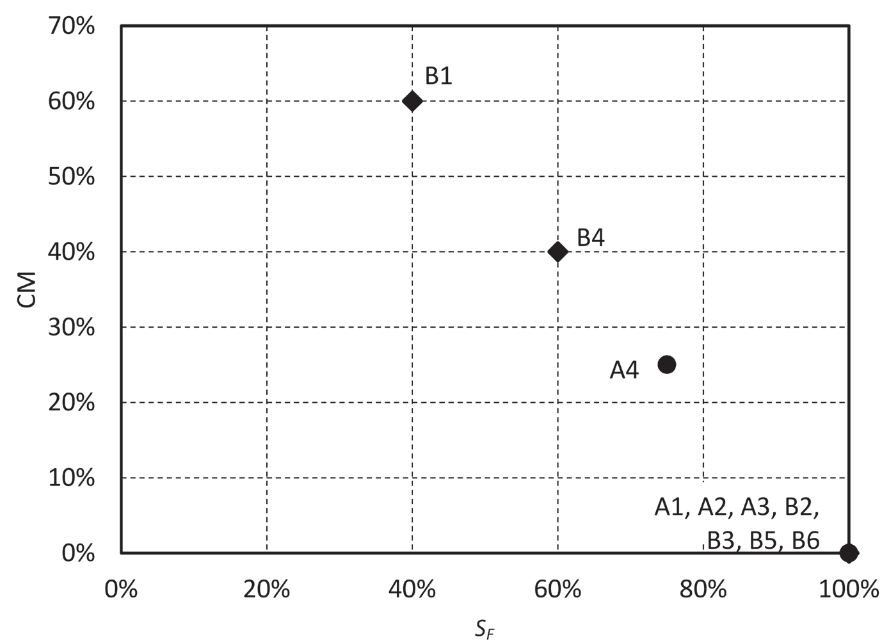



*Fig. 12. Support factor,* $S_F$*, versus collapse mechanisms, CM, for the building configurations.*

numbers of members requiring redesign, then the bracing factor, $B_F$, provides some indication of relative effectiveness of a given configuration. $B_F$ represents the percentage of exterior bays within the story that are laterally braced or moment connected. A braced bay was defined as a one-story, one-bay frame with diagonal braces or moment connections. The bracing factor, $B_F$, shown in Equation 2 as a percentage, was established as a simple way of representing the relative efficiency of the combined stiff story and lateral bracing system; $n_{BB}$ and $n_B$ are the number of braced bays and the total number of exterior bays on the structure (80 for building B).

$$B_F = \frac{n_{BB}}{n_B}(100\%) \tag{2}$$

The relative effectiveness of different configurations can be seen in a plot of the bracing factor, $B_F$, versus the number of elements requiring redesign, RE (Figure 13). A higher $B_F$ generally means a lower number of members requiring redesign. For most of the configurations studied, a higher bracing factor is preferred.

For buildings with braced frames, some effectiveness can be gained by running the stiff story around the entire perimeter of the building, as in configurations B3 and B6. This benefit can be seen in Figure 13, where the two configurations for building B with the lowest RE values are B3 and B6. Of these two configurations, B3 is the best option because it required fewer braced bays (lower $B_F$) to achieve a lower RE.

For building A, $B_F$ was at least 50% for all configurations. This large percentage, compared with building B, is partially due to the extra moment frames required to satisfy interstory drift limits.

The evaluation of the alternative configurations for the case study buildings demonstrates that essentially any stiff story solution that ties in with the LFRS and supports all columns will be more robust than traditional framing. This can be seen in a comparison of the support factor, $S_F$, and the number of collapse mechanisms, CMs.

## IMPLEMENTATION AND CONSIDERATIONS OF FRAMING STRATEGIES

The support factor and bracing factor can be used to evaluate the effectiveness of framing strategies. Higher percentages for these factors imply increased robustness. However, it is not the intent of this paper that designers should design excessively robust structures and always incorporate stiff-story framing that may compromise other design aspects. Instead, structural engineers must weigh options to determine the most effective design for the owner's needs.

