ASME TR A17.1-8.4–2013
(Technical Report)

Guide for Elevator Seismic Design

The American Society of
Mechanical Engineers

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Guide for Elevator Seismic Design

The American Society of Mechanical Engineers
Two Park Avenue • New York, NY • 10016 USA

Date of Issuance: March 31, 2014

This Technical Report will be revised when the Society approves the issuance of a new edition.

The American Society of Mechanical Engineers
Two Park Avenue, New York, NY 10016-5990

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FOREWORD

Seismic requirements have been part of ASME A17.1/CSA B44 since 1981 with their introduction in Appendix F. A17.1/B44 seismic requirements are based on input provided from building code seismic maps and charts. Since the mid-1980s, building codes and their seismic maps and charts have undergone major modifications. These modifications created difficulty for the user to properly apply A17.1/B44 requirements in jurisdictions using the latest building codes. This difficulty necessitated the need to realign the A17.1/B44 earthquake requirements with the latest building codes. The 2013 edition of ASME A17.1/CSA B44 introduces a completely revised Earthquake Safety Section 8.4, realigned with the latest building codes available at the time, IBC 2009 and NBCC 2010.

In conjunction with the publication of ASME A17.1-2013/CSA B44-13, this first edition of the Guide for Elevator Seismic Design is being released. The Guide was prepared by the ASME A17.1/CSA B44 Earthquake Safety Committee. This Guide is intended as an aid to the user to better understand the history behind the development of the latest building and elevator safety codes, the rationale behind the latest Section 8.4 revisions, and the proper application of the Section 8.4 requirements in conjunction with a jurisdiction's adopted building code.

Publication of this Technical Report has been approved by ASME in accordance with the Procedures for Development of ASME Technical Reports. This Guide is not an American National Standard and the material contained herein is not normative in nature. Comments on the content of this Guide should be sent to the Secretary, A17 Standards Committee, The American Society of Mechanical Engineers, Two Park Avenue, New York, NY 10016-5990.

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Part 1
Modification of ASME A17.1-2010, Section 8.4, Elevator Safety Requirements for Seismic Risk Zone 2 or Greater

1-1 SCOPE

This Guide provides rationale for elevator seismic force determination in Section 8.4. It details ASME A17.1 harmonization efforts with all building codes and summarizes the harmonization impact on elevator design via force comparisons based on component, component mounting location, and building geographical location, and provides an International Building Code (IBC) quick reference for seismic requirements and equivalent zone force levels.

1-2 INTRODUCTION

For many years, U.S. and Canadian model building codes such as the Uniform Building Code (UBC), Standard (Southern) Building Code (SBC), and National Building Code of Canada (NBCC) differentiated the force levels expected during seismic activity by zones. For example, a building in a zone 1 location was expected to see lower seismic forces than a building in a zone 2 location. A United States Geological Survey (USGS) map of the U.S. (see Fig. 1-2-1), published in the various building codes, indicated the appropriate zone for any part of the country.

Seismic requirements were first specified in ASME A17.1-1981, Appendix F. They were based on ANSI A58.1, the American National Standard Building Code Requirements for Minimum Design Loads in Buildings and Other Structures. Seismic force levels that the elevator must withstand would vary based on whether the subject building was in a zone 2 or zone 3 location. Zone 1 locations did not have elevator seismic requirements. Therefore, to determine elevator seismic forces for any part of the country, one would review the appropriate, adopted building code for that particular location, determine the zone for that location from the seismic zone map used by that building code, and then reference the appropriate elevator forces for that zone in A17.1.

In the mid-1980s, the National Earthquake Hazard Reduction Program (NEHRP) published its Recommended Provisions for the Development of Seismic Regulations for New Buildings with new seismic maps from the USGS. Instead of using zones, these new contour maps designated seismic ground motion in terms of a velocity-related coefficient, A_v. The ground motion parameter, in addition to other building variables, was input into an equation to determine seismic force levels for building structural (buildings) and nonstructural components (elevators, escalators, etc.). Throughout the late 1980s and 1990s, the model building codes [Building Officials and Code Administrators International, Inc. (BOCA), UBC, SBC] began adopting these new maps and variations of the NEHRP seismic force equation into their codes. In Canada, the 1985 edition of NBCC discarded Canada's traditional seismic zones for seven seismic zones based on the velocity-related seismic zone parameter, Z_v.

With different building codes using different seismic force equations and no longer using traditional seismic zone maps, the need to properly align the A17.1/B44 seismic requirements with the new building codes became imperative. Requirement 8.4.13, introduced in the harmonized ASME A17.1/CSA B44 2000 edition, correlated ground motion parameters (such as A_v and Z_v) to the traditional seismic zones. Using this correlation, the A17.1/B44 requirements could continue to be used as written.

For reference, the correlating values were as follows:

(U.S.: See A17.1/B44, 8.4.13.1)

Zone(s)	Affected Peak Velocity Acceleration, A_v
0 and 1	$A_v < 0.10$
2	$0.10 \le A_v < 0.20$
3 and 4	$0.20 \le A_v$

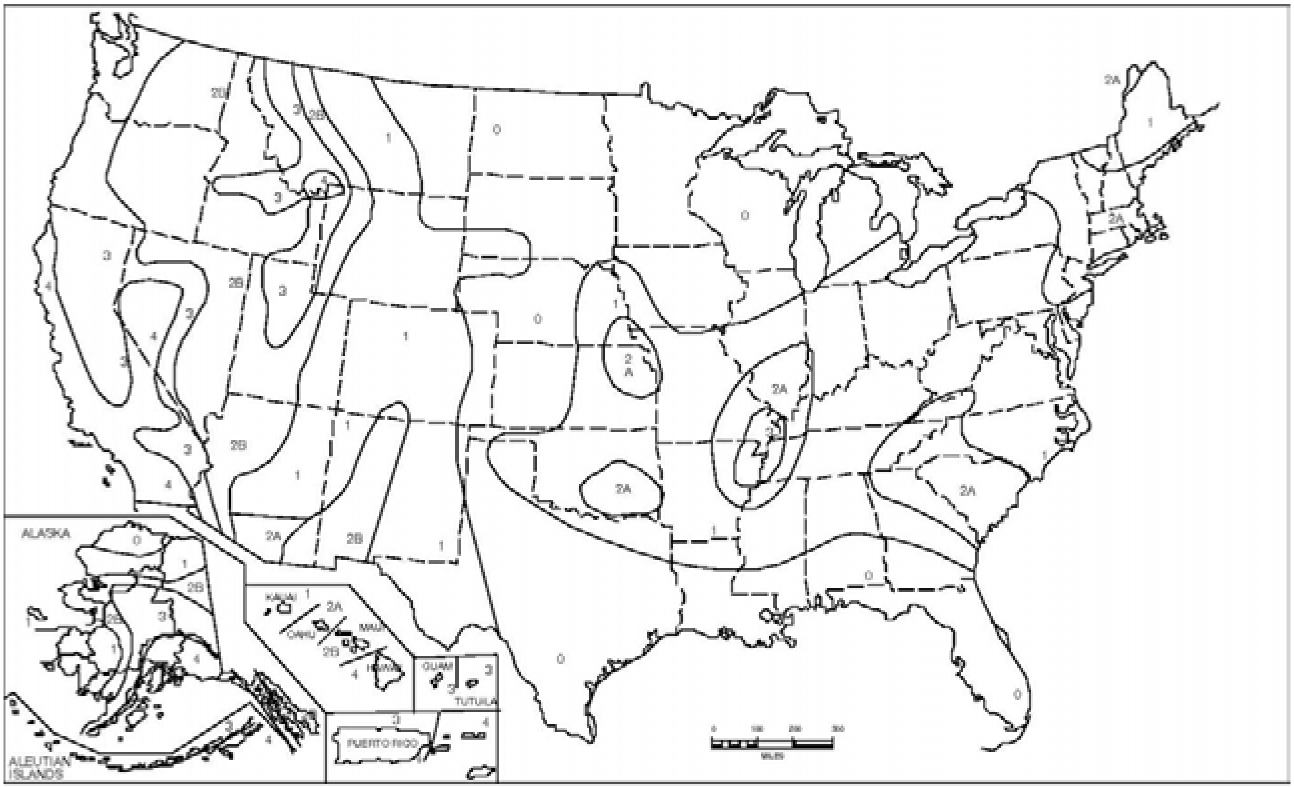
(Canada: See A17.1/B44, 8.4.13.2)

Zone(s)	Velocity-Related Seismic Zone, Z_v
2	$2 \le Z_v < 4$
≥ 3	$4 \le Z_v$

NOTE: All future references in this Guide refer to ASME A17.1/CSA B44 unless otherwise stated.

In 1994, the three U.S. model building codes [International Conference of Building Officials (ICBO), BOCA, and Southern Building Code Conference International

Fig. 1-2-1 Seismic Zone Map



NOTE:
As reproduced from
Seismic Zone Map
Excerpted from the 1994 SBCCI Standard Building Code, Copyright 1994.
Figure 16-2 Seismic Zone Map
Excerpted from the 1997 Uniform Building Code, Copyright 1997.
Washington, D.C.: International Code Council.

(SBCCI)] established the International Code Council (ICC). In 2000, ICC began publishing one comprehensive code, the International Building Code (IBC). The IBC 2000 code used the latest USGS maps (now contour maps with a ground motion parameter of earthquake spectral response acceleration) and NEHRP guidelines for its seismic force requirements. ASCE 7-02, recognized as the U.S. standard for seismic force requirements, was referenced by IBC 2003. As with IBC 2000, ASCE 7-02 and later editions referenced the latest USGS maps and NEHRP guidelines as the basis for its force requirements. Similar to IBC, the NBCC 2005 code used location-specific spectral response acceleration values (published in chart form) and NEHRP guidelines as the basis for its seismic force requirements.

Since their introduction in April 2000 and 2005, respectively, the IBC and NBCC 2005 have been adopted by a majority of jurisdictions as their building code. Because the maps or charts no longer refer to zones or the A_v or Z_v parameters, A17.1/B44 seismic requirements must now be properly aligned with the IBC and NBCC 2005.

A small number of jurisdictions still enforce building codes that predate IBC/NBCC 2005. To ensure complete coverage of all existing building codes, Section 8.4 provides a methodology to ensure elevator design seismic force levels meet either

(a) IBC and NBCC 2005 requirements
(b) traditional seismic zone requirements
(c) requirements of building codes preceding IBC and NBCC 2005, where seismic force levels are based on A_v or Z_v

Requirement 8.4(a) dictates whether seismic design is required based on the enforcing building code requirements. Requirement 8.4(b) specifies the appropriate seismic force level required for design, based on the enforcing building code requirements.

1-3 ELEVATOR DESIGN IMPACT WITH IBC/NBCC

A comparison of the A17.1/B44 and IBC/NBCC (2005 and later editions) seismic requirements was conducted to determine how elevator design will be impacted with the adoption of IBC/NBCC seismic requirements.

For equivalent-sized components, horizontal force levels as specified by each code were compared. From derived force levels, geographic areas that might be impacted with force levels above current A17.1/B44 seismic zone force levels were noted. Since IBC/NBCC force levels vary with component height in the building, force level comparisons throughout the building height were also conducted.

Horizontal seismic force levels as dictated in A17.1/B44, IBC/ASCE 7, and NBCC are specified in 1-3.1 below.

1-3.1 Horizontal Seismic Force Levels

1-3.1.1 A17.1/B44. For seismic zone 3 areas, A17.1/B44 requires elevator components to withstand the force required to produce an acceleration of ½ gravity or gravity, depending on the component being described.

For seismic zone 3

F_p = horizontal seismic force level (Allowable Stress Design) = $0.5W_p$ or $0.25W_p$

1-3.1.2 IBC/ASCE 7

F_p = horizontal seismic design force (Strength Design)

$$= \frac{0.4a_p S_{DS} W_p}{\left(\dfrac{R_p}{I_p}\right)}\left[1+2\left(\frac{z}{h}\right)\right]$$

NOTE:
As reproduced from
Equation 1621.4
Excerpted from the 2000 International Building Code, Copyright 2000.
Washington, D.C.: International Code Council.
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www.ICCSAFE.org

with F_p not required to be taken as greater than

$$F_p = 1.6S_{DS}I_pW_p$$

and F_p shall not be taken as less than

$$F_p = 0.3S_{DS}I_pW_p$$

where

a_p = component amplification factor = 1 for elevators and escalators (reference Table 1621.3 of IBC 2000 and Table 13.6-1 of ASCE 7-10)

h = average roof height of structure with respect to the defined building base, provided by the building structural engineer

I_p = component importance factor = 1.00 or 1.50

R_p = component response modification factor = 2.5 for elevators and escalators (reference Table 1621.3 of IBC 2000 and Table 13.6-1 of ASCE 7-10)

S_{DS} = design spectral response acceleration (short period, 5% damped) = $\left(\frac{2}{3}\right)(F_a)(S_s)$ [reference Table 1613.5.3(1) of IBC 2006 and Table 11.4-1 of ASCE 7-10]

- F_a = coefficient based on site class for building
- S_s = seismic map value (contour lines) = the mapped maximum considered earthquake spectral response acceleration parameter at short periods

W_p = component operating weight (now defined in A17.1/B44, 8.4.15)

z = height in structure of point of attachment of component with respect to the defined building base provided by the building structural engineer. For items at or below the base, z shall be taken as zero. The value of z/h need not exceed 1.0.

NOTE: S_{DS}, I_p, building base, and h to be provided by the building structural engineer (see Fig. 1-3.1.2-1).

1-3.1.3 NBCC 2005 and Later Editions

F_p = horizontal seismic force (Strength Design) = $0.3F_aS_a(0.2)I_ES_pW_p$

NOTE: As reproduced from NBCC 2010, Division B, Article 4.1.8.18, published by the National Research Council of Canada (NRC).

where

F_a = acceleration-based site coefficient, defined in NBCC 2010, Table 4.1.8.4.B

F_p = horizontal force applied through center of mass of the component (NBCC 2010 refers to its horizontal seismic force as V_p. The term F_p has been adopted by A17.1/B44 for consistency with IBC/ASCE 7.)

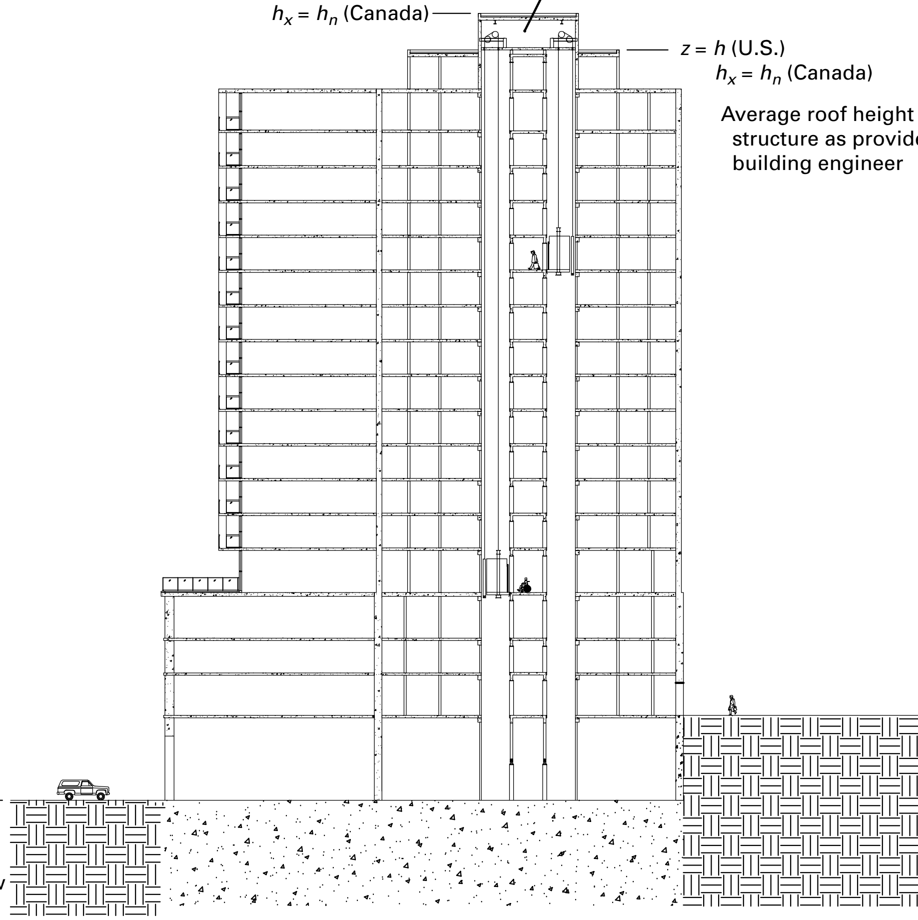
I_E = importance factor for the building, defined in NBCC 2010, Article 4.1.8.5

$S_a(0.2)$ = 5% damped spectral response acceleration value, expressed as a ratio to gravitational acceleration, for a period of 0.2 s, defined in NBCC 2010, 4.1.8.4(1)

S_p = $C_pA_rA_x/R_p$ (where S_p may range between 0.7 and 4.0) with

- A_r = component force amplification factor from NBCC 2010, Table 4.1.8.18
- A_x = height factor $(1 + 2h_x/h_n)$ with
  - h_n = average roof height of structure with respect to the base, provided by the building structural engineer. The value of h_x/h_n need not exceed 1.0.
  - h_x = height in structure of point of attachment of component with respect to the defined building

Fig. 1-3.1.2-1 Building Base Designation and Associated Variables



base provided by the building structural engineer. For items at or below the base, z shall be taken as zero.

