Construction Cost Premiums for Risk Category IV Special Moment Frame Buildings

Paul W. Richards and Amy J. McCall

ABSTRACT

The International Building Code uses risk categories to reduce the probability of damage and collapse for certain buildings. One proposal for improving post-earthquake functional recovery is to design more buildings as Risk Category IV. The purpose of this study was to investigate the construction cost premiums for Risk Category IV buildings with steel special moment frames (SMF). Mathematical derivations were used to bound the stiffness and strength amplifications required for Risk Category IV design, accounting for period effects (as buildings are strengthened/stiffened, design loads increase). To complement this mathematical approach, 12 case study SMF buildings were designed with heights ranging from 2 to 16 stories. The primary conclusion of the study is that construction cost premiums for drift-governed SMF buildings are an order-of-magnitude greater than for strength-governed buildings. For many strength-governed buildings, the cost premium for Risk Category IV design is around 1% of the total building cost. For drift-governed SMF buildings, the cost premiums for Risk Category IV design are 6 to 16% of the total building cost, with the greatest premiums for buildings around eight stories. These cost premiums should be considered when evaluating Risk Category IV design as a strategy for improving post-earthquake functional recovery.

Keywords: functional recovery, special moment frames, construction cost, risk category, drift limit.

INTRODUCTION

The International Building Code (IBC) (ICC, 2021) uses risk categories to reduce the probability of damage and collapse of certain buildings under earthquake loading. Buildings with Risk Category IV are designed with a 1.5 multiplier in the base shear equation and about half the allowable drift as compared to Risk Category II (ASCE, 2016). The actual increases in strength and stiffness for structures designed as Risk Category IV may be greater than 1.5 and 2.0, respectively, because as a building is strengthened/stiffened its natural period decreases and the design loads may increase.

Broader use of Risk Category IV design is being discussed as an interim measure to improve the post-earthquake functional recovery of buildings. This discussion is important for steel special moment frames (SMF) because functional recovery is expected to be poor for code-minimum SMF that form plastic hinges in the beams (Erochko et al., 2011; Harris and Speicher, 2018; Richards et al., 2023). Methods for improving functional recovery for SMF include designing to a lower drift limit, providing better post-yield stiffness, and/or using replaceable fuses. Recommendations from NIST for improving post-earthquake functional recovery (NIST, 2021) suggest that, as an interim measure, requiring Risk Category IV for a broader class of buildings could substantially increase the number of buildings that are able to recover quickly.

To evaluate this approach to functional recovery (more Risk Category IV design) against other options, it is important to know the construction cost premium for Risk Category IV buildings, relative to Risk Category II buildings. NIST (2021) notes that in the code development process, certain industry groups oppose proposals that include even modest increases in initial construction costs.

Some studies have explored the cost of improved seismic design, but most results have limited application to SMF buildings. NIST (2013) investigated the cost of six buildings designed for the Memphis metropolitan area. The buildings were a three-story apartment (wood frame), a four-story office (steel braced frame), a one-story retail (tilt-up), a one-story warehouse (tilt-up), a six-story hospital (steel braced frame), and a two-story elementary school (masonry walls). The comparisons were between no seismic design, seismic design per local code [2003 IBC (ICC, 2003)], and seismic design per national seismic code [2012 IBC (ICC 2012), ASCE/SEI 7-10 (ASCE, 2010)]. The conclusion was that the cost premium for designing per 2012 IBC, as compared to 2003 IBC, was 1% or less. None of the buildings considered in the study were drift-governed. For the braced-frame hospital, the importance factor was 1.5 for both the 2003 IBC and 2012 IBC designs, so the impact of importance factor on design was not investigated. The 2012

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IBC braced-frame hospital design had a 2.5% cost premium over a wind-only design.

Yu et al. (2015) investigated the cost of designing two school buildings in Oregon as Risk Category IV rather than Risk Category III so the buildings could serve as emergency shelters. Yu et al. estimated that the increase in structural construction costs would be less than 1%. The lateral force resisting system in both buildings was presumably strength-governed masonry walls.

Richards et al. (2022) mentioned the construction cost of three steel SMF buildings (4-, 6-, and 8-story) designed as both Risk Category II and Risk Category IV. The Risk Category IV buildings were designed with deeper columns and lower clear-heights between stories to help mitigate the cost impacts. Still, the cost premiums for the Risk Category IV designs were estimated as 4 to 14% of the total building costs, much higher than the premiums reported in the other studies.

The limited studies that have been cited suggest that drift-governed SMF buildings may have a substantially higher cost premium for Risk Category IV design than strength-governed buildings. As broader use of Risk Category IV design is being discussed, it would be helpful to have a more accurate sense of what the construction cost premiums are for Risk Category IV SMF buildings and a better theoretical basis for understanding the cost premiums.

The purpose of the present study was to quantify the impact of Risk Category IV design on the stiffness, strength, and weight of SMF buildings and determine the construction cost premiums for Risk Category IV SMF buildings.

METHODS

Two complementary methods were used for the study. The first method was to mathematically derive the difference in strength and stiffness for Risk Category II and Risk Category IV designs, accounting for period effects. Closed-form solutions were possible for single-degree-of-freedom (SDOF) systems. The derived stiffness and strength ratios (IV/II) were used with a cost index to bound the range of expected cost premiums.

The second method was to use case study buildings to compute cost premiums. Twelve SMF buildings with varying height (2-, 4-, 6-, 8-, 12-, and 16-story) and varying Risk Category (II and IV) were designed. The steel weights from the case study buildings were used with a cost index to estimate the Risk Category IV construction cost premium in terms of the total building cost.

Cost Index

A total building cost index was used in both approaches to estimate the cost premium for Risk Category IV designs. The cost of structural steel frames is influenced by labor more than material, but for estimating purposes, both labor and material costs were assumed to scale in proportion to the total steel weight. This approach indirectly accounts for added labor costs to make the heavier connections in the Risk Category IV designs. The index was based on the following assumptions:

- The total steel cost (gravity + lateral) was 10% of the total building cost for Risk Category II buildings.
- The total foundation cost (gravity + lateral) was another 10% of the total building cost for Risk Category II buildings.
- The total steel cost was proportional to the total steel weight.
- The foundation costs would rise in proportion to the increased steel weight for Risk Category IV buildings (only the foundation costs associated with the increased lateral frames would increase).

Restated, the cost index assumed that 80% of the total building cost was unrelated to the structure, and scaled the other 20%, associated with the steel and foundations, based on the ratio of the steel weights.

$$\text{Cost Index} = 0.8 + 0.2\left(\frac{W_{t,IV}}{W_{t,II}}\right) \tag{1}$$

where $W_{t,IV}$ is the total steel weight of a particular Risk Category IV building, and $W_{t,II}$ is the total steel weight of a comparison Risk Category II building.