Other considerations that must be taken into account include cost and aesthetics. Designers must consider not only material costs, but also fabrication and installation costs. For instance, a lightweight building may have a low material cost but many members, which increases both fabrication and installation costs. Also, when comparing moment frame systems to braced frame systems, these cost differences can

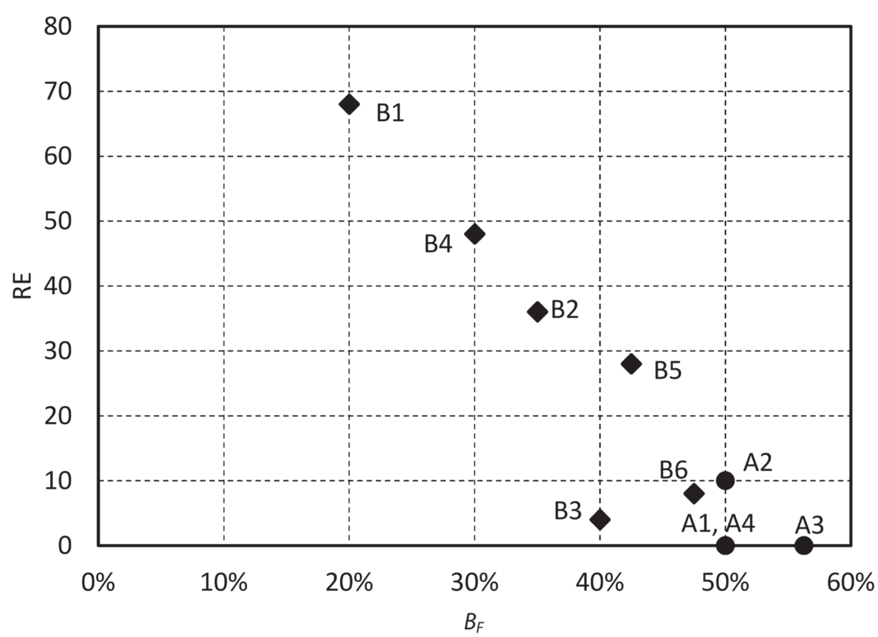



*Fig. 13. Bracing factor,* $B_F$*, versus elements requiring redesign, RE.*

| Table 3. Sample Relative Cost Considerations | | | |
|---|---|---|---|
| Cost Category | Braced Frames | Moment Frames | Gravity Frames |
| Material cost (per lb) | Wide flanges: $0.49<br>Tube shapes: $0.56 | | |
| Fabrication cost (per connection) | $200 | $200 | $200 |
| Installation (per connection) | $200 | $800 | |

be significant. Braced frame systems are typically lighter weight than moment frame systems, which require large sections to resist drift demands; however, braced frames do not typically have the redundancy of moment frame systems. As shown in the configurations that were presented, moment frame systems are less likely than braced frame systems to be susceptible to collapse mechanisms due to redundancy.

Table 3 displays some approximate cost estimates for a typical low-rise steel building in a low seismic region, provided by a steel fabricator in the Midwest. Transportation costs and detailing are not included in these cost estimates. The fabrication and installation costs provided in the table do not reflect a particular member or connection size. The braced frame connection cost assumes a shop-welded gusset plate that is field welded to the bracing member using plates. The moment frame connection cost conforms to a WUF-B configuration (welded unreinforced flange and a bolted web). Finally, the gravity connection assumed was a typical shear tab connection with a shop-welded single plate that is field-bolted to the beam web. While the authors acknowledge that the size of the framing members and specific details of the connection will alter the individual cost per connection, the costs reflected in the table are ballpark numbers based on industry experience to provide a quick comparison of different framing layouts as a means to weigh options.

As the table shows, moment connections can be 2.5 and 5 times more expensive than braced connections and gravity shear tab connections, respectively. To determine the percentage increase in building cost, a $15/ft$^2$ estimate can be used to conservatively estimate material, fabrication and installation costs for a traditionally framed building.