C_p = component factor as listed in NBCC 2010, Table 4.1.8.18

R_p = component response modification factor from NBCC 2010, Table 4.1.8.18

W_p = component operating weight (now defined in A17.1/B44, 8.4.15)

NOTE: F_a, I_E, building base, and h_n to be provided by the building structural engineer (see Fig. 1-3.1.2-1).

A17.1/B44 and older building code requirements are based on Allowable Stress Design (ASD). As noted in the AISC Manual of Steel Construction: Allowable Stress Design (AISC 335-89), a one-third increase in allowable stress was permitted for earthquake loads. Beams, components, and fastenings would be sized for earthquake loads with this increased stress.

IBC/ASCE 7 and NBCC 2005 (and later editions) provide requirements, including the horizontal seismic force levels above and load combination equations, in Strength Design (SD) [also known as Load and Resistance Factor Design (LRFD) in the U.S. or Limit States Design (LSD) in Canada]. For reference documents such as A17.1 that use ASD, IBC/ASCE 7 permits its earthquake loads to be multiplied by a factor of 0.7, provided the reference document (A17.1/B44) considers load combinations of dead, live, operating, and earthquake loads in addition to its other loads (reference ASCE 7-10, requirement 13.1.7). Acceptable factored load combinations for ASD are also offered by IBC in Section 1605.3 and ASCE 7-10 in Section 2.4. Requirement 13.1.7 and the factored load combinations effectively include the one-third stress increase (allowed under ASD) within the loading requirements.

Unlike the IBC, NBCC 2005 (and later editions) makes no provision for reference documents still using ASD.[1] NBCC 2005 (and later editions) provides its requirements,

[1] In recent years, SD (LRFD/LSD) has become more prevalent in use amongst many industries. SD methods have been largely accepted in Canada, but are still not wholly adopted in the U.S.

including horizontal seismic force and load combination equations, solely in SD/LSD. SD and ASD are separate design methodologies and cannot be used interchangeably. In order to incorporate the new NBCC requirements within the ASD-based A17.1/B44, the earthquake loads and load combination equations are factored as allowed by ASCE 7-10, requirement 13.1.7. The one-third stress increase allowance is also removed.

In generic terms of stress equality

A17.1/B44	IBC/ASCE 7	NBCC 2010
$1.33\sigma = \dfrac{F_{p\_A17.1}}{A_{A17.1}}$	$\sigma = \dfrac{0.7F_{p\_IBC}}{A_{IBC}}$	$\sigma = \dfrac{0.7F_{p\_NBCC}}{A_{NBCC}}$

The impact on elevator design may be determined by comparing force levels needed to generate similarly sized components under each code.

$$A_{A17.1} = A_{IBC} \text{ and } A_{A17.1} = A_{NBCC}$$

Substituting from the generic stress equalities yields

$$0.75F_{p\_A17.1} = 0.7F_{p\_IBC} \text{ and } 0.75F_{p\_A17.1} = 0.7F_{p\_NBCC}$$

or

$$F_{p\_A17.1} \approx F_{p\_IBC} \text{ and } F_{p\_A17.1} \approx F_{p\_NBCC}$$

The resulting equations indicate that to obtain similarly sized components, the IBC/NBCC SD-based seismic force would need to equal the A17.1/B44 ASD-based seismic force.

The IBC/NBCC seismic force equations can be written in terms of their geographically defined spectral response acceleration values, S_s and $S_a(0.2)$, respectively. By equating these formulas to a known A17.1/B44 seismic zone level force, the value of S_s and $S_a(0.2)$ that would equal the A17.1/B44 force can be determined. Any S_s or $S_a(0.2)$ that exceeds that value on the IBC contour maps or the NBCC 2005 seismic data tables would indicate locations where larger force levels and more robust elevator designs would be required.

The largest expected difference between A17.1/B44 and IBC/NBCC force levels was for guide rails/rail brackets at the upper portion of the building, due to the introduction of the amplification factor in IBC/NBCC force equations. Because of their dependence on component height placement in the building, IBC/NBCC forces at the top of the building would be up to 1.6 times greater than at the building base. When compared to height-invariant A17.1/B44 rail bracket forces, the force levels required by IBC/NBCC at the top of the building were expected to generate design changes for a large portion of the U.S. and Canada. The comparison of IBC/NBCC force levels and A17.1/B44 seismic zone 3 guide rail force levels is detailed in Table 1-3.1.3-1.

The comparison in Table 1-3.1.3-1 of A17.1/B44 zone 3 and IBC/NBCC 2010 forces is taken at the top of a building. Due to the height variable in the IBC (z) and NBCC (h_x) seismic force equation, IBC and NBCC forces in the center and lower portions of the building will be reduced. Therefore, the impact of changing to IBC and NBCC forces should be greatly reduced in the mid to lower half of buildings.

Table 1-3.1.3-2 indicates the impact of the introduction of IBC/NBCC 2005 (and later editions) seismic force levels for a building in the U.S. and Canada. The chart indicates that for the upper half of a building, in areas where A17.1/B44 zone 3 requires only 0.5g seismic forces (such as rail brackets), seismic forces will increase for some portions of the country. Other locations within the building will see little to no increase above A17.1/B44 seismic zone force levels.

1-4 USING IBC/ASCE 7 FOR ELEVATOR SEISMIC DESIGN (QUICK REFERENCE)

By obtaining the following IBC parameters, the need for elevator seismic design and required seismic force levels can be determined:

– Seismic Design Category (SDC)
– I_p
– S_{DS}
– location of the base of the building
– average roof height of the building

For quick reference, Table 1-4-1 correlates three IBC parameters (at the worst-case height ratio) and the equivalent seismic zone that would meet or exceed all necessary IBC force levels required.

1-5 SUMMARY

While at times requiring slightly increased seismic force levels in the upper half of the building, particularly in the area of rail bracket selection and spacing, adoption of the IBC/NBCC seismic force levels might result in less stringent seismic forces in the lower half of the building than are currently required by A17.1. Use of IBC contour maps and the NBCC seismic data chart may introduce seismic requirements in areas that had been traditionally nonseismic. Regardless of the changes these force levels will dictate, the benefits of clarity in the code and use of the latest and most accurate information in seismic force protection are warranted.

In addition to the code proposals and this Guide, a sample calculation section has been developed to further explain the proper force selection for all building codes and the proper use of the new IBC/NBCC seismic forces with existing A17.1, Section 8.4 requirements.

1-6 EXPLANATION OF TERMS

ASCE 7 = American Society of Civil Engineers Standard for Minimum Design Loads for Buildings and Other Structures. The

Table 1-3.1.3-1 Geographic Impact Comparison: IBC/NBCC Versus A17.1/B44 Seismic Zone 3 (Guide Rail)

Step	For A17.1, Seismic Zone 3 or Greater	For IBC/ASCE 7	For NBCC 2005 (and Later Editions)
1. Identify force formulas as given by code.	$F_p = 0.5W_p$	NOTE: An additional increase may be required for anchorage in concrete/masonry (reference ASCE 7, 13.4.2). $F_p = \left(\dfrac{0.4a_pS_{DS}}{\left(\dfrac{R_p}{I_p}\right)} \left[1+2\left(\dfrac{z}{h}\right)\right] W_p \right)$	$F_p = 0.3F_aS_a(0.2)I_ES_pW_p$ or (including all variables) $F_p = \left[\dfrac{0.3F_aS_a(0.2)I_EC_pA_r\left(1+2\dfrac{h_x}{h_n}\right)}{R_p} W_p \right]$
2. Simplify force formula where possible.	...	Values of a_p and R_p are standardized for elevators as listed in the equation description above. Inserting their values, F_p is simplified to $F_p = 0.16S_{DS}I_p[1 + 2(z/h)]W_p$	Value of $C_p = 1$ for any nonstructural component (rigid components or machinery) $A_r = 1$ for rigid components and machinery rigidly connected $A_r = 2.5$ for machinery flexibly connected $R_p = 2.5$ for rigid components and machinery flexibly connected $R_p = 1.25$ for machinery rigidly connected Inserting these values gives for rigid components with ductile material $F_p = 0.12F_aS_a(0.2)I_E\left(1+2\dfrac{h_x}{h_n}\right)W_p$ for machinery with rigid connections $F_p = 0.24F_aS_a(0.2)I_E\left(1+2\dfrac{h_x}{h_n}\right)W_p$ for machinery with flexible connections $F_p = 0.3F_aS_a(0.2)I_E\left(1+2\dfrac{h_x}{h_n}\right)W_p$
3. Look at worst case (top of building).	$F_p = 0.5W_p$ (listed for comparison reference only)	The highest values of F_p will occur at the top of the building, where $z = h$. Incorporating this condition simplifies F_p to $F_p = 0.48S_{DS}I_pW_p$	The highest values of F_p will occur at the top of the building, where $h_x = h_n$. Incorporating this condition simplifies F_p to for rigid components with ductile material $F_p = 0.36F_a S_a(0.2)\ I_EW_p$ for machinery with rigid connections $F_p = 0.72F_a S_a\ (0.2)\ I_E\ W_p$ for machinery with flexible connections $F_p = 0.9F_a S_a\ (0.2)\ I_E\ W_p$

Table 1-3.1.3-1 Geographic Impact Comparison: IBC/NBCC Versus A17.1/B44 Seismic Zone 3 (Guide Rail) (Cont'd)

Step	For A17.1, Seismic Zone 3 or Greater	For IBC/ASCE 7	For NBCC 2005 (and Later Editions)
4. Look at importance factors.	$F_p = 0.5W_p$ (listed for comparison reasons only)	I_p has two possible values $I_p = 1.0$ or $I_p = 1.5$ For buildings with $I_p = 1$ $F_p = 0.48S_{DS}W_p$ For buildings with $I_p = 1.5$ $F_p = 0.72S_{DS}W_p$	I_E has four possible values $I_E = 0.8, 1.0, 1.3,$ or 1.5 For comparison with IBC, only $I_E = 1.0$ and $I_E = 1.5$ will be detailed. For buildings with $I_E = 1$ for rigid components with ductile material $F_p = 0.36F_aS_a(0.2)W_p$ for machinery with rigid connections $F_p = 0.72F_aS_a(0.2)W_p$ for machinery with flexible connections $F_p = 0.9F_aS_a(0.2)W_p$ For buildings with $I_E = 1.5$ for rigid components with ductile material $F_p = 0.54F_aS_a(0.2)W_p$ for machinery with rigid connections $F_p = 1.08F_aS_a(0.2)W_p$ for machinery with flexible connections $F_p = 1.35F_aS_a(0.2)W_p$
5. Write IBC force levels in terms of spectral response acceleration values, S_s and $S_a(0.2)$.	$F_p = 0.5W_p$ (shown for reference only)	S_{DS} is related to the USGS map contour lines by $S_{DS} = \frac{2}{3}(F_a)(S_S)$ where F_a = site coefficient listed in Table 1613.5.3(1) S_S = contour lines on USGS 0.2-sec spectral response maps Inserting new value for S_{DS} yields for $I_p = 1$ $F_p = 0.32F_aS_SW_p$	F_a and $S_a(0.2)$ are referenced to NBCC 2010 as F_a = short period site coefficient listed in NBCC 2010, Table 4.1.8.4.B $S_a(0.2)$ = short period spectral response acceleration values for specific locations as listed in Appendix C of NBCC 2010 (Volume 2) For $I_E = 1$ for rigid components with ductile material $F_p = 0.36F_aS_a(0.2)W_p$ for machinery with rigid connections $F_p = 0.72F_aS_a(0.2)W_p$ for machinery with flexible connections $F_p = 0.9F_aS_a(0.2)W_p$

Table 1-3.1.3-1 Geographic Impact Comparison: IBC/NBCC Versus A17.1/B44 Seismic Zone 3 (Guide Rail) (Cont'd)

Step	For A17.1, Seismic Zone 3 or Greater	For IBC/ASCE 7	For NBCC 2005 (and Later Editions)
	$F_p = 0.5W_p$ (shown for reference only)	for $I_p = 1.5$ $F_p = 0.48F_aS_SW_p$	For buildings with $I_E = 1.5$ for rigid components with ductile material $F_p = 0.54F_a S_a (0.2)W_p$ for machinery with rigid connections $F_p = 1.08F_a S_a (0.2)W_p$ for machinery with flexible connections $F_p = 1.35F_a S_a (0.2)W_p$
6. Equate A17.1 force level with building code force levels.	$F_p = 0.5W_p$ (shown for reference only)	Per eq. (1), A17.1 and IBC will size similar components when $F_{p\_A17.1} = F_{p\_IBC}$ Setting the two force levels equal and eliminating W_p from each side yields for $I_p = 1$ $1.56 = F_aS_S$ for $I_p = 1.5$ $1.04 = F_aS_S$	Per eq. (2), A17.1 and NBCC 2010 will size similar components when $F_{p\_A17.1} = F_{p\_NBCC}$ Inserting the values for $F_{p\_A17.1}$ and F_{p_NBCC} above and eliminating W_p from each side of the equation yields for $I_E = 1$ for rigid components with ductile material $1.39 = F_aS_a(0.2)$ for machinery with rigid connections $0.69 = F_aS_a(0.2)$ for machinery with flexible connections $0.56 = F_aS_a(0.2)$ for $I_E = 1.5$ for rigid components with ductile material $0.92 = F_aS_a(0.2)$ for machinery with rigid connections $0.46 = F_aS_a(0.2)$ for machinery with flexible connections $0.37 = F_aS_a(0.2)$

Table 1-3.1.3-1 Geographic Impact Comparison: IBC/NBCC Versus A17.1/B44 Seismic Zone 3 (Guide Rail) (Cont'd)

Step	For A17.1, Seismic Zone 3 or Greater	For IBC/ASCE 7	For NBCC 2005 (and Later Editions)
7. Solve for S_s and $S_a(0.2)$ and determine geographic areas that will see force levels over current A17.1 seismic zone 3.	...	By solving for S_S, it can be determined what areas in the U.S. will see increased seismic force levels when using the IBC/ASCE 7 seismic requirements. Per IBC Table 1613.5.3(1), for values of S_S over 1.25, F_a becomes a maximum of 1, giving for $I_p = 1$ $1.56 = S_S$ IBC force levels will be greater than A17.1 force levels where the mapped spectral response acceleration is greater than 156% g. Reviewing IBC 2006, Figure 1613.5(1), Maximum Considered Earthquake Ground Motion for the Conterminous United States of 0.2 sec Spectral Response Acceleration (5% of Critical Dampening) Site Class B, indicates areas near fault lines and the area near Charleston, S.C. will see increased force levels with the use of IBC.	By solving for $S_a(0.2)$, it can be determined what areas in Canada will see increased seismic force levels when using the NBCC seismic requirements. Per NBCC 2010, Table 4.1.8.4.B, F_a has a maximum value of 1 for Site Classes A through C. Using this plus the appropriate maximum F_a values for Site Classes D and E yields a minimum value of $S_a(0.2)$. For each of the three possible component elements considered for $I_E = 1$ for rigid components with ductile material $1.39 = S_a(0.2)$ for machinery with rigid connections $0.69 = S_a(0.2)$ for Site Classes A through C $0.58 = S_a(0.2)$ for Site Class D $0.49 = S_a(0.2)$ for Site Class E for machinery with flexible connections $0.56 = S_a(0.2)$ for Site Classes A through C $0.47 = S_a(0.2)$ for Site Class D $0.27 = S_a(0.2)$ for Site Class E NBCC force levels for Site Classes A through C will be greater than A17.1 force levels when $S_a(0.2)$ values listed in NBCC 2010, Table C-2 of Appendix C exceed either 1.39, 0.69, or 0.56 as noted above. For rigid components, only La-Malbaie, north of Quebec and bordering the St. Lawrence River, would see an increased seismic force level with the use of IBC. All other values of $S_a(0.2)$ in Table C-2 are less than 1.39. For machinery components with rigid connections, additional locations north of Quebec as well as Montreal Region in Quebec and Victoria and Vancouver Regions in British Columbia would see increased force levels over A17.1. For machinery components with flexible connections, much of Quebec province, increased locations in British Columbia, as well as St. Stephen, Ontario, and certain locations in Ontario, St. Stephen, New Brunswick, and Destruction Bay and Snag, Yukon, would see increased for force levels.