Case Study Building Geometry, Loads, and Site Parameters

Various building plans and SMF layouts were used for the 12 case study buildings. The 2-, 4-, 6-, and 8-story buildings were 180 ft × 120 ft in plan and had SMF as indicated in Figure 1(a, b). The 12- and 16-story buildings were 100 ft × 100 ft in plan and had SMF as indicated in Figure 1(c). The floors extended 12 in. past the frame lines. The required clear height for the stories was 12 ft. Because most of the buildings had 3-ft-deep SMF beams, most of the story heights were 15 ft to achieve the clear height in the lower levels. However, for some of the buildings, shallower beams could be used, and the required story height to achieve the clear height was reduced (Table 1).

The loading used for design was:

- Floors: 45 psf (6.25 in. lightweight concrete on 3 in. metal deck)
- Steel framing: as designed (this turned out to be 5–15 psf, see Table 6)
- MEP: 7 psf

- Ceiling/lights/flooring: 3 psf
- Partitions: 20 psf (10 psf included for seismic weight)
- Floor live load: 50 psf (reducible)
- Roof live load: 20 psf (reducible)
- Exterior walls: 25 psf

The roof dead loads were assumed to be the same as the floors, roughly accounting for permanent equipment.

A Los Angeles site with $S_{DS} = 1.4$ and $S_{D1} = 0.75$ was used for the study.

Case Study Building Designs

The 12 case study buildings were designed using ASCE/SEI 7-16, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (ASCE, 2016), hereafter referred to as ASCE/SEI 7; the AISC *Seismic Provisions for Structural Steel Buildings* (AISC, 2016b), hereafter referred to as the AISC *Seismic Provisions*; and the AISC *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications* (AISC, 2016a). The drift limit was 2.5% or 2.0% for the Risk Category II buildings, with the higher limit for the two- and four-story designs. For the Risk Category IV buildings, the drift limit was 1.5% or 1.0%. Linear modal response spectrum analysis was used for the designs, typical of U.S. practice in high seismic areas.

A welded beam-to-column connection with no reduced or reinforced beam sections was used for the SMF buildings so that no modifications to the element properties were necessary to represent the elastic stiffness of the beams or connections in the analysis model. The maximum beam depth permitted in the designs was W36, but shallower beams were used when they were the lightest options. All the designs used W27 columns for consistency.

RAM Structural System (Bentley, 2021) was used to perform all design checks, although other tools were used for initial member sizing. The SMF beam and column sizes were optimized for drift outside of RAM using a genetic algorithm (McCall and Richards, 2022). Within RAM, all final drift and strength checks were performed using linear modal response spectrum analysis. For strength checks, the period was limited to C_uT_a, but for drift checks, the actual building period from the model was used, per ASCE/SEI 7. The AISC *Seismic Provisions* seismic checks (e.g., strong-column weak-beam, doubler plates) were performed within RAM. The flexibility of the panel zones was represented in the RAM models by limiting the rigid end offsets to 25% of the theoretical length. Preliminary work had found that centerline modeling was too flexible when W27 columns were used, but rigid offsets over the entire theoretical length were too stiff.

Table 1 summarizes the seismic response coefficients, C_s, used for the strength and drift checks for each building. C_s was the value used for strength checks, which,

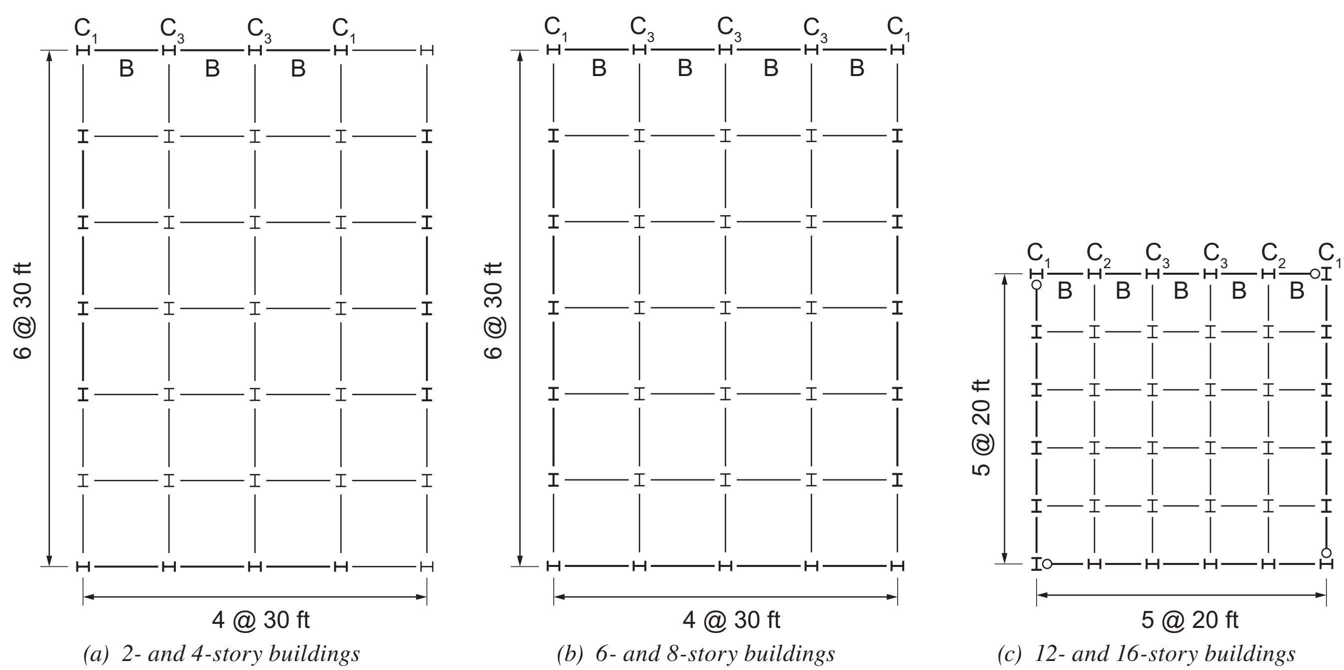


Fig. 1. Plan views of the case study buildings.