The costs provided in Table 3 were utilized to compare relative connection costs of each building configuration. For each configuration, the total perimeter connection costs were divided by the cost for shear connections at the perimeter framing. This provided a normalized connection cost that is reported on the Y-axis of Figure 14. When compared

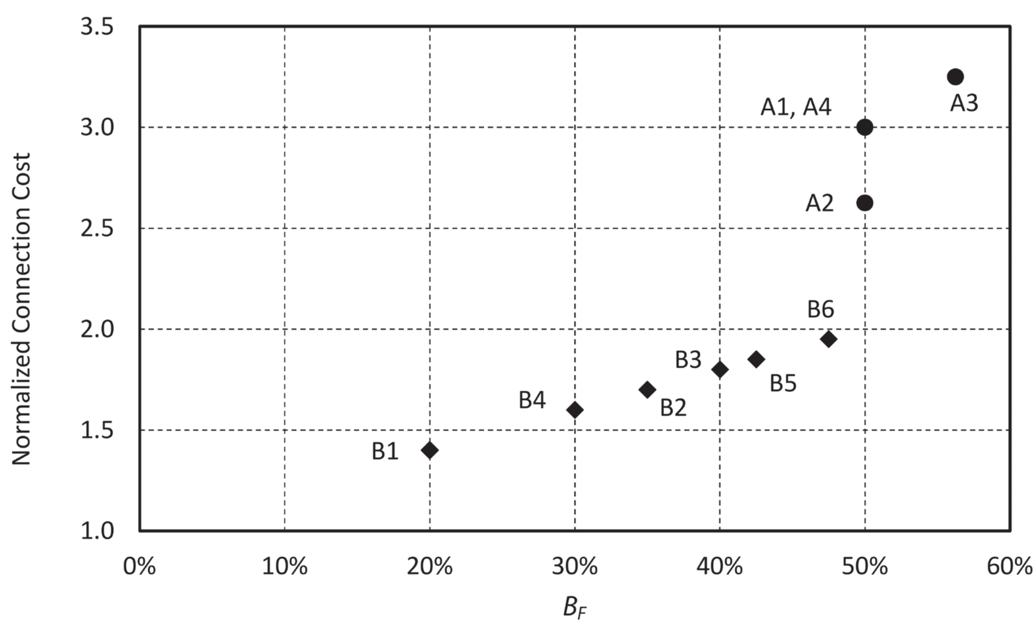



*Fig. 14. Normalized connection cost versus bracing factor,* $B_F$.

with the bracing factor, it is evident that an increased bracing factor typically constitutes an increase in connection cost. Additionally, it is clear that while moment frames are effective for distributing loads and providing a high bracing factor, they are significantly more expensive than braced frames.

When comparing the normalized connection cost for each configuration with the members requiring redesign, there is an inverse relationship, as shown in Figure 15. From this graph, it becomes evident that continuing the stiff story around the perimeter can result in a cost-effective increase in robustness. This is clearly observed in configurations B2–B3 and B5–B6, which, for a relatively small increase in connection cost, had a substantial reduction in RE. For more detailed cost comparisons between configurations, material costs could be considered as well.

Building aesthetics are also an important parameter to consider. Braces may negatively affect building views, compromise the exterior aesthetic envisioned by the architect, or inhibit the architectural programming and layout of the space. Similarly, moment frames, which require large members to control drift, could encroach on desired ceiling heights and views. To implement stiff stories without compromising the architectural aesthetic, engineers are encouraged to work with architects to locate stiff stories at levels would that minimize negative architectural impacts. For instance, hat-truss systems are sometimes located at mechanical levels, where views and aesthetics are less important. Skidmore, Owings & Merrill LLP (SOM) used this approach to design a 25-story office tower, which was constructed in Salt Lake City, Utah (SOM, 2016). By strategically designing framing to distribute stiffness and redundancy throughout the structure while balancing cost and aesthetics, the resulting structure can be robust, efficient, and aesthetically pleasing.