Table 1-3.1.3-1 Geographic Impact Comparison: IBC/NBCC Versus A17.1/B44 Seismic Zone 3 (Guide Rail) (Cont'd)

Step	For A17.1, Seismic Zone 3 or Greater	For IBC/ASCE 7	For NBCC 2005 (and Later Editions)
		for $I_p = 1.5$ Again, per IBC Table 1613.5.3(1), for values of S_S over 1.25, F_a becomes a maximum of 1, giving $1.04 = S_S$ For this case, areas further outside of fault lines, much of California and the Charleston, S.C. area will see increased forces with the use of IBC.	for $I_E = 1.5$ Again, using NBC-2010, Table 4.1.8.4.B for maximum F_a values, minimum values of $S_a(0.2)$ are found for rigid components with ductile material $0.92 = S_a(0.2)$ for Site Classes A through C and E $0.84 = S_a(0.2)$ for Site Class D for machinery with rigid connections $0.46 = S_a(0.2)$ for Site Classes A through C and E $0.38 = S_a(0.2)$ for Site Class D $0.22 = S_a(0.2)$ for Site Class E for machinery with flexible connections $0.37 = S_a(0.2)$ for Site Classes A through C $0.28 = S_a(0.2)$ for Site Class D $0.18 = S_a(0.2)$ for Site Class E With $I_E = 1.5$, NBC force levels (for Site Classes A through C) for rigid components would be greater for an increased number of locations in Quebec. Western British Columbia (near Victoria and Vancouver regions) would also see a number of locations with increased forces. For machinery with rigid and flexible connections, much of Quebec and additional locations in British Columbia would see increased forces. Locations in Ontario (surrounding Ottawa), Yukon, and a few select locations in Northwest Territories and Nunavut would now also be impacted.

Table 1-3.1.3-2 Impact of IBC/NBCC Forces on Elevator Components in U.S. and Canada (Comparison of IBC/NBCC Forces to A17.1/B44 Seismic Zone 3)

		IBC [Note (1)]		NBCC 2005 (and Later Editions) [Note (1)]											
		Maximum IBC S_s Contour That Does Not Exceed A17.1/B44 Zone 3 Force		Maximum $S_a(0.2)$ Value (Site Classes A Through C) That Does Not Exceed A17.1/B44 Seismic Zone 3 Force											
Building Location/ Component Description	A17.1/ B44 Zone 3 Force Level	Component Importance Factor, $I_p = 1$	Component Importance Factor, $I_p = 1.5$	Building Importance Factor, $I_E = 0.8$			Building Importance Factor, $I_E = 1$			Building Importance Factor, $I_E = 1.3$			Building Importance Factor, $I_E = 1.5$		
				Note (2)	Note (3)	Note (4)	Note (2)	Note (3)	Note (4)	Note (2)	Note (3)	Note (4)	Note (2)	Note (3)	Note (4)
Top of building (machine beams)	W_p	312 [Note (5)]	208 [Note (6)]	3.47 [Note (6)]	1.74 [Note (6)]	1.39 [Note (6)]	2.78 [Note (6)]	1.39 [Note (6)]	1.11 [Note (7)]	2.14 [Note (6)]	1.07 [Note (7)]	0.85	1.85 [Note (6)]	0.92	0.74
Top of building (rail brackets)	$0.5W_p$	156 [Note (7)]	104 [Notes (7) and (8)]	1.74 [Note (6)]	0.87	0.69	1.39 [Note (6)]	0.69	0.56	1.07 [Note (7)]	0.53	0.43	0.92	0.46	0.37
Building midheight (rail brackets)	$0.5W_p$	234 [Note (6)]	156 [Note (7)]	2.60 [Note (6)]	1.30 [Note (7)]	1.04 [Note (7)]	2.08 [Note (6)]	1.04 [Note (7)]	0.83	1.60 [Note (6)]	0.80	0.64	1.39 [Note (6)]	0.69	0.56
Building ground level (rail brackets) [Note (9)]	$0.5W_p$	250 [Note (6)]	167 [Note (7)]	2.98 [Note (6)]	2.60 [Note (6)]	2.08 [Note (6)]	2.38 [Note (6)]	2.08 [Note (6)]	1.67 [Note (6)]	1.83 [Note (6)]	1.60 [Note (6)]	1.28 [Note (7)]	1.59 [Note (6)]	1.39 [Note (6)]	1.11 [Note (7)]

GENERAL NOTE: This chart equates A17.1 seismic zone 3 forces with the equivalent seismic contour band, S_s, or value of $S_a(0.2)$ that will generate the same force. Contour bands, S_s, or $S_a(0.2)$ values above that shown will generate a greater force than the A17.1 seismic zone 3 force and may require more robust elevator component designs.

For example, at the top of the building, for machine components, areas in the U.S. that have a contour band over 312 (for $I_p = 1.0$) or 208 (for $I_p = 1.5$) will see an increased force when using IBC. Impact across the U.S. will be fairly minor as contour bands are mostly below these levels. For rail brackets at the top of the building, areas of the U.S. with contour bands over 156 or 104 will see higher forces when using IBC. Therefore, IBC will have a larger impact across the country with these component sizings.

NOTES:
(1) For equal component sizing comparisons, force levels were compared as follows:
  (a) A17.1 and IBC: $F_{p\_A17.1} = F_{p\_IBC}$
  (b) A17.1 and NBCC: $F_{p\_A17.1} = F_{p\_NBCC}$
(2) The values shown are for rigid components with ductile material.
(3) The values shown are for machinery with rigid connections.
(4) The values shown are for machinery with flexible connections.
(5) For IBC, A17.1 forces will be greater except possibly fault lines.
(6) For IBC, A17.1 forces will be greater except for a small band near fault lines. For NBCC, A17.1 forces will be greater.
(7) For IBC, A17.1 forces will be greater except for high seismic areas such as Charleston, S.C. For NBCC, A17.1 forces will be greater except for possibly 13 locations in British Columbia and two in Quebec.
(8) Excludes Site Class D (NBCC values found for Site Classes A through C).
(9) F_p used to generate values may have been limited due to allowed IBC/NBCC minimums.

Table 1-4-1 IBC/ASCE 7 Seismic Parameters Correlation to A17.1 Zones

IBC (2000 and Later)/ASCE 7 (2002 and Later)			A17.1/B44
Seismic Design Category	I_p	S_{DS} [Notes (1) and (2)]	Seismic Zone
A or B	–	Not required	0, 1
C	1	Not required	0, 1
	1.5	$0 < S_{DS} \leq 0.496$	2
		$0.496 < S_{DS} \leq 0.993$	3 or greater
		> 0.993	Special analysis required
D or E or F	1	$0 < S_{DS} \leq 0.745$	2
		$0.745 < S_{DS} \leq 1.487$	3 or greater
		> 1.487	Special analysis required
	1.5	$0 < S_{DS} \leq 0.496$	2
		$0.496 < S_{DS} \leq 0.993$	3 or greater
		> 0.993	Special analysis required

NOTES:
(1) For equivalencies, IBC force values have been reduced by 0.7 to convert from SD to ASD (working stress).
(2) Assumed $(z/h) = 1$.

2002 (ASCE 7-02) and subsequent publications reference the latest USGS earthquake maps.

BOCA = Building Officials and Code Administrators International, Inc. The last publication of this code was in 1999. This building code referenced the older, seismic zone earthquake maps. BOCA is now a member of the International Code Council (ICC).

IBC = International Building Code. The 2000 and subsequent publications reference the latest USGS earthquake maps.

ICBO = International Conference of Building Officials. Responsible for the publication of the Uniform Building Code, a national building code. Now a member of the International Code Council (ICC).

NBCC = National Building Code of Canada. The 2005 and later editions use a seismic force equation similar to that of IBC.

NEHRP = National Earthquake Hazards Reduction Program. A U.S. government program. Recommendations from NEHRP are often incorporated into building codes and standards.

SBC = The Standard (Southern) Building Code (Standard Building Code). Previously used in many areas in southeastern U.S. Last published in 1999. This building code referenced the older, seismic zone earthquake maps.

SBCCI = Southern Building Code Conference International. Organization responsible for the development and maintenance of the model building code known as the Standard (Southern) Building Code. Now a member of the International Code Council (ICC).

UBC = Uniform Building Code. A national building code (also referred to as ICBO) published by the International Conference of Building Officials. 1997 was the last published edition. This building code referenced the older, seismic zone earthquake maps.

USGS = United States Geological Survey. Responsible for the most recent earthquake maps currently being referenced by the latest building codes and standards.

Part 2
Derivations

Detailed derivations of selected equations included in Section 8.4 are presented in Part 2 to provide additional background.

2-1 FIGURES 8.4.8.2-1 THROUGH 8.4.8.2-7

To use Figs. 8.4.8.2-1 through 8.4.8.2-7 without generating new rail load versus bracket span curves, a relationship between W and F_p was derived that allows use of the existing seismic zone 3 curves. This was done in two ways: by comparing allowable stress and by going through the original bending stress calculations.

(a) Comparing allowable stresses (generic stress formula)

$$\sigma = \frac{F}{A}$$

where
A = area (for axial or shear stress)
= Z/L (for bending stress)
F = force
L = bending moment length
Z = elastic section modulus

Current Zone 3 Stress Analog	Unfactored IBC/NBCC Stress Analog
$0.88F_y = \frac{0.5W}{A}$	$0.6F_y = \frac{F_p}{A}$
$F_y = \frac{25}{44} \times \frac{W}{A}$	$F_y = \frac{5}{3} \times \frac{F_p}{A}$

(b) The yield stress is the same for both cases. Therefore, we can equate the F_y formulas

$$\frac{25}{44} \times \frac{W}{A} = \frac{5}{3} \times \frac{F_p}{A}$$

(c) For a given rail size, the equipment size (i.e., bracket span) is the same, thus A drops out.

$$\frac{25}{44} W = \frac{5}{3} F_p$$

$$W = \frac{44}{15} F_p$$

$$W = 2.93F_p$$

(d) IBC/ASCE 7 allows a 0.7 load factor to convert strength level to working stress on earthquake loads (see ASCE 7-10, 13.1.7).

For calculating deflections

$$W = 2.93F_p$$

For calculating stresses

$$W = (2.93)(0.7)F_p$$

2-2 REQUIREMENT 8.4.8.9

For each equation, the seismic force has been applied at the center of gravity (CG) of the car or counterweight. The CG is taken to be one-third above the lower guide that contacts the rail. For a seismic force introduced perpendicular to the x-x axis of the rail (see Fig. 8.4.8.9), one upper and lower guide will come in contact with the rail. The resultant forces on the guide can be found by a force and moment equilibrium equation. For comparison, the guide forces for a seismic zone 3 force and an IBC/NBCC force will be derived (see Fig. 2-2-1).

seismic zone 3 force = force to generate a ½ gravity acceleration

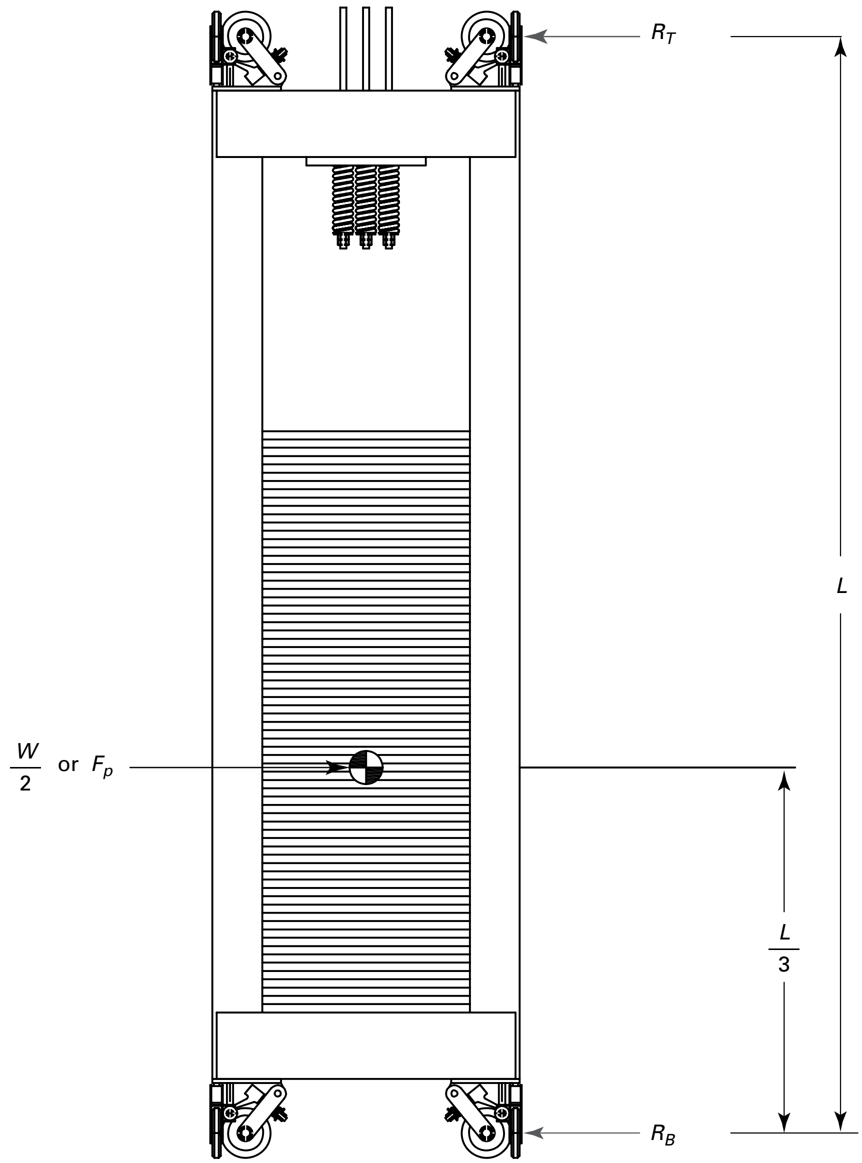
$$\text{seismic zone 3 force} = ma = \left(\frac{W_p}{g}\right)\left(\frac{g}{2}\right) = \frac{W_p}{2}$$

$$\text{IBC/NBCC force} = F_p$$

where W_p = weight of counterweight or car plus 40% capacity.

The following comparison equations show F_p and W_p (seismic zone 3) are related by $W_p = 2F_p$. In all subsequent force equations, the F_p equations could be found by substituting $2F_p$ for W.

Fig. 2-2-1 Sample Counterweight Force Diagram



Seismic Zone 3	IBC/NBCC
$\Sigma F = 0 = -R_B + \frac{W_p}{2} - R_T$	$\Sigma F = 0 = -R_B + F_p - R_T$
$R_B = \frac{W_p}{2} - R_T$	$R_B = F_p - R_T$
$\Sigma M_B = 0 = -\left(\frac{W_p}{2}\right)\frac{L}{3} + R_T(L)$	$\Sigma M_B = 0 = -F_p\frac{L}{3} + R_T(L)$
$R_T = \left(\frac{W_p}{6}\right)$	$R_T = \frac{F_p}{3}$
$R_B = \left(\frac{W_p}{3}\right)$	$R_B = \frac{2F_p}{3}$

The counterweight force exerted at the lower guide is the greater force.

For the case where the vertical distance between upper and lower position restraints, L, is greater than or equal to the distance between adjacent guide brackets, ℓ, the guides are acting separately on each rail span.

Therefore the maximum force perpendicular to the x-x axis of the guide rail will be the lower guide force.

For the condition where the car/counterweight length is less than the guide rail length, $L < \ell$ [see 8.4.8.9.1(b)], both upper and lower guides will be in contact with a supported rail span. To look at the worst-case scenario, the lower guide is positioned at the lower rail support point. With this setup, the rail/support takes the full load of the highest car/counterweight guide force plus the force due to the bending load of the smaller guide force. As with the example above, W_p (seismic zone 3) and F_p are related by $W = 2F_p$.