Table 1. Seismic Response Coefficients Used for Case Study Buildings								
Stories	Risk	Story Height (ft)	C_uT_a (sec)	T_{RAM} (sec)	C_s	Governing ASCE/SEI 7 Equation for C_s	$C_{s,drift}$	$C_s/C_{s,drift}$
2	II	14.0	0.56	1.02	0.166	12.8-3	0.092	1.81
	IV	14.5	0.58	0.66	0.243	12.8-3	0.213	1.14
4	II	14.0	0.98	1.83	0.096	12.8-3	0.051	1.86
	IV	15.0	1.04	1.26	0.136	12.8-3	0.112	1.21
6	II	14.5	1.40	2.20	0.067	12.8-3	0.043	1.58
	IV	15.0	1.43	1.22	0.115	12.8-3	0.115	1.00
8	II	14.5	1.76	2.93	0.062	12.8-5	0.032	1.93
	IV	15.0	1.81	1.57	0.092	12.8-5	0.090	1.03
12	II	14.0	2.36	3.75	0.062	12.8-5	0.025	2.46
	IV	14.5	2.43	2.25	0.092	12.8-5	0.063	1.48
16	II	14.0	2.97	3.92	0.062	12.8-5	0.024	2.58
	IV	15.0	3.14	2.90	0.092	12.8-5	0.048	1.91

for the shorter buildings, was governed by ASCE/SEI 7, Equation 12.8-3, with the upper-limit period C_uT_a (Equation 12.8.2) used for the design period, T. For eight-story and taller buildings, C_s was governed by the minimum given by ASCE/SEI 7, Equation 12.8-5, which was period independent. $C_{s,drift}$ was the effective value used for drift checks, for which the upper limit on period did not apply (ASCE/SEI 7, Section 12.8.6.2). For drift checks, Equation 12.8-5 did not apply (ASCE/SEI 7, Section 12.8.6.1).

The frame designs are summarized in the Appendix.

Case Study Building Outputs

The primary outputs from each case study building were the steel weight (gravity and lateral), lateral stiffness, and lateral strength. The steel weights were obtained from the RAM weight take-offs. Beam and column weights were determined based on the assigned beam and column sizes and the centerline dimensions in the model. The steel weights obtained from RAM were used with the cost index (Equation 1) to determine cost premiums for Risk Category IV buildings. Steel weights did not include continuity plates, doubler plates, or column base plates.

The elastic lateral stiffness of each building was computed using an equivalent lateral force (ELF) load combination. The ELF base shear was divided by the center-of-mass drift at the roof (in rad) under ELF loading to obtain an effective building stiffness.

The first-yield strength of each case study building, V_y, was computed based on the demand/capacity (D/C) ratios under the ELF load combinations. The ELF base shear was divided by the highest beam D/C to estimate the base shear associated with first yielding (a phi factor of 1.0 was used for the beam capacity for this calculation).

RESULTS FROM MATHEMATICAL DERIVATIONS

The results from the study are presented in two parts. This first part presents the results from the mathematical derivations and has four subsections. The first subsection presents the mathematical derivation of the IV/II stiffness ratio for drift-governed designs. The second subsection presents the derivation of the IV/II strength ratio for strength-governed designs. The third subsection presents upper and lower bounds for the cost premium for strength-governed Risk Category IV design, based on the mathematical derivations, additional assumptions, and the total building cost index (Equation 1). The final subsection presents upper and lower bounds of the cost premium for drift-governed Risk Category IV design, which is pertinent for most steel SMF.

Derivation of Stiffness Ratio

The following derivation is for a single-degree-of-freedom system on the descending branch of the design spectra. A later section will discuss the results in the context of multi-degree-of-freedom systems. The derivation establishes the relationship between the required stiffness for Risk Category II and Risk Category IV designs when the designs are drift-governed.

Figure 2 defines the terms used in the derivation. The points on the spectra represent two designs. The Risk Category II design has a design base shear, V_2, and a period,

T_2. The Risk Category IV design has a design base shear, V_4, and a period, T_4. Both designs are assumed to be on the descending branch of the spectra, where S_{D1} is the design spectral response acceleration parameter at a period of 1.0 second, W is the seismic weight of the system, and I_e is the importance factor (ASCE, 2016).

Equations 2 and 3 relate the base shears and periods, according to the design spectra (Figure 2).

$$V_2 = \frac{S_{D1}}{T_2\left(\frac{R}{1.0}\right)}W \tag{2}$$

$$V_4 = \frac{S_{D1}}{T_4\left(\frac{R}{1.0}\right)}W \tag{3}$$

Note that the importance factor is taken as 1.0 for convenience in both drift-governed cases because the importance factor gets canceled out later in the drift calculation [I_e is in the denominator of ASCE/SEI 7, Equation 12.8-15, when computing the drift (ASCE, 2016)].

Combining Equations 2 and 3 and rearranging relates the periods and base shears of the Risk Category II and IV designs to each other, as shown in Equation 4.

$$\frac{T_4}{T_2} = \frac{V_2}{V_4} \tag{4}$$

Assuming the designs are drift-governed, the elastic displacement of each system under the design base shear (design base shear divided by the stiffness, k) will be equal to the elastic drift limit (δ_2 for the Risk Category II design and δ_4 for the Risk Category IV design).

$$\frac{V_2}{k_2} = \delta_2 \tag{5}$$

$$\frac{V_4}{k_4} = \delta_4 \tag{6}$$

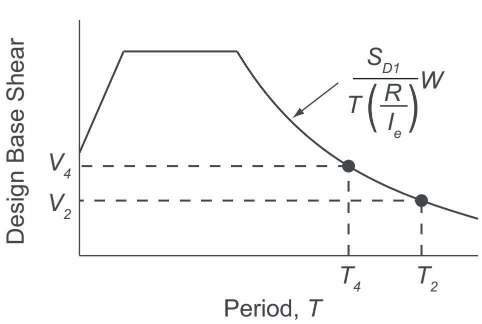


Fig. 2. Terms used for stiffness ratio derivation for drift-governed systems.

Combining Equations 5 and 6 and rearranging relates the drift ratio of the Risk Category II and IV designs to the design base shears and system stiffnesses.

$$\frac{\delta_2}{\delta_4} = \left(\frac{V_2}{V_4}\right)\left(\frac{k_4}{k_2}\right) \tag{7}$$

To combine Equations 4 and 7, the relationship between the system periods, T, and stiffnesses, k, is needed. The natural periods for the SDOF systems are:

$$T_2 = 2\pi\sqrt{\frac{m}{k_2}} \tag{8}$$

$$T_4 = 2\pi\sqrt{\frac{m}{k_4}} \tag{9}$$

where m is the mass (the same for both systems).