## CONCLUSIONS

This paper described a study of framing strategies, many of which implemented stiff stories, and an approach for evaluating various configurations for system robustness in the event of a column loss. The configurations were studied using alternate path linear static analysis to evaluate unique column removal scenarios at the ground level.

Results from the analyses showed that any stiff-story framing strategy that is integrated with the LFRS and supports all columns will be more robust than traditional framing. Stiff stories were shown to effectively redistribute loads in column loss scenarios. In particular, when using a braced frame system, continuing the stiff story around the entire perimeter of the building story can substantially increase the number of members that meet acceptance criteria. The moment frame structures studied were more redundant than the braced frame systems, providing more opportunity for load redistribution; however, moment frame structures are also typically more expensive.

Vulnerability indices were developed to evaluate and measure relative robustness among framing strategies. The support factor, $S_F$, provides some indication of the relative robustness of the system, while the bracing factor, $B_F$,

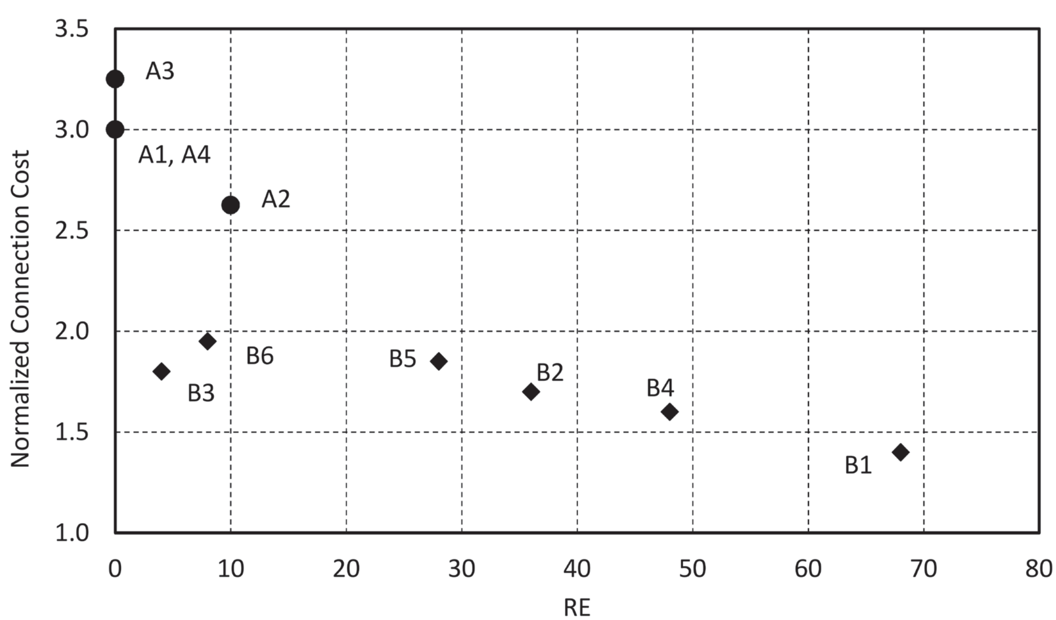



*Fig. 15. Normalized connection cost versus members requiring redesign, RE.*

provides an indication of the relative effectiveness of a particular framing configuration in resisting collapse. These factors can be used as a first screening when determining the capacity of a particular building in resisting column loss, while also considering cost and aesthetics. A higher $B_F$ generally means a lower number of members requiring redesign (RE); thus, a higher bracing factor is typically preferred. However, an increased bracing factor and lower RE also generally means an increased building cost.

The implications of each framing strategy vary based on building size, location, use, etc. Structural engineers must work with the client and architect to determine the proper trade-off of desired building performance versus cost and aesthetics. From the analyses conducted in this study, the most successful configurations integrated LFRSs with a stiff story around the perimeter to minimize connection costs while still achieving significantly improved robustness.

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