2-3 REQUIREMENT 8.4.12

These equations are used for determining rail support spacing. Derivation of the formula (in imperial units) is shown below for 8.4.12.1. The current formula and new F_p formula are both derived for comparison.

For each equation, the seismic force has been applied at the center of gravity of the car or counterweight. The CG is taken to be one-third above the lower guiding member that contacts the rail. Previous derivations show that the lower guide shoe force will be $\frac{W_p}{3}$ or $\frac{2F_p}{3}$ (see rationale for 8.4.8.9).

Using continuous beam analysis for a 2 span beam with guide shoe forces acting in the center of each span (worst-case) gives reaction force, R_B, of $\frac{23}{64}P$. Substituting the previously derived lower guide forces for P gives

A17.1/B44	IBC
$R_B = \frac{23}{64}P = \frac{23}{64}\left(\frac{W_p}{3}\right) = \frac{23W_p}{192}$	$R_B = \frac{23}{64}P = \frac{23}{64}\left(\frac{2F_p}{3}\right) = \frac{46F_p}{192}$

Maximum moment will occur at point "a" on rail

$M_{max.} = M_a = \frac{1}{2}R_B\ell = \frac{23W_p\ell}{384}$	$M_{max.} = M_a = \frac{1}{2}R_B\ell = \frac{46W_p\ell}{384}$

The maximum bending stress equation is

$$\sigma_{allow} = \frac{M_{max.}c}{I} = \frac{M_{max.}}{Z}$$

where

$M_{max.}$ = maximum bending moment
Z = elastic section modulus for the beam
σ_{allow} = maximum bending stress

Assuming beam (or rail) of A36 steel

maximum bending stress = $\sigma_{allow} = 0.6F_y = 0.6(36{,}000) =$ 21,600 psi

For jurisdictions enforcing seismic zones, ASD and Section 8.4 allow a one-third stress increase for earthquake loads.

maximum bending stress (ASD) = $\sigma_{allow} = 1.33(0.66F_y) =$ 31,600 psi

For standards using ASD, IBC allows for a force reduction rather than a stress increase. NBCC does not allow for a force reduction or a stress increase. Therefore the maximum bending stress equations for A17.1 and IBC/NBCC become

A17.1	IBC/NBCC
$31{,}600 = \frac{M_{max.}}{Z}$	$21{,}600 = \frac{M_{max.}}{Z}$

Substituting the maximum bending moment derived above

$31{,}600 = \frac{23W_p\ell}{384Z}$	$21{,}600 = \frac{46F_p\ell}{384Z}$

Solving for W_p or F_p yields

$W_p = 527{,}583\frac{Z}{\ell}$	$F_p = 180{,}313\frac{Z}{\ell}$

The basic formula was adjusted by certain modification factors that were obtained as a result of extensive computer analysis.

Constant modification factors are as follows:

$$X_1 = 1.6$$

where X_1 accounts for the redistribution of forces due to the bending in counterweight frame upright member.

$$X_2 = 1.13$$

where X_2 accounts for the case where L is less than ℓ.

$$C = \text{Ratio}\,\frac{X_1}{X_2} = 1.41593$$

The final formulas were arrived at as follows:

(a) maximum weight of car or counterweight, W_0, with no intermediate tie brackets

$$W_0 = Cf_0W_p$$
$$W_0 = 1.36028W_p$$

where
f_0 = maximum moment occurring at 0.406ℓ
= 0.9607

(b) maximum weight of car or counterweight, W_1, with one intermediate tie bracket

$$W_1 = CV_1f_1W_p$$
$$W_1 = 1.80444W_p$$

where
f_1 = maximum moment occurring at 0.302ℓ
= 0.7420
V_1 = one intermediate tie bracket at $\frac{\ell}{2}$
= 1.7175

(c) maximum weight of car or counterweight, W_2, with two intermediate tie brackets

$$W_2 = CV_2f_2W_p$$
$$W_2 = 1.996832W_p$$

where
f_2 = maximum moment occurring at 0.458ℓ
= 0.9891
V_2 = two intermediate tie brackets equally spaced
= 1.425803

Substituting these formulas into basic formula yields

A17.1	IBC/NBCC
$W_p = 717{,}661\frac{Z}{\ell}$	$F_p = 245{,}276\frac{Z}{\ell}$

To use the same graphs as done for A17.1, the F_p equation is modified to

$$2.93F_p = 717{,}661\frac{Z}{\ell}$$

To convert F_p to ASD levels as used in A17.1, F_p shall be multiplied by a factor of 0.7. Reference ASCE 7-10, 13.1.7.

$$2.93(0.7F_p) = 717{,}661\frac{Z}{\ell}$$

2-4 REQUIREMENT 8.4.12.2.1(a) (ZONE > 3)

2-4.1 General

Derivation of the formula 8.4.12.2.1(a) is shown. The equation is derived using continuous beam theory with the guide shoe forces, F_1 and F_2, impacting at the center of the guide rail spans (see Fig. 2-4.1-1).

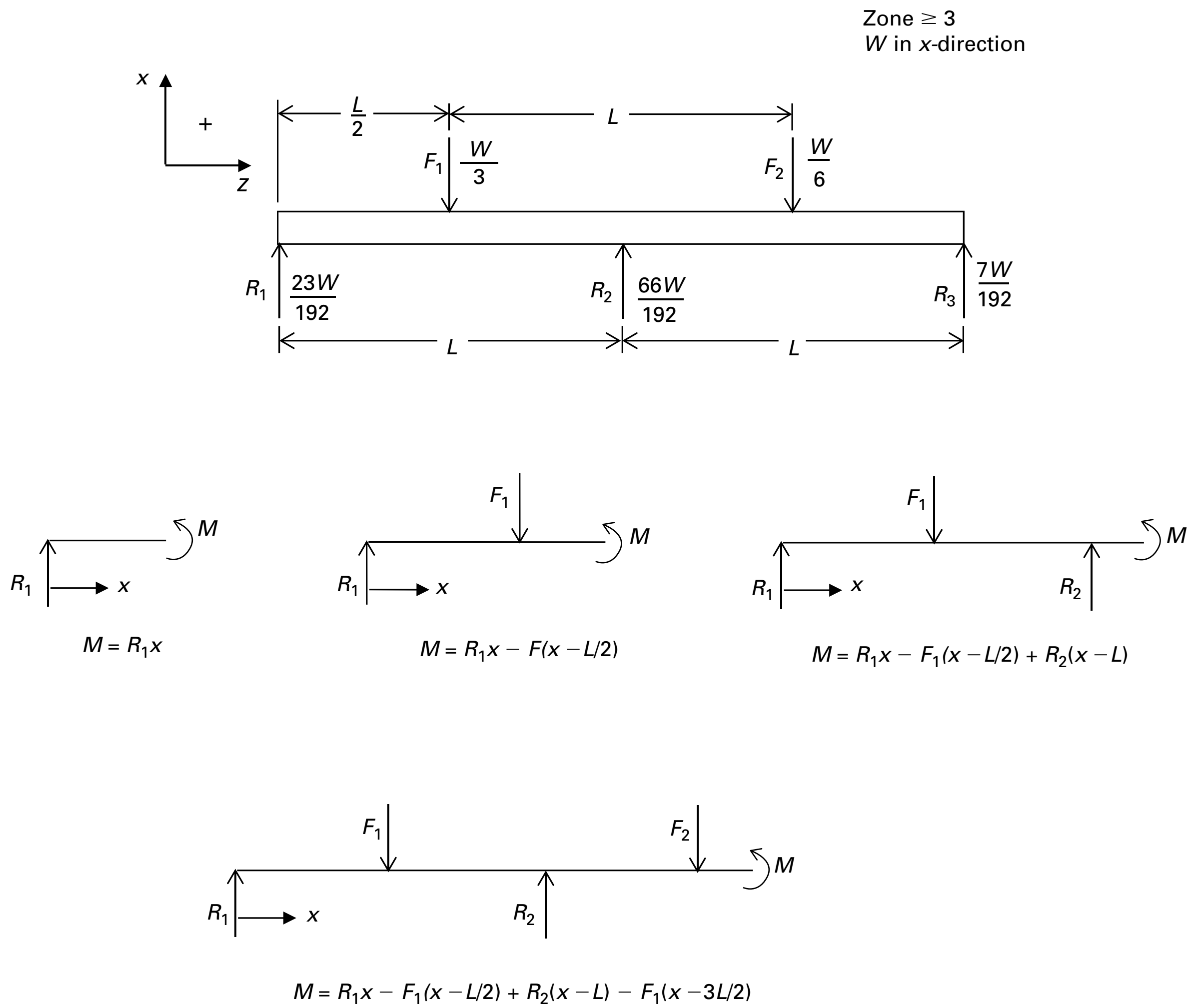
2-4.2

Solve for rotations and deflections by integration of the negative of the bending moment equations. "R" and "F" are used for simplicity at this point.

$$EIV_1'' = -R_1x$$
$$EIV_1' = -R_1\frac{x^2}{2} + c_1$$
$$EIV_1 = -R_1\frac{x^3}{6} + c_1x + c_2$$

$$EIV_2'' = -R_1x + F_1\left(x - \frac{L}{2}\right)$$
$$EIV_2' = -R_1\frac{x^2}{2} + F_1\left(\frac{x^2}{2} - \frac{Lx}{2}\right) + c_3$$
$$EIV_2 = -R_1\frac{x^3}{6} + F_1\left(\frac{x^3}{6} - \frac{Lx^2}{4}\right) + c_3x + c_4$$

Fig. 2-4.1-1 Rail Force Free Body Diagrams for A17.1/B44



$$EIV_3'' = -R_1 x + F_1(x - \frac{L}{2}) - R_2(x - L)$$

$$EIV_3' = -R_1 \frac{x^2}{2} + F_1(\frac{x^2}{2} - \frac{Lx}{2}) - R_2(\frac{x^2}{2} - Lx) + c_5$$

$$EIV_3 = -R_1 \frac{x^3}{6} + F_1(\frac{x^3}{6} - \frac{Lx^2}{4}) - R_2(\frac{x^3}{6} - \frac{Lx^2}{2}) + c_5 x + c_6$$

$$EIV_4'' = -R_1 x + F_1(x - \frac{L}{2}) - R_2(x - L) + F_2(x - \frac{3L}{2})$$

$$EIV_4' = -R_1 \frac{x^2}{2} + F_1(\frac{x^2}{2} - \frac{Lx}{2}) - R_2(\frac{x^2}{2} - Lx) + F_2(\frac{x^2}{2} - \frac{3Lx}{2}) + c_7$$

$$EIV_4 = -R_1 \frac{x^3}{6} + F_1(\frac{x^3}{6} - \frac{Lx^2}{4}) - R_2(\frac{x^3}{6} - \frac{Lx^2}{2}) + F_2(\frac{x^3}{6} - \frac{3Lx^2}{4}) + c_7 x + c_8$$

2-4.3 Boundary Conditions

$$V_1(0)=0 \qquad V_2'(L)=V_3'(L) \qquad V_3(\frac{3L}{2})=V_4(\frac{3L}{2})$$
$$V_1'\frac{L}{2}=V_2'\frac{L}{2} \qquad V_2(L)=V_3(L) \qquad V_4(2L)=0$$
$$V_1\ \frac{L}{2}=V_2\ \frac{L}{2} \qquad V_3'\left(\frac{3L}{2}\right)=V_4'\left(\frac{3L}{2}\right)$$

2-4.4 Integration Constants in Terms of *W*

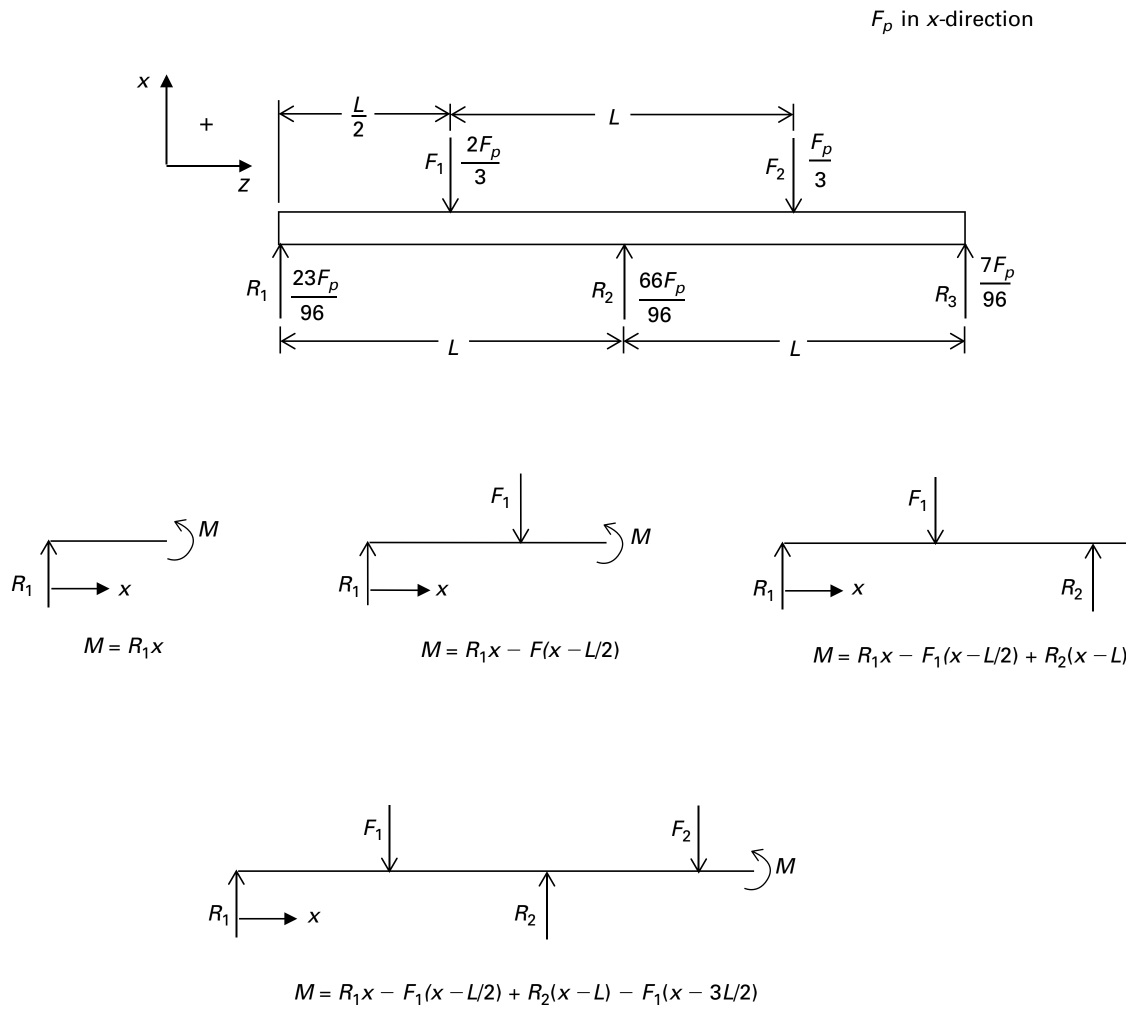
$$c_1=\frac{5L^2}{384}W \qquad c_5=\frac{15L^2}{384}W$$
$$c_2=0 \qquad c_6=\frac{29L^3}{576}W$$
$$c_3=\frac{7L^2}{128}W \qquad c_7=\frac{9L^2}{128}W$$
$$c_4=\frac{L^3}{144}W \qquad c_8=\frac{25L^3}{576}W$$