Combining Equations 8 and 9 and rearranging terms gives Equation 10.

$$\left(\frac{T_4}{T_2}\right)^2 = \frac{k_2}{k_4} \tag{10}$$

Combining Equations 10 and 4 and rearranging terms gives the relationship between the system stiffnesses and the design base shears, shown in Equation 11.

$$\frac{k_2}{k_4} = \left(\frac{V_2}{V_4}\right)^2 \tag{11}$$

And finally, combining Equations 11 and 7 and rearranging terms gives the relationship for the stiffness ratio in terms of the elastic drift limit ratio, shown in Equation 12.

$$\frac{k_4}{k_2} = \left(\frac{\delta_2}{\delta_4}\right)^2 \tag{12}$$

This result shows that if the elastic drift limit is decreased by a ratio of (δ_2/δ_4), then the stiffness of the design must increase by a factor of $(\delta_2/\delta_4)^2$ to meet the drift limit. This squared amplification of the drift limit ratio is the result of the period effect (as the system is stiffened the design force increases). For the common case where the drift limit for a Risk Category II design is twice that of a Risk Category IV design, the Risk Category IV design will require a stiffness that is four times that of the Risk Category II design.

For reference, a different substitution and rearrangement of Equations 11 and 7 gives the relationship between the design base shears for the two systems, shown in Equation 13.

$$\frac{V_4}{V_2} = \frac{\delta_2}{\delta_4} \tag{13}$$

Derivation of Strength Ratio

The following derivation is for a single-degree-of-freedom system on the descending branch of the design spectra. Later sections will discuss the result in the context of multi-degree-of-freedom systems. The derivation shows the relationship between the required strength for Risk Category II and Risk Category IV systems when the designs are strength-governed and based on the actual building period.

Figure 3 defines the terms used in the derivation. The points on the spectra represent two designs. The Risk Category II design has a design base shear, V_2, and a period, T_2. The Risk Category IV design has a design base shear, V_4, and a period, T_4. The Risk Category IV design is on the upper curve corresponding to the 1.5 importance factor.

Equations 14 and 15 relate the base shears and periods, according to the respective spectra.

$$V_2 = \frac{S_{D1}}{T_2\left(\frac{R}{1.0}\right)}W \tag{14}$$

$$V_4 = \frac{S_{D1}}{T_4\left(\frac{R}{1.5}\right)}W \tag{15}$$

Combining Equations 14 and 15 and rearranging relates the periods and base shears of the Risk Category II and IV systems to each other, shown in Equation 16.

$$V_4T_4 = 1.5V_2T_2 \tag{16}$$

Rearranging Equation 16 gives:

$$\left(\frac{T_4}{T_2}\right)^2 = 1.5^2\left(\frac{V_2}{V_4}\right)^2 \tag{17}$$

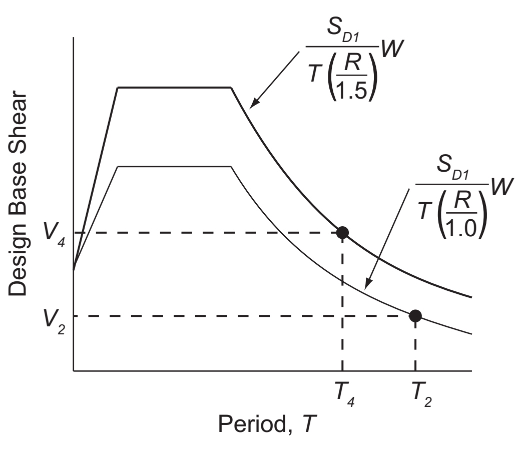


Fig. 3. Terms used for strength ratio derivation for strength-governed system.

The natural periods for the SDOF systems are:

$$T_2 = 2\pi\sqrt{\frac{m}{k_2}} \tag{18}$$

$$T_4 = 2\pi\sqrt{\frac{m}{k_4}} \tag{19}$$

where m is the mass (the same for both systems), and k_2 and k_4 are the respective stiffnesses.

Combining Equations 18 and 19 and rearranging terms gives Equation 20.

$$\left(\frac{T_4}{T_2}\right)^2 = \frac{k_2}{k_4} \tag{20}$$

Combining Equations 20 and 17 results in Equation 21.

$$\frac{k_2}{k_4} = 1.5^2\left(\frac{V_2}{V_4}\right)^2 \tag{21}$$

Assuming that the strength and stiffness ratios of the systems are similar results in Equation 22.

$$\frac{k_2}{k_4} \approx \frac{V_2}{V_4} \tag{22}$$

Combining Equations 22 and 21 gives the result shown in Equation 23.

$$\frac{V_4}{V_2} \approx 1.5^2 \approx 2.25 \tag{23}$$

This shows that using an importance factor of 1.5 to increase strength demands results in Risk Category IV designs that can be about 2.25 times stronger than a Risk Category II design. The squared amplification of the importance factor is the result of the period effect (as the system is strengthened the force demands increase).

The result in Equation 23 is based on the assumption that the actual building period is being used in determining the design base shear. For strength-governed design, the design period is limited to C_uT_a (ASCE, 2016), which affects the validity of Equation 22. As such, Equation 22 should be viewed as an upper bound. If C_uT_a governed the design period for the Risk Category II design, then the strength ratio would be lower. If C_uT_a governed the design period for both the Risk Category II and Risk Category IV design, then the required strength ratio would simply be 1.5 because there would be no period effect.

As a final consideration, for some strength-governed systems, the elastic overstrength of a Risk Category II design approaches 1.5 so that even if the design base shear were 1.5 times greater, the structural design could remain mostly unchanged for a Risk Category IV design. This is not pertinent for steel SMF design but will be discussed for broader applications. For example, in a Risk Category II steel braced

Table 2. Cost Estimates for Stiffness-Governed Risk IV Design Scenarios

Scenario	Example	Stiffness Ratio (IV/II)	W_{SMF}/W_t [a]	$W_{t,IV}/W_{t,II}$ [b]	Cost Index [c]
Upper bound	Over four stories and high percentage of lateral frame weight	4.0	0.60	2.08	1.22
Middle	Over four stories	4.0	0.40	1.72	1.14
Lower bound	Under four stories	2.78	0.35	1.33	1.07

[a] This is the assumed ratio of the weight of the SMF steel to the total steel weight.
[b] This ratio was estimated as: $(1 - W_{SMF}/W_t) + (W_{SMF}/W_t)$(Stiffness ratio)(0.7). The 0.7 is a multiplier that accounts for higher efficiency in the Risk IV frame due to more efficient shapes and was based on previous design experience.
[c] See Equation 1.

Table 3. Cost Estimates for Strength-Governed Risk IV Design Scenarios

Scenario	Example	Strength Ratio (IV/II)	W_{SMF}/W_t [a]	$W_{t,IV}/W_{t,II}$ [b]	Cost Index [c]
Upper bound	Actual building periods used for designs	2.25	0.40	1.5	1.10
Middle	C_uT_a used for designs	1.5	0.30	1.15	1.03
Lower bound	C_uT_a used for designs and natural elastic overstrength	1.2	0.20	1.04	1.01

[a] This is the assumed ratio of the weight of the SMF steel to the total steel weight.
[b] This ratio was estimated as: $(1 - W_{SMF}/W_t) + (W_{SMF}/W_t)$(Stiffness ratio).
[c] See Equation 1.

frame, the braces may be oversized to meet width-thickness requirements, and the same frame may be almost adequate for the Risk Category IV design. For some systems, then, the actual IV/II strength ratio will be less than even 1.5. For discussion purposes, we will pick 1.2 as an estimated lower bound.

Cost Premiums for Drift-Governed Designs (Mathematical Bounds)

For stiffness governed designs, the stiffness ratio is estimated from Equation 12. The upper-bound stiffness ratio for buildings four stories or less is $(2.5/1.5)^2 = 2.78$. For buildings greater than four stories, it is $(2.0/1.0)^2 = 4.0$. Table 2 shows cost estimates for upper-bound, lower-bound, and middle cases based on this range of possible stiffness ratios (IV/II), assumptions about the SMF weight relative to the total steel weight, W_{SMF}/W_t, and estimates of corresponding weight ratios, $W_{t,IV}/W_{t,II}$.

Table 2 indicates a range of cost premiums for stiffness-governed designs of 1.07 to 1.22. Because the cost premiums in Table 2 are based on estimates for $W_{t,IV}/W_{t,II}$, they are less reliable than those obtained from the case studies, which will be discussed later.

Cost Premiums for Strength-Governed Designs (Mathematical Bounds)

For strength-governed designs, the expected strength ratio based on the mathematical derivation ranged from about 1.2 to 2.25. Table 3 shows cost estimates for upper-bound, lower-bound, and middle cases, based on this range of possible strength ratios (IV/II), assumptions about the SMF weight relative to the total steel weight, W_{SMF}/W_t, and estimates of corresponding weight ratios, $W_{t,IV}/W_{t,II}$.

The cost index values in Table 3 indicate a range of cost premiums from 1 to 10% for strength-governed designs. The lower-bound premium of about 1% is consistent with the strength-governed studies previously cited (NIST, 2013; Yu et al., 2015). The characteristics of buildings that will be near the lower-bound premium are as follows:

- Design is strength-governed.
- C_uT_a is used for the design period.
- There is some elastic overstrength.
- The weight of the lateral force-resisting system is a small percentage of the total structural weight.

Table 4. Summary of Stiffness Results from the Case Study Buildings				
	Stiffness			
Stories	II (kips/rad)[a]	IV (kips/rad)[a]	IV/II	Equation 12 Stiffness Ratio
2	83476	194515	2.33	2.78
4	82884	197741	2.39	2.78
6	129247	459756	3.56	4.00
8	137112	471614	3.44	4.00
12	76406	232186	3.04	4.00
16	123299	247306	2.01	4.00

[a] Because the Risk Category II and Risk Category IV designs have slightly different building heights (Table 1), the stiffness is defined as lateral force over drift ratio, so that the IV/II ratio will not be distorted by the varying heights.

Many buildings have these characteristics and would have Risk Category IV cost premiums around 1 to 2%, but steel SMF generally do not have these characteristics. The results from the mathematical derivations demonstrate how drift-governed buildings can have a much greater construction cost premium for Risk Category IV design than strength-governed buildings.

RESULTS FROM CASE STUDY BUILDINGS

This second part of the results focuses on the case study buildings and has four subsections. The first subsection presents results on the stiffness of the case study buildings and discusses them relative to Equation 12. The second subsection discusses the strength of the case study buildings. The third subsection presents the weights and weight ratios for the case study buildings. The final subsection presents the cost premiums calculated for case study buildings and discusses them in the context of the theoretical ranges established in the previous sections.

Stiffness of Case Study Buildings

The elastic stiffness of each of the case study buildings (lateral force/roof drift) was obtained from the corresponding RAM model. Table 4 summarizes the stiffness results, including IV/II stiffness ratios. Also shown in the Table 4 is the upper-bound stiffness ratio from Equation 12, which differs depending on the ratio of the allowable drifts (different for buildings four stories and less).

The stiffness ratios from the case study buildings were lower than those from Equation 12. For the 2- and 4-story buildings, the stiffness ratios (IV/II) were 14 to 16% less than those from Equation 12 (Table 4). For the 6- and 8-story buildings, the case study stiffness ratios were 11 to 14% less than those from Equation 12, with the best match for the six-story buildings. For the 12- and 16-story buildings, the designs were not governed by the equations assumed in the derivation of Equation 12 (they were governed by minimum base shear equations, Table 1), so the stiffness ratios were farther from Equation 12.

The case study values of the stiffness ratio are valuable for understanding the stiffness ratios of as-designed buildings, while Equation 12 (the derivation) is helpful for understanding why the stiffness ratio is substantially higher than the ratio of the drift limits for designs on the descending branch of the design spectra. The case studies confirm that Risk Category IV SMF buildings on the descending branch of the design spectra can approach four times the stiffness of Risk Category II SMF buildings.

Strength of Case Study Buildings

The strengths, V_y, of the case study buildings are summarized in Table 5, including the IV/II strength ratios. Strength ratios for the case study SMF buildings (Table 5) were often higher than the range that was developed for strength-governed systems (Table 2) because most of the case study buildings were drift-governed. For the 6- to 12-story buildings, the IV/II strength ratios exceeded 3.0 (Table 5). This observation has implications for foundation design and may be surprising for designers that expect a Risk Category IV building to only be 1.5 to 2.0 times stronger than a Risk Category II building.

Weight of Case Study Buildings

The weight of each case study building was obtained from the RAM takeoffs. Table 6 summarizes the weight results for the individual buildings as well as various IV/II weight ratios. Figures 4 and 5 illustrate some relationships of interest.