2-4.5 Solve for Deflection Equations

$$EIV_1(x)=\frac{-23W}{192}\left(\frac{x^3}{6}\right)+\frac{5L^2x}{384}W$$
$$=\frac{-23Wx^3}{1152}+\frac{15L^2x}{1152}W$$
$$=\frac{W}{1152}\left(-23x^3+15L^2x\right)$$
$$V_1(x)=\frac{W}{1152EI}\left(-23x^3+15L^2x\right)$$
$$EIV_2(x)=\frac{-23W}{192}\left(\frac{x^3}{6}\right)+\frac{W}{3}\left(\frac{x^3}{6}-\frac{Lx^2}{4}\right)+\frac{7L^2x}{128}W-\frac{L^3}{144}W$$
$$=\frac{-23x^3}{1152}W+\frac{64x^3}{1152}W-\frac{96Lx^2}{1152}W+\frac{63L^2x}{1152}W-\frac{8L^3}{1152}W$$
$$V_2(x)=\frac{W}{1152EI}\left(41x^3-96Lx^2+63L^2x-8L^3\right)$$
$$EIV_3(x)=\frac{-23W}{192}\left(\frac{x^3}{6}\right)+\frac{W}{3}\left(\frac{x^3}{6}-\frac{Lx^2}{4}\right)-\frac{66W}{192}\left(\frac{x^3}{6}-\frac{Lx^2}{2}\right)-\frac{15L^2x}{128}W+\frac{29L^3}{576}W$$
$$=\frac{-23x^3}{1152}W+\frac{64x^3}{1152}W-\frac{96Lx^2}{1152}W-\frac{66x^3}{1152}W+\frac{198Lx^2}{1152}W-\frac{135L^2x}{1152}W+\frac{58L^3}{1152}W$$
$$V_3(x)=\frac{W}{1152EI}\left(-25x^3+102Lx^2-135L^2x+58L^3\right)$$

$$EIV_4(x)=\frac{-23W}{192}\left(\frac{x^3}{6}\right)+\frac{W}{3}\left(\frac{x^3}{6}-\frac{Lx^2}{4}\right)-\frac{66W}{192}\left(\frac{x^3}{6}-\frac{Lx^2}{2}\right)+\frac{W}{6}\left(\frac{x^3}{6}-\frac{Lx^2}{2}\right)+\frac{9L^2x}{128}W-\frac{25L^3}{576}W$$
$$=\frac{-23x^3}{1152}W+\frac{64x^3}{1152}W-\frac{96Lx^2}{1152}W-\frac{66x^3}{1152}W+\frac{198Lx^2}{1152}W+\frac{32x^3}{1152}W-\frac{144Lx^2}{1152}W+\frac{81L^2x}{1152}W-\frac{50L^3}{1152}W$$
$$V_4(x)=\frac{W}{1152EI}\left(7x^3-42Lx^2+81L^2x-50L^3\right)$$

Fig. 2-5.1-1 Rail Force Free Body Diagrams for IBC/NBCC



2-4.6

A17.1, requirement 8.4.12.2.1 takes maximum deflection at $x = L/2$.

$$\Delta_{\max} = V_1\left(\frac{L}{2}\right) = \frac{W}{1152EI}\left[-23\left(\frac{L}{2}\right)^3 + 15L^2\left(\frac{L}{2}\right)\right]$$

$$= \frac{W}{1152EI}\left[\frac{-23L^3}{8} + \frac{15L^3}{2}\right]$$

$$= \frac{W}{1152EI}\left(\frac{37L^3}{8}\right)$$

$$\Delta_{\max} = \frac{37L^3}{9216EI}W \approx \frac{WL^3}{249EI}$$

$$I_x = \frac{WL^3}{249E\Delta}$$

This matches 8.4.12.2.1(a) for I_x for zone ≥ 3.

2-5 REQUIREMENT 8.4.12.2.1 (IBC/NBCC JURISDICTIONS)

2-5.1 General

Derivation of the formula 8.4.12.2.1(a) is shown. The equation is derived using continuous beam theory with the guide shoe forces, F_1 and F_2, impacting at the center of the guide rail spans (see Fig. 2-5.1-1).

2-5.2

Solve for rotations and deflections by integration of the negative of the bending moment equations. "R" and "F" are used for simplicity at this point.

$$EIV_1'' = -R_1 x$$
$$EIV_1' = -R_1 \frac{x^2}{2} + c_1$$
$$EIV_1 = -R_1 \frac{x^3}{6} + c_1 x + c_2$$

$$EIV_2'' = -R_1 x + F_1(x - \frac{L}{2})$$
$$EIV_2' = -R_1 \frac{x^2}{2} + F_1(\frac{x^2}{2} - \frac{Lx}{2}) + c_3$$
$$EIV_2 = -R_1 \frac{x^3}{6} + F_1(\frac{x^3}{6} - \frac{Lx^2}{4}) + c_3 x + c_4$$

$$EIV_3'' = -R_1 x + F_1(x - \frac{L}{2}) - R_2(x - L)$$
$$EIV_3' = -R_1 \frac{x^2}{2} + F_1(\frac{x^2}{2} - \frac{Lx}{2}) - R_2(\frac{x^2}{2} - Lx) + c_5$$
$$EIV_3 = -R_1 \frac{x^3}{6} + F_1(\frac{x^3}{6} - \frac{Lx^2}{4}) - R_2(\frac{x^3}{6} - \frac{Lx^2}{2}) + c_5 x + c_6$$

$$EIV_4'' = -R_1 x + F_1(x - \frac{L}{2}) - R_2(x - L) + F_2(x - \frac{3L}{2})$$
$$EIV_4' = -R_1 \frac{x^2}{2} + F_1(\frac{x^2}{2} - \frac{Lx}{2}) - R_2(\frac{x^2}{2} - Lx) + F_2(\frac{x^2}{2} - \frac{3Lx}{2}) + c_7$$
$$EIV_4 = -R_1 \frac{x^3}{6} + F_1(\frac{x^3}{6} - \frac{Lx^2}{4}) - R_2(\frac{x^3}{6} - \frac{Lx^2}{2}) + F_2(\frac{x^3}{6} - \frac{3Lx^2}{4}) + c_7 x + c_8$$

2-5.3 Boundary Conditions

$$V_1(0) = 0$$
$$V_1'\left(\frac{L}{2}\right) = V_2'\left(\frac{L}{2}\right)$$
$$V_1\left(\frac{L}{2}\right) = V_2\left(\frac{L}{2}\right)$$
$$V_2'(L) = V_3'(L)$$
$$V_2(L) = V_3(L)$$
$$V_3'\left(\frac{3L}{2}\right) = V_4'\left(\frac{3L}{2}\right)$$
$$V_3\left(\frac{3L}{2}\right) = V_4\left(\frac{3L}{2}\right)$$
$$V_4(2L) = 0$$

2-5.4 Integration Constants in Terms of F_p

$$c_1 = \frac{5L^2}{192} F_p$$
$$c_2 = 0$$
$$c_3 = \frac{21L^2}{192} F_p$$
$$c_4 = \frac{-L^3}{72} F_p$$
$$c_5 = \frac{-45L^2}{192} F_p$$
$$c_6 = \frac{29L^3}{288} F_p$$
$$c_7 = \frac{27L^2}{192} F_p$$
$$c_8 = \frac{-25L^3}{288} F_p$$

2-5.5 Deflection Equations in Terms of F_p

$$EIV_1(x) = \frac{-23F_p}{96}\left(\frac{x^3}{6}\right) + \frac{5L^2x}{192}F_p$$

$$V_1(x) = \frac{F_p}{576EI}\left(-23x^3 + 15L^2x\right)$$

$$EIV_2(x) = \frac{-23}{96}F_p\left(\frac{x^3}{6}\right) + \frac{2}{3}F_p\left(\frac{x^3}{6} - \frac{Lx^2}{4}\right) + \frac{21L^2x}{192}F_p - \frac{L^3}{72}F_p$$

$$V_2(x) = \frac{F_p}{576EI}\left(41x^3 - 96Lx^2 + 63L^2x - 8L^3\right)$$

$$EIV_3(x) = \frac{-23}{96}F_p\left(\frac{x^3}{6}\right) + \frac{2}{3}F_p\left(\frac{x^3}{6} - \frac{Lx^2}{4}\right) - \frac{66}{96}F_p\left(\frac{x^3}{6} - \frac{Lx^2}{2}\right) - \frac{45L^2x}{192}F_p + \frac{29L^3}{288}F_p$$

$$V_3(x) = \frac{F_p}{576EI}\left(-25x^3 + 102Lx^2 - 135L^2x + 58L^3\right)$$

$$EIV_4(x) = \frac{-23}{96}F_p\left(\frac{x^3}{6}\right) + \frac{2}{3}F_p\left(\frac{x^3}{6} - \frac{Lx^2}{4}\right) - \frac{66}{192}F_p\left(\frac{x^3}{6} - \frac{Lx^2}{2}\right) + \frac{F_p}{3}\left(\frac{x^3}{6} - \frac{3Lx^2}{2}\right) + \frac{27L^2x}{192}F_p - \frac{25L^3}{288}F_p$$

$$V_4(x) = \frac{F_p}{576EI}\left(7x^3 - 42Lx^2 + 81L^2x - 50L^3\right)$$

2-5.6

A17.1, requirement 8.4.12.2.1 takes maximum deflection at $x = L/2$.

$$\Delta_{\max} = V_1\left(\frac{L}{2}\right) = \frac{F_p}{576EI}\left[-23\left(\frac{L}{2}\right)^3 + 15L^2\left(\frac{L}{2}\right)\right]$$

$$= \frac{F_p}{576EI}\left[\frac{-23L^3}{8} + \frac{60L^3}{8}\right]$$

$$= \frac{F_p}{576EI}\left(\frac{37L^3}{8}\right)$$

$$\Delta_{\max} = \frac{37L^3}{4608EI}F_p \approx \frac{F_pL^3}{124.5EI} = \frac{2F_pL^3}{249EI}$$

$$I_x = \frac{2F_pL^3}{249E\Delta}$$

This matches 8.4.12.2.1(a) for I_x for IBC/NBCC.

2-6 REQUIREMENT 8.4.14.1.1(b)

Requirement 8.4.14.1.1(a) and IBC/ASCE 7 require a vertical seismic force of $\pm 0.2S_{DS}W_p$. This equation can be rewritten in terms of F_a and S_s with the following substitutions:

$$S_{MS} = F_aS_s$$

NOTE: This equation also appears as eq. 11.4-1 in ASCE 7-10.

where

F_a = site coefficient

S_{MS} = the maximum considered earthquake, 5% damped, spectral response acceleration at short periods adjusted for site class effects

S_s = spectral response acceleration at 0.2 sec, normalized for Site Class B

$$S_{DS} = \frac{2}{3} S_{MS} \qquad \text{(eq. 11.4-3, ASCE 7-10)}$$

Therefore

$$S_{DS} = \frac{2}{3} F_a S_S$$

Expanding 8.4.14.1.1(a) in terms of F_a and S_s yields

$$\pm 0.2\left(\frac{2}{3} F_a S_S\right) W_p$$

NBCC does not provide a vertical seismic force. Both IBC and NBCC are based off of NEHRP provisions. To provide a more conservative approach and seismic forces similar to those seen in the U.S., an equivalent vertical force was added in A17.1/B44 for NBCC jurisdictions.

Provided in NBCC 2010 terms

$$F_v = \pm\left(0.2\left[\frac{2}{3} F_a S_a(0.2)\right]\right) W_p$$

where

F_a = NBCC site class coefficient (NBCC 2010, Table 4.1.8.4B)

$S_a(0.2)$ = spectral response acceleration value at 0.2 sec

2-7 REQUIREMENT 8.4.14.1.2, LOAD COMBINATIONS

NBCC provides a load combination (in LSD) of

$$D + E$$

Converting to ASD (dividing by 1.4) would yield

$$0.7D + 0.7E$$

IBC/ASCE 7 provides two load combinations (in ASD)

$$D + 0.7E$$

and

$$0.6D + 0.7E$$

The IBC/ASCE 7 combinations provide a worst-case loading, particularly in consideration of overturning with a vertical seismic force.

With the addition of a vertical seismic force for NBCC jurisdictions, the IBC/ASCE 7 combinations have been adopted for use in both IBC and NBCC jurisdictions.

Part 3
Sample Calculations

Sample calculations are provided to assist the user in applying A17.1/B44 requirements. Sample calculation 3-1 shows examples of when A17.1/B44 seismic requirements are necessary and what force level requirements govern for each case. Sample calculations 3-2 through 3-4 show A17.1/B44 requirements using IBC and NBCC applications with SI units. Sample calculations 3-5 through 3-7 show A17.1/B44 requirements using IBC and NBCC applications with imperial units. Imperial dimensional units are used since most of the building codes and standards favor imperial units. The applicable A17.1/B44 code sections are found under each calculation header. All references to Sections or requirements within each calculation are for A17.1/B44 unless otherwise specified.

3-1 SAMPLE CALCULATION(S) 1: DETERMINING PROPER SEISMIC REQUIREMENTS AND FORCES

The forces are based on the applicable building code and A17.1. The applicable A17.1/B44 code requirements are 8.4(a), 8.4(b), 8.4.13, and 8.4.14.

3-1.1 Sample Calculation 1a

3-1.1.1 Given:

(a) Building installed in jurisdiction where International Building Code (IBC) 2006 is in effect.

(b) Latest Safety Code for Elevators and Escalators (ASME A17.1) is also in effect.

(c) Building is in Seismic Design Category C.

(d) Building has component importance factor of 1.5.

(e) Building has an S_{DS} of 0.95.

3-1.1.2 Determination of Proper Seismic Requirements and Force Levels. Per 8.4(a)(1), A17.1 seismic requirements (Section 8.4) are in effect for buildings with Seismic Design Category C and component importance factor of 1.5. Therefore, A17.1, Section 8.4 requirements are in effect.

Per 8.4(b)(1), building codes referencing Seismic Design Categories shall use force levels as referenced in 8.4.14.

Per 8.4.14.1(a), the horizontal earthquake component force level will be

$$F_p = \frac{0.4a_p S_{DS}}{\left(\dfrac{R_p}{I_p}\right)}\left[1+2\left(\frac{z}{h}\right)\right]W_p$$

Maximum force level (components at the top of building, $z/h=1$)

$$F_p = \frac{0.4(1)(0.95)}{\left(\dfrac{2.5}{1.5}\right)}[1+2]W_p = 0.68W_p$$

Minimum force level = $0.43W_p$.

NOTE: This minimum is dictated by requirement that F_p must not be less than $0.3S_{DS}I_pW_p$. Minimum force level calculated for components at building base ($z/h=0$) would yield $0.23W_p$.

3-1.1.3 Combining Nominal Loads Using Allowable Stress Design. The load combinations and load factors below will be used only in those cases in which they are specifically authorized by the applicable material design standards. The loads will be considered to act in the following combinations, whichever produces the most unfavorable effect on the component, fastenings, or supports:

$$0.6D + 0.7E$$

or

$$1.0D + 0.7E$$

where E = earthquake load as defined in 8.4.14.

Detailed examples of these force levels are shown in additional sample calculations provided in this Guide.

3-1.2 Sample Calculation 1b

3-1.2.1 Given:

(a) Building installed in jurisdiction where National Building Code of Canada (NBCC) 2010 is in effect.

(b) Latest Safety Code for Elevators and Escalators (ASME A17.1) is also in effect.

(c) Building is in Site Class C.

(d) 5% damped spectral response, $S_a(0.2)$, is 0.5.

(e) Earthquake importance factor for building, I_E, is 1.3.

(f) Building is not designated a post-disaster building.

(g) All connections for the elevator components/systems are rigid connections.

3-1.2.2 Determination of Proper Seismic Requirements and Force Levels. Per 8.4(a)(3), A17.1 seismic rules (Section 8.4) are in effect for buildings with design spectral response acceleration for a 0.2-s time period greater than 0.12 and $I_E F_a S_a(0.2)$ greater than or equal to 0.35. From NBCC 2010, Table 4.1.8.4.B, for Site Class C, $F_a = 1.0$ for $S_a(0.2)$ of 0.5.

$$I_E F_a S_a(0.2) = (1.3)(1.0)(0.5) = 0.65.$$

Therefore, A17.1, Section 8.4 rules are in effect.

Per 8.4(b)(1), building codes referencing design spectral response acceleration, $S(0.2)$, shall use force levels as referenced in 8.4.14.

Per 8.4.14.1(b), the horizontal earthquake component force level, F_p, will be

$$F_p = 0.3F_a S_a(0.2) I_E S_p W_p = 0.3F_a S_a(0.2) I_E \left(\left[C_p\right]\left[A_r\right] \left[\frac{1+2\left(\dfrac{h_x}{h_n}\right)}{R_p} \right] \right) W_p$$

For machinery with rigid connections, C_p, A_r, and R_p are found to be 1, 1, and 1.25, respectively, in NBCC 2010, Table 4.1.8.18, Category 11: Machinery, fixtures, equipment, ducts, and tanks (including contents) that are rigid and rigidly connected.

Maximum force level (machinery at top of building, $h_x/h_n = 1$)

$$F_p = 0.3(1.0)(0.5)(1.3)\left[(1)(1)\left(\frac{1+2}{1.25}\right)\right]W_p = 0.47W_p$$

Minimum force level (machinery at building base, $h_x/h_n = 0$)

$$F_p = 0.3(1.0)(0.5)(1.3)\left[(1)(1)\left(\frac{1+0}{1.25}\right)\right]W_p = 0.156W_p$$

Similarly, for rigid components (i.e., rail brackets, etc.), C_p, A_r, and R_p are found to be 1, 1, and 2.5, respectively, in NBCC 2010, Table 4.1.8.18.