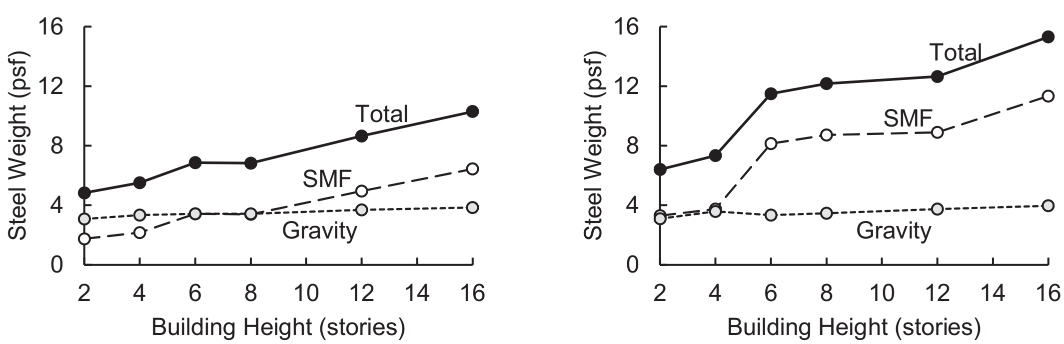
Figure 4 shows the steel weights (gravity, SMF, and total) for the different buildings, expressed in pounds per square foot (psf). The gravity steel weights were similar for all the buildings (around 3 psf, a little higher for the taller

Table 5. Summary of Strength Results from the Case Study Buildings							
	Seismic Weight, W (kips)		Strength, V_y			V_y/W	
Stories	II	IV	II (kips)	IV (kips)	IV/II	II	IV
2	3506	3583	653	1597	2.45	0.19	0.45
4	7084	7293	741	1586	2.14	0.10	0.22
6	10849	11470	1059	3374	3.18	0.10	0.29
8	14466	15432	1015	3206	3.16	0.07	0.21
12	10847	11451	568	1721	3.03	0.05	0.15
16	14736	15696	989	1730	1.75	0.07	0.11

Table 6. Weight Summaries for the Case Study Buildings										
	SMF Steel Weight, W_{SMF}			Gravity Steel Weight, W_{grav} (psf)		Total Steel Weight, W_t			W_{SMF}/W_t	
Stories	II (psf)	IV (psf)	IV/II	II	IV	II (psf)	IV (psf)	IV/II	II	IV
2	1.75	3.31	1.89	3.08	3.10	4.83	6.40	1.32	0.36	0.52
4	2.17	3.74	1.72	3.35	3.59	5.52	7.33	1.33	0.39	0.51
6	3.43	8.15	2.37	3.44	3.35	6.87	11.5	1.67	0.50	0.71
8	3.40	8.72	2.56	3.44	3.46	6.84	12.2	1.78	0.50	0.72
12	4.96	8.91	1.79	3.68	3.74	8.65	12.6	1.46	0.57	0.70
16	6.44	11.3	1.76	3.85	3.97	10.3	15.3	1.49	0.63	0.74

buildings). For the 2- and 4-story Risk Category II buildings [Figure 4(a)], the SMF steel weight was less than the gravity, but for other buildings, SMF steel weight was greater than the gravity. Figure 4(b) shows that for the 6-story and taller Risk Category IV buildings, the SMF weight was several times the gravity steel weight.

The "jumps" in SMF weight in Figure 4(b) between the 4- and 6-story designs and between the 12- and 16-story designs were due to changes in the design criteria. The allowable drifts were different for the 4-story-and-under designs [ASCE/SEI 7, Table 12.12-1 (ASCE, 2016)], which explained the weight jump at 6 stories. The 16-story



(a) Risk Category II designs

(b) Risk Category IV designs

Fig. 4. Steel weights for the case study buildings.

Table 7. Cost Index for Case Study Buildings		
Stories	$W_{t,IV}/W_{t,II}$	Cost Index
2	1.32	1.06
4	1.33	1.07
6	1.67	1.13
8	1.78	1.16
12	1.46	1.09
16	1.49	1.10

buildings had higher $C_s/C_{s,drift}$ ratios in design than the 12-story buildings (Table 1), making the 16-story buildings more strength-controlled.

Figure 5 shows two weight ratios (total and SMF) along with two other IV/II ratios of interest from the study. The SMF weight ratio (long dash) is as high as 2.6, indicating the substantial increase in SMF steel required for Risk Category IV designs on the descending branch of the spectra. The total weight ratio is lower, with a maximum of 1.7, because the relatively constant gravity steel dilutes the SMF steel increase. The total weight ratio in Figure 5 is of particular interest since it is the parameter in the cost index (Equation 1). Figure 5 shows that the total steel weight ratio increases for buildings over 4 stories, peaks around 8 stories, and decreases for taller buildings when minimum base shear equations start to govern both the Risk Category II and IV designs.

The SMF weight ratios were 12 to 40% lower than the stiffness ratios (comparing dashed lines Figure 5), reflecting better lateral weight efficiency for the Risk Category IV buildings. The primary source of that efficiency was the deeper heavier beams. For example, the 8-story Category Risk Category II buildings used W27×94 beams at the bottom (because they were sufficient) while the Risk Category IV buildings used W36×231. One measure of stiffness efficiency of a flexural member is I/w, where I is the moment of inertia and w is the weight in pounds per foot. For a W36×231, I/w was 68 in.^{4}/lb, whereas for a W27×94, I/w was 35 in.^{4}/lb. The more efficient sections in the Risk Category IV buildings allowed the SMF weight increase to be less than the stiffness increase.

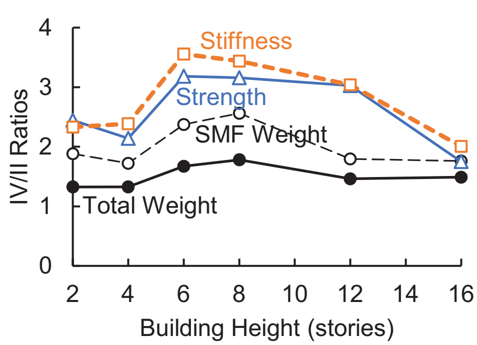


Fig. 5. Various ratios comparing the Risk Category IV and II buildings.

Also note in Figure 5 that the stiffness and strength IV/II ratios track each other quite closely. This was expected for the steel buildings, where both strength and stiffness were directly related to the cross-sectional properties of the steel members.

Cost Premium of Case Study Buildings

The total weight ratio, $W_{t,IV}/W_{t,II}$, was used with Equation 1 to determine cost premiums for the case study buildings. Table 7 repeats the IV/II weight ratio from Table 6 and shows the associated cost index. Table 7 indicates a range of 1.07 to 1.17 for the cost index, which corresponds to cost premiums of 7 to 17% for the Risk IV SMF buildings. The premiums increased for buildings over 4 stories, peaked at 8 stories, and decreased for taller buildings when minimum base shear equations start to govern both the Risk Category II and IV designs.

The 6 to 16% cost premium range for the SMF buildings in this study was a little higher than the 4 to 14% range mentioned in Richards et al. (2023). The shift was due to the following differences in the SMF design criteria of the two studies:

- In the previous study, a constant story height of 12 ft was assumed for all the buildings to match the buildings of an earlier study. As a result, the clear-heights varied and were unrealistically low (9 ft) for modern steel buildings. In the present study, a consistent clear-height (at least 12 ft) was used for all the buildings.
- In the previous study, the column depths were different for the Risk Category II (W24) and IV (W27) buildings. In the present study, the column depths were constrained to be the same for all (W27).

- In the previous study, a constant conservative steel weight was assumed for dead loads and seismic weight. In the present study, the actual steel weight based on member sizes was used so the seismic weight was different for the Risk Category II and IV buildings.

All things considered, both studies had similar findings, and the 6 to 16% construction cost premium range from the present study is more accurate for current practice. These cost premiums can be compared with the cost of other alternatives for post-earthquake functional recovery. Other studies have demonstrated much more economical approaches for the functional recovery of steel SMF buildings rather than using Risk Category IV design [see Richards et al. (2023)].

Mitigating Cost Premiums

Even deeper columns may be used to help mitigate construction cost premiums for Risk Category IV design but can only bring prices down a little. In the study buildings, the column depth was kept consistent (W27) for the Risk Category II and IV designs to allow a comparison of architecturally equivalent systems. To investigate the potential savings from deeper columns, an alternative design for the Risk Category IV 8-story building was generated with W33 columns. The member sizes are shown in the Appendix (Table 9). The design with W33 columns saved 7% on the total weight, but the cost index for the Risk Category IV design with W33 columns was still 1.13 (13% total building cost premium as compared to Risk Category II design).

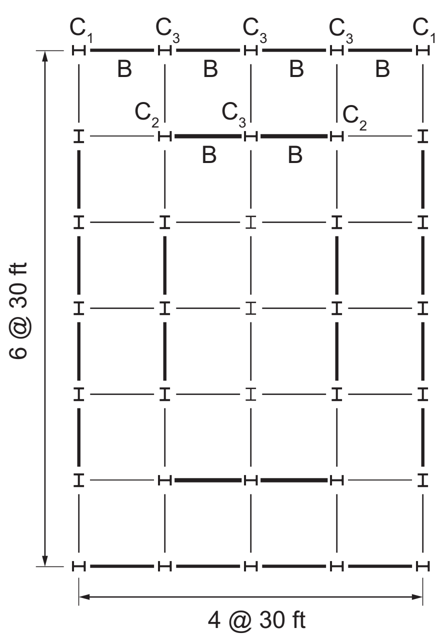


Fig. 6. SMF layout for an alternative design for a Risk Category IV 8-story building that did not reduce overall steel weight.

Adding additional moment frames was not helpful in reducing the construction cost premiums much. To investigate the effect of additional moment frames bays on cost, an alternative design for the Risk Category IV 8-story building was generated with the frame layout shown in Figure 6. The number of moment frame bays in each direction was increased from 8 to 12. The member sizes for this design are shown in the Appendix (Table 10). Adding additional frames was helpful for reducing the weights of individual members (beams and columns) but not for significantly reducing the overall building weight. The total steel weight (beams and columns) for the design with extra frames (11.9 psf) was 2% less than the baseline Risk Category IV design (Table 6), but the added moment connections would increase fabrication and erection costs, and the W27 columns in the interior spaces would be architecturally intrusive. The observations from this comparison are consistent with another study (McCall and Richards, 2022), and professional practice that generally uses SMF with deep columns on the perimeters and minimizes the number of SMF connections as long as the beam and column sizes are prequalified and architecturally acceptable.

SUMMARY AND CONCLUSIONS

As engineers contemplate the use of Risk Category IV design to address post-earthquake functional recovery, it is helpful to quantify the cost of Risk Category IV design for strength-governed and drift-governed buildings. Limited studies have suggested that cost premiums are higher for drift-governed SMF buildings.

Two complementary methods were used in this study. The first method was to mathematically derive the difference in strength and stiffness for Risk Category II and IV designs, accounting for period effects. Closed-form solutions were possible for single-degree-of-freedom (SDOF) systems. The second method was to use case study buildings to compute cost premiums. Twelve SMF buildings with varying height (2-, 4-, 6-, 8-, 12-, and 16-story) and varying Risk Category (II and IV) were designed. The steel weights from the case study buildings were used with a cost index to estimate the construction cost premiums in terms of total building cost.

Conclusions from the mathematical derivations included the following:

- For drift-governed buildings, the upper bound for the stiffness multiplier for Risk Category IV design was the square of the allowable drift ratio. For buildings over four-stories, it was a factor of four.
- For strength-governed buildings, the upper bound for the required strength multiplier for Risk Category IV design was the square of the importance factor. For an importance factor of 1.5, the upper bound on the strength

Table 8. Case Study SMF Designs									
		Risk Category II				Risk Category IV			
Stories	**Story**	**C1**	**C2**	**C3**	**B**	**C1**	**C2**	**C3**	**B**
2	1	W27×94	—	W27×94	W24×55	W27×217	—	W27×102	W30×108
	2	W27×94	—	W27×94	W21×44	W27×217	—	W27×102	W27×94
4	1	W27×114	—	W27×114	W24×76	W27×258	—	W27×146	W33×130
	2	W27×114	—	W27×114	W24×76	W27×258	—	W27×146	W33×130
	3	W27×94	—	W27×94	W24×62	W27×217	—	W27×114	W30×108
	4	W27×94	—	W27×94	W21×44	W27×217	—	W27×114	W24×62
6	1	W27×178	—	W27×114	W27×94	W27×539	—	W27×235	W36×194
	2	W27×178	—	W27×114	W27×94	W27×539	—	W27×235	W36×182
	3	W27×161	—	W27×114	W24×84	W27×539	—	W27×194	W36×182
	4	W27×161	—	W27×114	W24×84	W27×539	—	W27×194	W36×150
	5	W27×114	—	W27×94	W24×76	W27×307	—	W27×178	W33×130
	6	W27×114	—	W27×94	W18×35	W27×307	—	W27×178	W24×76
8	1	W27×178	—	W27×146	W27×94	W27×539	—	W27×307	W36×231
	2	W27×178	—	W27×146	W27×94	W27×539	—	W27×307	W36×231
	3	W27×161	—	W27×114	W27×94	W27×539	—	W27×235	W36×194
	4	W27×161	—	W27×114	W27×94	W27×539	—	W27×235	W36×170
	5	W27×129	—	W27×114	W24×76	W27×539	—	W27×178	W36×160
	6	W27×129	—	W27×114	W24×76	W27×539	—	W27×178	W33×141
	7	W27×94	—	W27×94	W24×55	W27×281	—	W27×161	W33×141
	8	W27×94	—	W27×94	W18×40	W27×281	—	W27×161	W24×84

multiplier was 2.25. This upper bound would rarely be reached in practice because of the $C_u T_a$ limit on the design period.