Maximum force level (rigid components at top of building)

$$F_p = 0.3(1.0)(0.5)(1.3)\left[(1)(1)\left(\frac{1+2}{2.5}\right)\right]W_p = 0.23W_p$$

Minimum force level (rigid components at building base)

$$F_p = 0.3(1.0)(0.5)(1.3)(0.7)W_p = 0.14W_p$$

NOTE: S_p is taken as the minimum allowable value of 0.7. Calculated value of S_p with given NBCC parameters and component heights would yield 0.4, below the allowed minimum.

$$S_p = [(1)(1)((1 + 0)/2.5)\ ] = 0.4$$

3-1.2.3 Combining Nominal Loads Using Allowable Stress Design. The load combination and load factors below will be used only in those cases in which they are specifically authorized by the applicable material design standards. The loads will be considered to act in the following combinations:

$$0.6D + 0.7E$$

or

$$1.0D + 0.7E$$

where E = earthquake load as defined in 8.4.14.

Detailed examples of these force levels are shown in additional sample calculations provided in this Guide.

3-1.3 Sample Calculation 1c

3-1.3.1 Given:

(a) Building installed in jurisdiction where UBC 1997 is in effect.

(b) Latest Safety Code for Elevators and Escalators (ASME A17.1) is also in effect.

(c) Per UBC seismic map, building is in area of seismic zone 2b.

(d) Building is considered an essential facility.

(e) Soil profile type is S_B.

3-1.3.2 Determination of Proper Seismic Requirements and Force Levels. Per 8.4(a)(5), seismic design is required for buildings in seismic zone 2 or greater. A17.1 seismic requirements (Section 8.4) are in effect.

Per 8.4(b)(3), 8.4 force levels as dictated by seismic zone or the building code's component force level shall be used, whichever is greater.

A17.1/B44 and UBC 1997 are compared to determine appropriate force level.

(a) Per A17.1 requirements, horizontal force level will be either $0.25W_p$ or $0.5W_p$, depending on the specific requirement.

$$F_p = 0.25W_p$$

or

$$F_p = 0.5W_p$$

(b) Per UBC 1997, Rule 1632.2, component seismic horizontal force, F_p, is given as

$$F_p = 0.4C_a I_p W_p$$

From Table 16-I, a seismic zone factor, Z, is chosen based on the seismic zone map.

$$Z = 0.20$$

Using the Z value and the soil profile type, the seismic coefficient, C_a, is found fromTable 16-Q.

$$C_a = 0.20$$

For essential facilities, Table 16-K dictates the $I_p = 1.5$

Inserting all values into the force equation yields

$$F_p = 0.4(0.20)(1.5)W_p = 0.12W_p$$

The A17.1 force is greater. Forces as listed per seismic zone in A17.1, Section 8.4, will be used.

3-1.4 Sample Calculation 1d

3-1.4.1 Given:

(a) Building installed in jurisdiction where BOCA 1996 is in effect.

(b) Latest Safety Code for Elevators and Escalators (ASME A17.1) is also in effect.

(c) Per BOCA seismic map, building is in area with $A_v = 0.15$.

(d) Building is designated with Seismic Performance Category C and Seismic Hazard Exposure Group II.

(e) All connections for the elevator components/systems are direct connections.

3-1.4.2 Determination of Proper Seismic Requirements and Force Levels. Per 8.4(a)(4), A17.1 seismic requirements (Section 8.4) are in effect for buildings with Seismic Performance Category C and Seismic Hazard Exposure Group II. Therefore, A17.1, Section 8.4 applies.

Per 8.4(b)(3), force levels as dictated by 8.4.13 shall be used when building code references ground motions in terms of A_v. Per 8.4.13, the greater of the building code or A17.1 seismic zone component force level shall be used.

Compare force levels per A17.1 and BOCA 1996 to determine appropriate force level.

(a) Per A17.1, requirement 8.4.13.1, for $A_v = 0.15$, the equivalent A17.1 seismic zone will be zone 2.

Per requirements in A17.1, minimum force level for seismic zone 2 will be

$$F_p = 0.25W_p$$

(b) Per BOCA 1996, Rule 1610.6.4, component seismic force, F_p, is given as

$$F_p = A_vC_cPa_cW_p$$

where

W_C = the operating weight of the mechanical, electrical component or system

(1) Per BOCA 1996, Table 1610.6.4(1)

component seismic coefficient, $C_c = 1.25$

performance criteria factor, $P = 1$

(2) Per BOCA 1996, Table 1610.6.4(2)

attachment amplification factor, $a_c = 1.0$

(c) Calculating force level

$$F_p = (0.15)(1.25)(1)(1)W_p = 0.1875W_p$$

(d) Comparing Force Levels. Minimum A17.1 force level is greater ($0.1875W_p < 0.25W_p$). Therefore, A17.1 force levels described for seismic zone 2 should be used.

3-1.5 Sample Calculation 1e

3-1.5.1 Given:

(a) Building installed in jurisdiction where Standard Building Code (SBC) 1994 is in effect.

(b) Latest Safety Code for Elevators and Escalators (ASME A17.1) is also in effect.

(c) Per SBC 1994, Contour Map of Effective Peak Velocity-Related Acceleration Coefficient, A_v (see Fig. 3-1.5.1-1), building is in area between peak velocity-related acceleration coefficient contours 0.1 and 0.2, with $A_v = 0.18$ (e.g., the western part of Tennessee).

(d) Building information states Seismic Hazard Exposure Group III.

(e) All connections for the elevator components/systems are direct connections.

3-1.5.2 Determination of Proper Seismic Requirements and Force Levels. Per 8.4(a)(4), A17.1 seismic requirements (Section 8.4) are in effect for buildings with Seismic Performance Category C and Seismic Hazard Exposure Group II and greater. Therefore, A17.1, Section 8.4 requirements are in effect.

Per 8.4(b)(2), force levels as dictated by 8.4.13 shall be used when building code references ground motions in terms of A_v. Per 8.4.13, the greater of the building code or A17.1 seismic zone component force level shall be used.

Compare force levels per A17.1 and SBC 1994 to determine appropriate force level.

(a) Per A17.1, requirement 8.4.13.1, for $A_v = 0.18$, the equivalent A17.1 seismic zone will be zone 2.

Per requirements in A17.1, minimum force level for seismic zone 2 will be

$$F_p = 0.25W_p$$

(b) Per SBC 1994, Rule 1607.6.4

$$F_p = A_vC_cPa_cW_p$$

(1) Per SBC 1994, Table 1607.6.4A

component seismic coefficient, $C_c = 1.25$

performance criteria factor, $P = 1$

(2) Per SBC 1994, Table 1607.6.4B

attachment amplification factor, $a_c = 1.0$

(c) Calculating force level

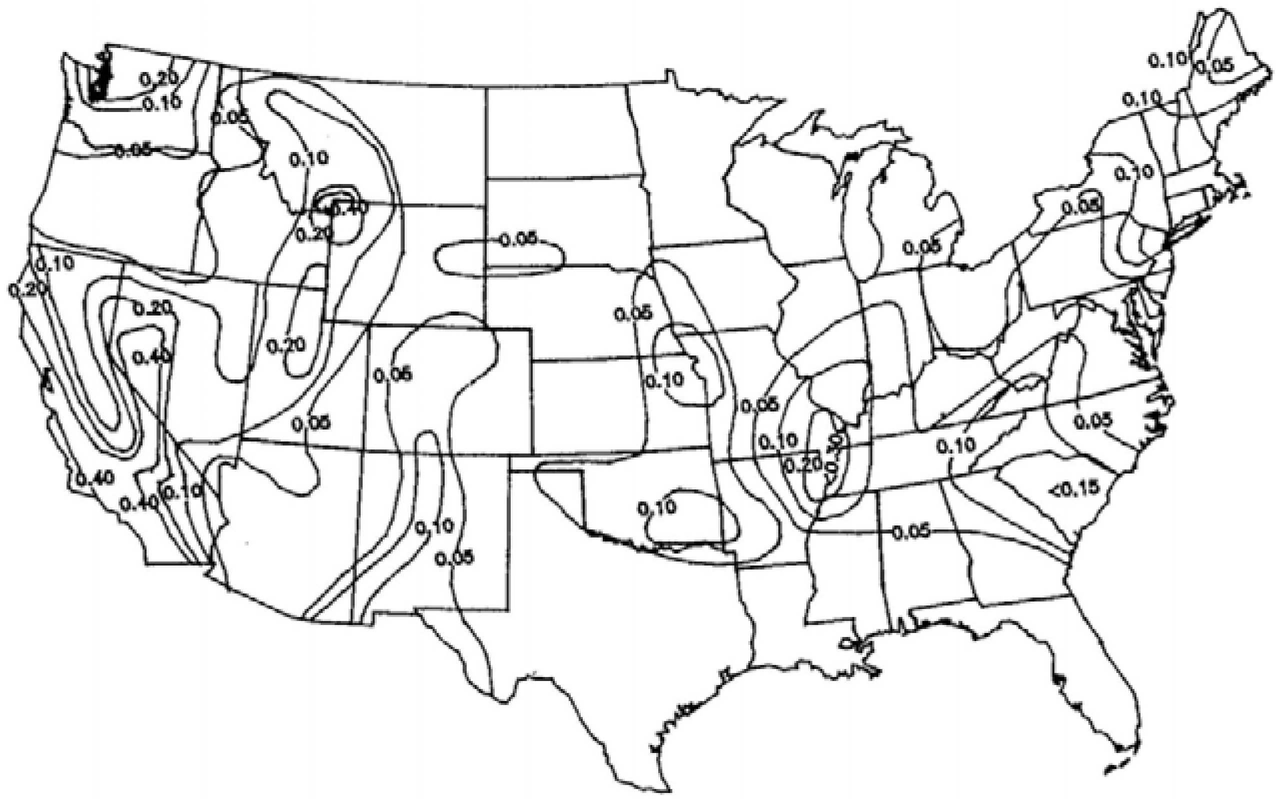
$$F_p = (0.18)(1.25)(1.5)(1)W_p = 0.3375W_p$$

(d) Comparing Force Levels. Depending on Section 8.4 requirement, the A17.1 seismic zone 2 force will be either $0.25W_p$ or $0.5W_p$. The SBC 1994 force level is greater than the minimum A17.1 level, but less than the maximum A17.1 force level ($0.25W_p < 0.3375W_p < 0.5W_p$).

(e) Force Level Determination. Force level to use will vary based on each Section 8.4 requirement.

For requirements using maximum A17.1 force level (for example, 8.4.2.1 with $F_p = 0.5W_p$), A17.1 force levels will be used.

Fig. 3-1.5.1-1 SBC 1994, Fig. 1607.1.5B, Contour Map of Effective Peak Velocity-Related Acceleration Coefficient, A_v



For section 8.4 requirements using minimum A17.1 force level (for example, 8.4.5.2.1 with $F_p = 0.25W_p$), SBC 1994 force levels will be used.

3-1.6 Sample Calculation 1f

3-1.6.1 Given:

(a) Building installed in jurisdiction where Standard Building Code (SBC) 1982 is in effect.

(b) Latest Safety Code for Elevators and Escalators (ASME A17.1) is also in effect.

(c) SBC 1982 uses a seismic zone map.

(d) Per SBC seismic zone map, building is in a seismic zone 2.

3-1.6.2 Determination of Proper Seismic Requirements and Force Levels. Per 8.4(a)(3), only buildings in seismic risk zones 2 or greater must adhere to Section 8.4. Since this building is in seismic zone 2, A17.1, Section 8.4 requirements are in effect.

Seismic force levels as described for seismic zone 2 throughout 8.4.1 through 8.4.12 apply. Requirements 8.4.13 and 8.4.14 will not apply.

3-1.7 Sample Calculation 1g

3-1.7.1 Given:

(a) Building installed in jurisdiction where Standard Building Code (SBC) 1982 is in effect.

(b) Latest Safety Code for Elevators and Escalators (ASME A17.1) is also in effect.

(c) SBC 1982 uses a seismic zone map.

(d) Per SBC seismic zone map, the building is in a seismic zone 0.

3-1.7.2 Determination of Proper Seismic Requirements and Force Levels. Per 8.4(a)(5), only buildings in seismic risk zones 2 or greater must adhere to Section 8.4. Since this building is in seismic zone 0, A17.1, Section 8.4 requirements are not applicable.

3-2 SAMPLE CALCULATION(S) 2: CONTROLLER ANCHORAGE (SI UNITS)

The applicable A17.1/B44 code requirements are 8.4(a), 8.4(b), 8.4.14, 8.4.15, and 8.4.2.3.

3-2.1 Sample Calculation 2a (SI Units – IBC)

3-2.1.1 Given:

(a) Building installed in jurisdiction where IBC 2006 is in effect.

(b) Latest Safety Code for Elevators and Escalators (ASME A17.1/CSA B44) is also in effect.

(c) $I_p = 1.5$

(d) $S_{DS} = 0.78$

(e) Seismic Design Category C

(f) Controller weight= 3 560 N

(g) Controller attachment elevation with respect to base, $z = 58$ m

(h) Average roof height of structure with respect to base, $h = 61$ m

3-2.1.2 Determination of Proper Seismic Requirements and Force Levels

(a) Per requirement 8.4(a)(1)

Seismic Design Category = C

component importance factor, $I_p = 1.5$

Therefore, Section 8.4 requirements are in effect.

(b) Per requirement 8.4(b)(1), building code references Seismic Design Categories. Therefore, force levels per 8.4.14 are to be used.

(c) Per requirement 8.4.14.1(a)

$$F_p = \text{horizontal force based on SD}$$
$$= 9.807\left(\frac{0.4a_pS_{DS}}{\dfrac{R_p}{I_p}}\left[1+2\left(\frac{z}{h}\right)\right]W_p\right)$$

where

a_p = 1.0 (also reference ASCE 7-10, Table 13.6-1)
R_p = 2.5 (also reference ASCE 7-10, Table 13.6-1)
W_p = 3 560 N [per requirement 8.4.15(a)]

$$F_p = \left[\frac{0.4(1)(0.78)}{\dfrac{2.5}{1.5}}\right]\left[1+2\left(\frac{58\text{ m}}{61\text{ m}}\right)\right](3\,560\text{N}) = 1\,933.7\text{ N}$$

F_p is not required to be greater than

$$F_p = 1.6S_{DS}\,I_p\,W_p = 1.6(0.78)3\,560\text{ N} = 4\,442.9\text{ N}$$

F_p shall not be taken as less than

$$F_p = 0.3S_{DS}\,I_p\,W_p = 0.3(0.78)3\,560\text{ N} = 1\,249.6\text{ N}$$

Therefore, F_p = 1 933.7 N is acceptable.

(d) Per requirement 8.4.14.1.1(a)

$$F_v = \text{vertical force} = \pm 0.2S_{DS}W_p$$
$$= \pm 0.2(0.78)3\,560\text{ N} = \pm 555.4\text{ N}$$

3-2.1.3 Determination of Proper Seismic Loading

Per requirement 8.4.14.1.2, IBC/ASCE 7 basic load combinations for ASD are

$0.6D + 0.7E$ [load combination 8.4.14.1.2(b)]

or

$D + 0.7E$ [load combination 8.4.14.1.2(a)]

whichever is more stringent, where

D = dead load = W_p for this application
E = earthquake load = $F_p + F_v$

The seismic loading to be used will be the most stringent of the four cases outlined in Mandatory Appendix I, Figs. I-1 through I-4.

Maximum tension on the controller anchors will be generated with Case 1 (see Mandatory Appendix I, Fig. I-1).

It is not the purpose of this example to design a specific anchorage of the controller to its supports. Depending on the medium to which the controller is attached, design guidance is given in requirements 8.4.2.3.3(a) through (d). The analysis of the fastening will be based on best engineering practice.

NOTE: ASCE 7-10, Section 13.4.2, Anchors in Concrete or Masonry: Anchors embedded in concrete or masonry shall be proportioned to carry the least of the following:

(a) 1.3 times the force in the component and its supports due to the prescribed forces.

(b) the maximum force that can be transferred to the anchor by the component and its supports. The value of R_p used in Section 13.3.1 to determine the forces in the connected part shall not exceed 1.5 unless

(1) the component anchorage is designed to be governed by the strength of a ductile steel element

(2) the design of the post-installed anchors in concrete used for component anchorage is prequalified for seismic applications in accordance with ACI 355.2

(3) the anchor is designed in accordance with Section 14.2.2.14

3-2.2 Sample Calculation 2b (SI Units – NBCC)

3-2.2.1 Given:

(a) Building installed in jurisdiction where NBCC is in effect.

(b) Latest Safety Code for Elevators and Escalators (ASME A17.1/CSA B44) is also in effect.

(c) I_E= 1.5

(d) Site Class C

(e) $S_a(0.2)$= 0.98

(f) F_a = 1 (per NBCC, Table 4.1.8.4.B)

(g) Controller weight = 3 560 N

(h) Controller attachment elevation with respect to base, h_x = 58 m

(i) Average roof height of structure with respect to base, h_n = 61 m

3-2.2.2 Determination of Proper Seismic Requirements and Force Levels

(a) Per requirement 8.4(a)(3)

$$I_E F_a S_a(0.2) = (1.5)(1)(0.98) = 1.47 > 0.35$$

Therefore, Section 8.4 requirements are in effect.

(b) Per requirement 8.4(b)(1), building code references $S_a(0.2)$. Therefore, force levels per 8.4.14 are to be used.

(c) Per requirement 8.4.14.1(b) (and NBCC 2010, 4.1.8.18)

$$F_p = \text{horizontal force based on SD} = 0.3F_a S_a(0.2) I_E S_p W_p$$

NOTE: NBCC 2010, 4.1.8.18 lists F_p as V_p. A171.1/B44 uses the F_p term to maintain a common term for similar IBC/NBCC equations.

(1) Per NBCC 2010, Table 4.1.8.18, Category 11

$A_r = 1.0$
$C_p = 1.0$
$R_p = 1.25$

NOTE: Controllers can be considered machinery that are rigid and rigidly connected. See Note (3) from requirement 8.4.14.1(b).

$$S_p = \frac{C_p A_r A_x}{R_p} = \frac{C_p A_r\left(1+2\dfrac{h_x}{h_n}\right)}{R_p} = \frac{(1)(1)\left(1+2\dfrac{58}{61}\right)}{1.25} = 2.32$$

where S_p minimum allowed = 0.7 and S_p maximum need not be more than four. Calculated S_p falls within the acceptable range and will be used.

(2) Per requirement 8.4.15(a)

$$W_p = 3\,560 \text{ N}$$

Therefore

$$F_p = 0.3F_a S_a(0.2) I_E S_p W_p = 0.3(1)(0.98)(1.5)(2.32)(3\,560 \text{ N}) = 3\,642.3 \text{ N}$$

(d) Per requirement 8.4.14.1.1(b)

$$F_v = \pm 0.2\left[\frac{2}{3}F_a S_a(0.2)\right]W_p$$

$$= \pm 0.2\left[\frac{2}{3}(1)(0.98)\right](3\,560 \text{ N}) = \pm 465.2 \text{ N}$$

3-2.2.3 Determination of Proper Seismic Loading. Per requirement 8.4.14.1.2, IBC/ASCE 7 basic load combinations for ASD are

$$0.6D + 0.7E \text{ [load combination 8.4.14.1.2(b)]}$$

or

$$D + 0.7E \text{ [load combination 8.4.14.1.2(a)]}$$

whichever is more stringent, where

D = dead load = W_p for this application
E = earthquake load = $F_p + F_v$

The seismic loading to be used will be the most stringent of the four cases outlined in Mandatory Appendix I, Figs. I-1 through I-4.

Maximum tension on the controller anchors will be generated with Case 1 (see Mandatory Appendix I, Fig. I-1).

It is not the purpose of this example to design a specific anchorage of the controller to its supports. Depending on the medium to which the controller is attached, design guidance is given in requirements 8.4.2.3.3(a) through (d). The analysis of the fastening will be based on best engineering practice.

3-3 SAMPLE CALCULATION(S) 3: GUIDE RAIL BRACKET SPACING (SI UNITS)

The applicable A17.1/B44 code requirements are 8.4(a), 8.4(b), 8.4.8.9, 8.4.12, 8.4.14, and 8.4.15.

3-3.1 Sample Calculation 3a (SI Units – IBC)

3-3.1.1 Given:

(a) Building installed in jurisdiction where IBC 2006 is in effect.

(b) Latest Safety Code for Elevators and Escalators (ASME A17.1/CSA B44) is also in effect.

(c) $I_p = 1.5$

(d) $S_{DS} = 0.78$

(e) Seismic Design Category C

(f) A standard overhead traction elevator system with the following:

(1) 38 426 N car weight
(2) 15 575 N capacity
(3) 2 100 N traveling cable weight
(4) 4 619 N compensation weight
(5) 4.9 m car guide overall height (CL lower to CL upper guide)
(6) Overall building height = 61 m
(7) Center of gravity of car at its highest point = 52 m
(8) Center of gravity of car at its lowest point = 2 m
(9) Center of gravity located one-third above lower car position restraints
(10) 22.5 kg/m steel car guide rails to be used

3-3.1.2 Determination of Proper Seismic Requirements and Force Levels

(a) Per requirement 8.4(a)(1), Seismic Design Category C

$$I_p = 1.5$$

Therefore, Section 8.4 requirements are in effect.

(b) Per requirement 8.4(b)(1), building code references Seismic Design Categories. Therefore, force levels per 8.4.14 are to be used.

(c) Per requirement 8.4.14.1(a)

$$F_p = \text{horizontal force based on SD}$$

$$= \frac{0.4a_p S_{DS}}{\frac{R_p}{I_p}}\left(1 + 2\frac{z}{h}\right)W_p$$

where

a_p = 1.0 (also reference ASCE 7-10, Table 13.6-1)
R_p = 2.5 (also reference ASCE 7-10, Table 13.6-1)

$$\text{max. } F_p = \frac{0.4(1)(0.78)}{\frac{(2.5)}{(1.5)}}\left(1 + 2\frac{52}{61}\right)W_p = 0.506W_p$$

$$\text{min. } F_p = \frac{0.4(1)(0.78)}{\frac{(2.5)}{(1.5)}}\left(1 + 2\frac{2}{61}\right)W_p = 0.199W_p$$

F_p is not required to be greater than

$$F_p = 1.6S_{DS}I_pW_p = 1.6(0.78)(1.5)W_p = 1.872W_p$$

F_p shall not be taken as less than

$$F_p = 0.3S_{DS}I_pW_p = 0.3(0.78)(1.5)W_p = 0.351W_p$$

Then maximum $F_p = 0.506W_p$ is acceptable (within minimum/maximum F_p range). Minimum $F_p = 0.199W_p$ is not acceptable. A minimum $F_p = 0.351W_p$ must be used.

Therefore

$$\text{max. } F_p = 0.506W_p$$

$$\text{min. } F_p = 0.351W_p$$

NOTE: Equating F_p formula with minimum allowed F_p and solving for z will indicate highest point where minimum allowed z will be used.

$$F_p = \frac{0.4(1)(0.78)}{\left(\frac{2.5}{1.5}\right)}\left(1 + 2\frac{z}{61}\right)W_p = 0.351W_p$$

or

$$z = 26.7 \text{ m}$$

Therefore, the minimum F_p value, $0.351W_p$, will be used for all heights up to 26.7 m (see Mandatory Appendix I, Fig. I-5). At heights above 26.7 m, the calculated F_p based on z will be used. The calculated F_p will continue to be used until $z=h$ or the maximum F_p value is reached. (Maximum F_p is not reached in this calculation.)

3-3.1.3 Determination of Seismic Forces for Layouts

(a) Per requirement 8.4.15(b)

$$W_p = \text{car weight} + 40\% \text{ capacity}$$

Insert given values

$$W_p = [38\,426 + (0.4)(15\,575)] = 44\,656 \text{ N}$$

Updating maximum and minimum F_p calculated above yields

$$\text{max. } F_p = (0.506)(44\,656) = 22\,595.9 \text{ N}$$

$$\text{min. } F_p = (0.351)(44\,656) = 15\,674.3 \text{ N}$$

Therefore

$$\text{max. } F_p = 22\,595.9 \text{ N}$$

$$\text{min. } F_p = 15\,674.3 \text{ N}$$

(b) Per requirement 8.4.8.9, the following force levels are to be shown on elevator layouts (see Mandatory Appendix I, Fig. I-6).

3-3.1.3.1 Requirement 8.4.8.9.1(a). Maximum guide rail force normal to *x-x* axis of guide rail, $F_{x\text{-}x}$

$$F_{x\text{-}x} = \frac{2F_p}{3} = \frac{2(22\,595.9)}{3} = 15\,063.9 \text{ N}$$

3-3.1.3.2 Requirement 8.4.8.9.2(a). Maximum guide rail force normal to *y-y* axis of guide rail, $F_{y\text{-}y}$

$$F_{y\text{-}y} = \frac{F_p}{3} = \frac{(22\,595.9)}{3} = 7\,532 \text{ N}$$

3-3.1.4 Determination of Car Rail Bracket Spacing Based on Seismic Requirements (Section 8.4). The force levels calculated in 3-3.1.3(b) are based on SD. To convert to ASD, IBC allows a factored load, $0.7F_p$, to be used.

A17.1/B44 has already accounted for this factored value as $(0.7F_p)$.

The factored value is used when sizing equipment and determining spacing of rail brackets (in stress calculations). See bending stress calculation section under A17.1/B44, requirement 8.4.12.1, and 3-4, Sample Calculation 4.

(a) Nomenclature

E = modulus of elasticity for steel, $E = 2.068 \times 10^5$ N/mm^2
F_p = horizontal seismic rail force (strength level)
I = moment of inertia, mm^4
ℓ = distance between car guide rail brackets, mm
Z = elastic section modulus, mm^3
Δ = maximum allowable deflection at center of rail span, mm (based on A17.1/B44, Table 8.4.12.2.2, reproduced in Mandatory Appendix I, Table I-1)

(1) Rail Section Properties for 22.5 kg/m Rail (Per A17.1/B44, Fig. 8.4.8.9; See Mandatory Appendix I, Fig. I-7)

$I_{x\text{-}x} = 1.99 \times 10^6$ mm^4 $\quad$ $I_{y\text{-}y} = 2.29 \times 10^6$ mm^4

$Z_{x\text{-}x} = 3.1 \times 10^4$ mm^3 $\quad$ $Z_{y\text{-}y} = 3.62 \times 10^4$ mm^3

(2) Maximum Allowable Deflection, 22.5 kg/m Rail (Per A17.1/B44, Table 8.4.12.2.2; See Mandatory Appendix I, Table I-1)

$$\Delta = 38 \text{ mm}$$

(b) Requirement 8.4.12.1, Maximum Weight Per Pair of Guide Rails

(1) Requirement 8.4.12.1.1(a)(1). Force normal to *x-x* axis of rail (no intermediate tie brackets)

NOTE: ℓ_1 can also be obtained from Fig. 3-3.1.4-1 with $2.93(0.7F_p) = 46\,344.2$ N.

$$\ell_1 = 4\,948\left[\frac{Z_x}{2.93\left(0.7F_p\right)}\right] = 4\,948\left[\frac{3.10\times10^4\,\text{mm}^3}{2.93\left(0.7\times22\,595.9\ \text{N}\right)}\right]$$
$$= 3\,309.8\ \text{mm}$$

$$\ell_1 = 3.31\ \text{m} \leftarrow \text{maximum length}$$

(2) Requirement 8.4.12.1.2(a)(1). Force normal to *y-y* axis of rail (no intermediate tie brackets)

$$\ell_2 = 9\,896\left[\frac{Z_y}{2.93\left(0.7F_p\right)}\right] = 9\,896\left[\frac{3.62\times10^4\,\text{mm}^3}{2.93\left(0.7\times22\,595.9\ \text{N}\right)}\right]$$
$$= 7\,729.9\ \text{mm}$$

$$\ell_2 = 7.73\ \text{m}$$

(c) Requirement 8.4.12.2, Required Moment of Inertia of Guide Rails

(1) Requirement 8.4.12.2.1. Force normal to *x-x* axis of rail

$$\ell_3 = \left(\frac{I_x 249\Delta E}{2F_p}\right)^{1/3} = \left[\frac{\left(1.99\times10^6\right)(249)(38)\left(2.068\times10^5\right)}{\left(2\times22\,595.9\right)}\right]^{1/3}$$
$$= 4\,416.8\ \text{mm}$$

$$\ell_3 = 4.42\ \text{m}$$

(2) Requirement 8.4.12.2.2. Force normal to *y-y* axis of rail

$$\ell_4 = \left(\frac{I_y 498\Delta E}{2F_p}\right)^{1/3} = \left[\frac{\left(2.29\times10^6\right)(498)(38)\left(2.068\times10^5\right)}{\left(2\times22\,595.9\right)}\right]^{1/3}$$
$$= 5\,831.5\ \text{mm}$$

$$\ell_4 = 5.83\ \text{m}$$

Per seismic requirements, ℓ_1 controls and maximum allowable rail bracket spacing is 3.31 m. This same spacing can be found using A17.1/B44, Fig. 8.4.8.2-4 (see Fig. 3-3.1.4-1).

3-3.1.5 Comparison of Car Rail Bracket Spacing Based on Part 2 Rail Requirements (Section 2.23). A17.1/B44, Part 2 rail requirements must also be checked against safety loading.

The shortest rail bracket spacing result from Section 8.4 and Section 2.23 would control the design.

(a) Per requirement 2.23.4.1

total load on safety, W_{safety} = car weight + capacity + traveling cable weight + compensation weight

$$W_{\text{safety}} = 38\,426 + 15\,575 + 2\,100 + 4\,619 = 60\,720\ \text{N or } 6\,192\ \text{kg}$$

The allowed bracket spacing is interpolated from Fig. 3-3.1.5-1.

For 22.5 kg/m rail
7 000 kg safety load has maximum bracket spacing of 3 m
5 443 kg safety load has maximum bracket spacing of 4.3 m

$$\left(\frac{6\,192\ \text{kg} - 7\,000\ \text{kg}}{5\,443\ \text{kg} - 7\,000\ \text{kg}}\right) = \left(\frac{\ell_{\text{Section 2.23}} - 3\ \text{m}}{4.3\ \text{m} - 3\ \text{m}}\right)$$

$$\ell_{\text{Section 2.23}} = 3.675\ \text{m} > \ell_1 = 3.31\ \text{m}$$

Per Section 2.23, maximum allowable rail spacing is 3.675 m. Therefore, Section 8.4 bracket spacing controls and maximum bracket spacing allowed is 3.31 m. This same spacing can be found using A17.1/B44, Fig. 2.23.4.1-1 (see Fig. 3-3.1.5-1).