Conclusions from the case study buildings included the following:

- Risk Category IV SMF buildings had 2.0 to 3.6 times the stiffness of Risk Category II buildings. The stiffness ratios for buildings on the descending branch of the design spectra were 11 to 16% lower than the upper bound found from the closed-form solution (SDOF).
- Risk Category IV SMF buildings had 1.8 to 3.2 times the strength of Risk Category II buildings.
- Risk Category IV SMF buildings had 1.8 to 2.6 times the SMF weight of Risk II buildings, but when gravity steel weight was included, the Risk Category IV buildings only had 1.3 to 1.9 times the total steel weight.
- The cost premiums for Risk Category IV SMF buildings were 6 to 16% of the total building cost, with the greatest premiums for the eight-story building. These premiums were substantially greater than the 1% cost premiums that have been reported in studies with strength-governed buildings.

Some limitations of the study were that only steel SMF buildings were included in the case studies, and only one SMF connection type was considered. However, results were similar to another SMF study (Richards et al., 2023) that considered three different types of connections.

APPENDIX

The designs for the case study buildings are summarized in Table 8. See Figure 1 for the definitions of the column and beam tags.

Table 8. Case Study SMF Designs (continued)									
Stories	Story	Risk Category II				Risk Category IV			
		C1	C2	C3	B	C1	C2	C3	B
12	1	W27×129	W27×161	W27×146	W24×55	W27×217	W27×281	W27×217	W30×124
	2	W27×129	W27×161	W27×146	W24×55	W27×217	W27×281	W27×217	W30×124
	3	W27×129	W27×146	W27×129	W24×55	W27×194	W27×258	W27×178	W30×116
	4	W27×129	W27×146	W27×129	W24×55	W27×194	W27×258	W27×178	W30×108
	5	W27×129	W27×146	W27×114	W21×50	W27×194	W27×258	W27×146	W30×108
	6	W27×129	W27×146	W27×114	W21×50	W27×194	W27×258	W27×146	W30×108
	7	W27×94	W27×114	W27×102	W21×50	W27×178	W27×194	W27×114	W27×102
	8	W27×94	W27×114	W27×102	W21×44	W27×178	W27×194	W27×114	W27×94
	9	W27×94	W27×94	W27×94	W18×40	W27×161	W27×178	W27×102	W27×94
	10	W27×94	W27×94	W27×94	W18×35	W27×161	W27×178	W27×102	W24×76
	11	W27×94	W27×94	W27×94	W18×35	W27×102	W27×129	W27×194	W24×76
	12	W27×94	W27×94	W27×94	W18×35	W27×102	W27×129	W27×194	W18×40
16	1	W27×146	W27×235	W27×217	W24×84	W27×336	W27×539	W27×307	W33×130
	2	W27×146	W27×235	W27×217	W24×84	W27×336	W27×539	W27×307	W33×130
	3	W27×146	W27×217	W27×217	W24×84	W27×336	W27×368	W27×258	W33×130
	4	W27×146	W27×217	W27×217	W24×84	W27×336	W27×368	W27×258	W30×116
	5	W27×146	W27×194	W27×161	W24×84	W27×307	W27×307	W27×235	W30×116
	6	W27×146	W27×194	W27×161	W24×76	W27×307	W27×307	W27×235	W30×116
	7	W27×129	W27×178	W27×146	W24×76	W27×258	W27×281	W27×161	W30×108
	8	W27×129	W27×178	W27×146	W24×76	W27×258	W27×281	W27×161	W30×108
	9	W27×129	W27×161	W27×129	W24×76	W27×258	W27×258	W27×146	W30×108
	10	W27×129	W27×161	W27×129	W24×62	W27×258	W27×258	W27×146	W27×102
	11	W27×94	W27×114	W27×114	W24×62	W27×235	W27×235	W27×114	W27×102
	12	W27×94	W27×114	W27×114	W24×55	W27×235	W27×235	W27×114	W27×94
	13	W27×94	W27×102	W27×102	W24×55	W27×194	W27×194	W27×114	W27×94
	14	W27×94	W27×102	W27×102	W21×44	W27×194	W27×194	W27×114	W24×76
	15	W27×94	W27×94	W27×94	W18×35	W27×161	W27×161	W27×94	W24×55
	16	W27×94	W27×94	W27×94	W18×35	W27×161	W27×161	W27×94	W18×35

Table 9. Eight-Story Comparison Design with Deeper Columns									
Stories	Story	Risk Category II				Risk Category IV			
		C1	C2	C3	B	C1	C2	C3	B
8	1	W27×178	—	W27×146	W27×94	W33×263	—	W33×387	W36×232
	2	W27×178	—	W27×146	W27×94	W33×263	—	W33×387	W36×232
	3	W27×161	—	W27×114	W27×94	W33×263	—	W33×354	W36×194
	4	W27×161	—	W27×114	W27×94	W33×263	—	W33×354	W36×194
	5	W27×129	—	W27×114	W24×76	W33×221	—	W33×354	W36×160
	6	W27×129	—	W27×114	W24×76	W33×221	—	W33×354	W36×150
	7	W27×94	—	W27×94	W24×55	W33×169	—	W33×354	W36×150
	8	W27×94	—	W27×94	W18×40	W33×169	—	W33×354	W21×44

Table 10. Eight-Story Comparison Design with Additional Moment Frames									
Stories	Story	Risk Category II				Risk Category IV			
		C1	C2	C3	B	C1	C2	C3	B
8	1	W27×178	—	W27×146	W27×94	W27×217	W27×235	W27×368	W36×160
	2	W27×178	—	W27×146	W27×94	W27×217	W27×235	W27×368	W36×160
	3	W27×161	—	W27×114	W27×94	W27×194	W27×194	W27×368	W36×150
	4	W27×161	—	W27×114	W27×94	W27×194	W27×194	W27×368	W36×150
	5	W27×129	—	W27×114	W24×76	W27×161	W27×146	W27×258	W33×141
	6	W27×129	—	W27×114	W24×76	W27×161	W27×146	W27×258	W33×130
	7	W27×94	—	W27×94	W24×55	W27×102	W27×102	W27×194	W30×116
	8	W27×94	—	W27×94	W18×40	W27×102	W27×102	W27×194	W24×55

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