3-3.1.6 Section 2.23 Versus Section 8.4 Control of Design: Additional Example. For comparison, the bracket spacing for the minimum F_p force will be found

$$\text{for } F_p = 0.351\ W_p = 15\,674.3\ \text{N}$$

(a) Requirement 8.4.12.1, Maximum Weight Per Pair of Guide Rails

(1) Requirement 8.4.12.1.1(a)(1). Force normal to *x-x* axis of rail (no intermediate tie brackets)

$$\ell_1 = 4\,948\left[\frac{Z_x}{2.93\left(0.7F_p\right)}\right] = 4\,948\left[\frac{3.1\times10^4}{2.93\left(0.7\times15\,674.3\right)}\right]$$
$$= 4\,771.31\ \text{mm}$$
$$\ell_1 = 4.77\ \text{m} \leftarrow \text{maximum length}$$

(2) Requirement 8.4.12.1.2(a)(1). Force normal to *y-y* axis of rail (no intermediate tie brackets)

$$\ell_2 = 9\,896\left[\frac{Z_y}{2.93\left(0.7F_p\right)}\right] = 9\,896\left[\frac{3.62\times10^4}{2.93\left(0.7\times15\,674.3\right)}\right]$$
$$= 11143.3\ \text{mm}$$
$$\ell_2 = 11.14\ \text{m}$$

(b) Requirement 8.4.12.2, Required Moment of Inertia of Guide Rails

(1) Force normal to *x-x* axis of rail

$$\ell_3 = \left(\frac{I_x 249\Delta E}{2F_p}\right)^{1/3} = \left[\frac{\left(1.99\times10^6\right)(249)(38)\left(2.068\times10^5\right)}{\left(2\times15\,674.3\right)}\right]^{1/3}$$
$$= 4\,989.5\ \text{mm}$$

$$\ell_3 = 5.0\ \text{m}$$

(2) Force normal to *y-y* axis of rail

$$\ell_4 = \left(\frac{I_y 498\Delta E}{2F_p}\right)^{1/3} = \left[\frac{\left(2.29\times10^6\right)(498)(38)\left(2.068\times10^5\right)}{\left(2\times15\,674.3\right)}\right]^{1/3}$$
$$= 6\,587.6\ \text{mm}$$

$$\ell_4 = 6.59\ \text{m}$$

Fig. 3-3.1.4-1 A17.1/B44, Fig. 8.4.8.2-4, 22.5 kg/m (15 lb/ft) Guide-Rail Bracket Spacing (Marked for Sample Calculation 3a)

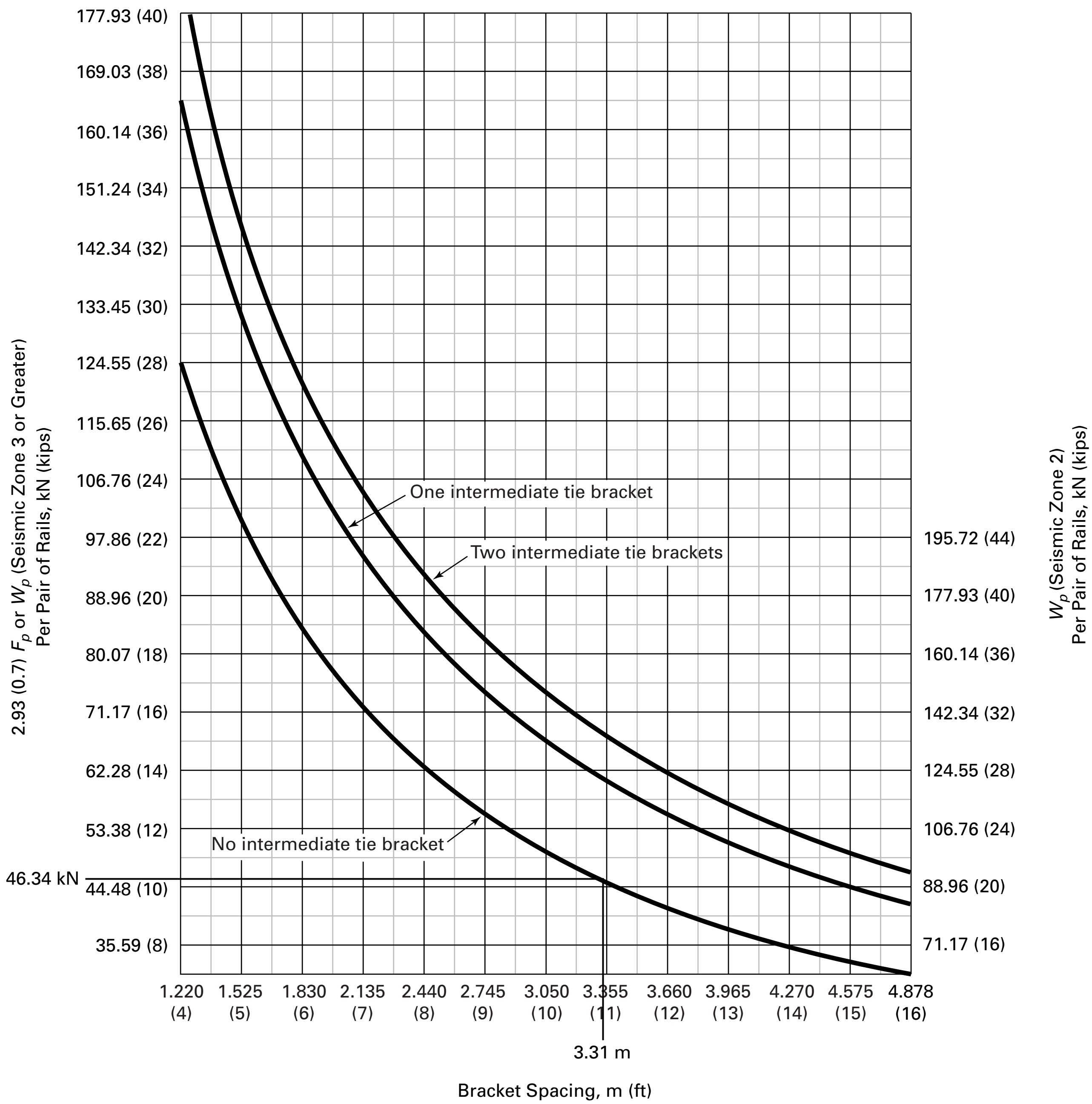
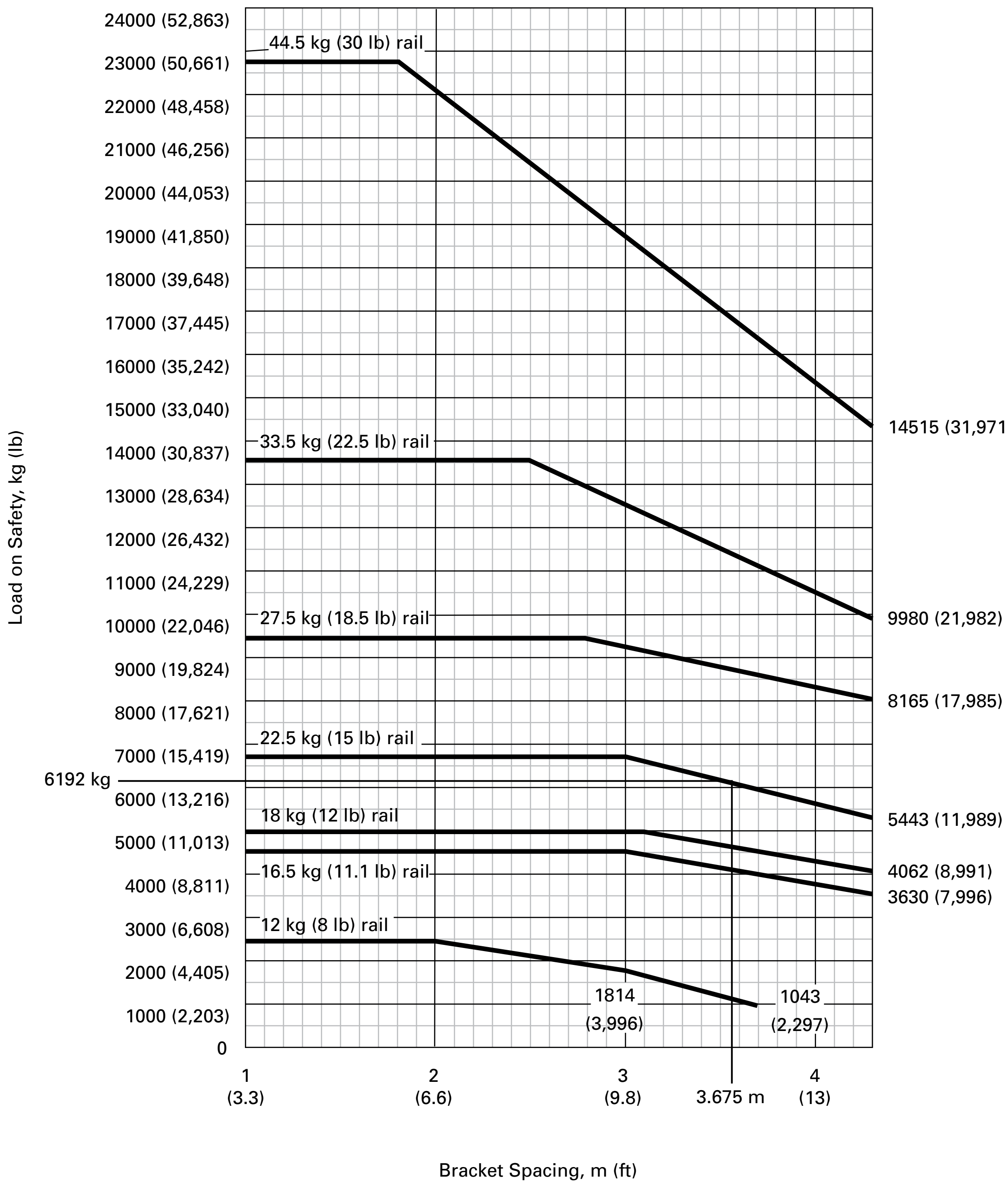


Fig. 3-3.1.5-1 A17.1/B44, Fig. 2.23.4.1-1 (Marked for Sample Calculation 3a and 3b)



Per seismic requirements, maximum rail bracket spacing will be 4.77 m. Comparing this to the bracket spacing found for Section 2.23

$$\ell_{\text{Section 2.23}} = 3.675 \text{ m} < \ell_1 = 4.77 \text{ m}$$

For the minimum F_p, the bracket spacing found in Section 2.23 controls the design.

3-3.2 Sample Calculation 3b (SI Units – NBCC)

3-3.2.1 Given:

(a) Building installed in jurisdiction where NBCC 2010 is in effect.

(b) Latest Safety Code for Elevators and Escalators (ASME A17.1/CSA B44) is also in effect.

(c) $I_E = 1.5$

(d) Site Class C

(e) $S_a(0.2) = 0.98$

(f) $F_a = 1$ (per NBCC, Table 4.1.8.4.B)

(g) A standard overhead traction elevator system with

(1) 38 426 N car weight

(2) 15 575 N capacity

(3) 2 100 N traveling cable weight

(4) 4 619 N compensation weight

(5) 4.9 m car guide overall height (CL lower to CL upper guide)

(6) Overall building height = 61 m

(7) Center of gravity of car at its highest point = 52 m

(8) Center of gravity of car at its lowest point = 2 m

(9) Center of gravity is located one-third above lower car position restraints.

(10) 22.5 kg/m steel car guide rails to be used

3-3.2.2 Determination of Proper Seismic Requirements and Force Levels

(a) Per requirement 8.4(a)(3)

$$I_E F_a S_a(0.2) = (1.5)(1)(0.98) = 1.47 > 0.35$$

Therefore, Section 8.4 requirements are in effect.

(b) Per requirement 8.4(b)(1), building code references $S(0.2)$ values. Therefore, force levels per 8.4.14 are to be used.

(c) Per 8.4.14.1(b) (and NBCC 2010, 4.1.8.18)

$$F_p = \text{horizontal seismic force based on SD} = 0.3F_a S_a(0.2) I_E S_p W_p$$

NOTE: NBCC 2010, 4.1.8.18 lists F_p as V_p. ASME A171.1/B44 uses the F_p term to maintain a common term for similar IBC/NBCC equations.

(1) Per NBCC 2010, Table 4.1.8.18, Category 18

$A_r = 1.0$
$C_p = 1.0$
$R_p = 2.5$

NOTE: Rails and rail brackets are considered rigid components with ductile material and connections.

$$S_p = \frac{C_p A_r A_x}{R_p} = \frac{C_p A_r \left(1 + 2\dfrac{h_x}{h_n}\right)}{R_p}$$

where S_p minimum allowed = 0.7 and S_p maximum need not be more than four.

Maximum S_p will be taken at the highest car position.
Minimum S_p will be taken at the lowest car position.

$$\text{calculated max. } S_p = \frac{(1)(1)\left[1 + 2\left(\dfrac{52}{61}\right)\right]}{2.5} = 1.08$$

$\leftarrow$ within allowed S_p range

$$\text{calculated min. } S_p = \frac{(1)(1)\left[1 + 2\left(\dfrac{2}{61}\right)\right]}{2.5} = 0.43$$

$\leftarrow$ outside allowed S_p range

Calculated maximum S_p is within allowed range
Calculated minimum S_p is below minimum allowed.
Therefore, minimum S_p used will be 0.7.

NOTE: Equating S_p formula with minimum allowed S_p and solving for h_x will indicate highest point where minimum allowed S_p will be used.

$$\frac{(1)(1)\left[1 + 2\dfrac{h_x}{h_n}\right]}{2.5} = 0.7$$

or

$$h_x = 0.375\, h_n$$

(This constraint is true for all rigid components with ductile material.)

$$h_x = 0.375(61 \text{ m}) = 22.9 \text{ m}$$

Then the minimum S_p will be used for all heights up to 22.9 m.

Inserting values for F_a, $S_a(0.2)$, I_E, and min./max. F_p yields

$$\text{max. } F_p = 0.3(1)(0.98)(1.5)(1.08)W_p = 0.476W_p$$

$$\text{min. } F_p = 0.3(1)(0.98)(1.5)(0.7)W_p = 0.309W_p$$

Therefore

$$\text{max. } F_p = 0.476W_p$$

$$\text{min. } F_p = 0.309W_p$$

3-3.2.3 Determination of Seismic Forces for Layouts

(a) Per requirement 8.4.15(b)

$$W_p = \text{car weight} + 40\% \text{ capacity}$$

Insert given values

$$W_p = [38\,426 + (0.4)(15\,575)] = 44\,656 \text{ N}$$

Updating maximum and minimum F_p yields

$$\text{max. } F_p = (0.476)(44\,656) = 21\,256.3 \text{ N}$$

$$\text{min. } F_p = (0.309)(44\,656) = 13\,798.7 \text{ N}$$

Therefore

$$\text{max. } F_p = 21256.3 \text{ N}$$

$$\text{min. } F_p = 13798.7 \text{ N}$$

(b) Per requirement 8.4.8.9, the following force levels are to be shown on elevator layouts (see Mandatory Appendix I, Fig. I-6):

(1) Requirement 8.4.8.9.1(a). Maximum guide rail force normal to *x-x* axis of guide rail, $F_{x\text{-}x}$

$$F_{x\text{-}x} = \frac{2F_p}{3} = \frac{2(21\,256.3)}{3} = 14\,170.9 \text{ N}$$

(2) Requirement 8.4.8.9.2(a). Maximum guide rail force normal to *y-y* axis of guide rail, $F_{y\text{-}y}$

$$F_{y\text{-}y} = \frac{F_p}{3} = \frac{(21\,256.3)}{3} = 7\,085.4 \text{ N}$$

Therefore

$$F_{x\text{-}x} = 14\,170.9 \text{ N}$$

$$F_{y\text{-}y} = 7\,085.4 \text{ N}$$

3-3.2.4 Determination of Car Rail Bracket Spacing Based on Seismic Requirements (Section 8.4). The force levels calculated in 3-3.2.3(b) are based on SD. To convert to ASD, IBC allows a factored load, $0.7F_p$, to be used. This same factored load will be used for NBCC to convert to ASD.

A17.1/B44 has already accounted for this factored value as $(0.7F_p)$.

The factored value is used when sizing equipment and determining spacing of rail brackets (in stress calculations). See bending stress calculation section under A17.1/B44, requirement 8.4.12.1 and 3-4, Sample Calculation 4.

(a) Nomenclature

E = modulus of elasticity for steel, $E = 2.068 \times 10^5$ N/mm^2
F_p = horizontal seismic rail force (strength level)
I = moment of inertia, mm^4
ℓ = distance between car guide rail brackets, mm
Z = elastic section modulus, mm^3
Δ = maximum allowable deflection at center of rail span, mm (based on Mandatory Appendix I, Table I-1)

(1) Rail Section Properties for 22.5 kg/m Rail (See Mandatory Appendix I, Fig. I-7)

$I_x = 1.99 \times 10^6$ mm^4 $\quad$ $I_y = 2.29 \times 10^6$ mm^4

$Z_x = 3.1 \times 10^4$ mm^3 $\quad$ $Z_y = 3.62 \times 10^4$ mm^3

(2) Maximum Allowable Deflection, 22.5 kg/m Rail (See Mandatory Appendix I, Table I-1)

$$\Delta = 38 \text{ mm}$$

(b) Requirement 8.4.12.1, Maximum Weight Per Pair of Guide Rails

(1) Requirement 8.4.12.1.1(a)(1). Force normal to *x-x* axis of rail (no intermediate tie brackets)

NOTE: ℓ_1 can also be obtained from A17.1/B44, Fig. 8.4.8.2-4 with $2.93(0.7F_p) = 10{,}433.8$ lbf. See Fig. 3-3.2.4-1.

$$\ell_1 = 4\,948\left[\frac{Z_x}{2.93(0.7F_p)}\right] = 4\,948\left[\frac{3.10\times10^4\,\text{mm}^3}{2.93(0.7\times 21\,256.3\text{ N})}\right]$$
$$= 3\,518.3\,\text{mm}$$
$$\ell_1 = 3.5 \text{ m} \leftarrow \text{maximum length}$$

(2) Requirement 8.4.12.1.2(a)(1). Force normal to *y-y* axis of rail (no intermediate tie brackets)

$$\ell_2 = 9\,896\left[\frac{Z_y}{2.93(0.7F_p)}\right] = 9\,896\left[\frac{3.62\times10^4\,\text{mm}^3}{2.93(0.7\times 21\,256.3\text{ N})}\right]$$
$$= 8\,217.0\text{ mm}$$
$$\ell_2 = 8.2 \text{ m}$$

(c) Requirement 8.4.12.2, Required Moment of Inertia of Guide Rails

(1) Requirement 8.4.12.2.1. Force normal to *x-x* axis of rail

$$\ell_3 = \left(\frac{I_x 249\Delta E}{2F_p}\right)^{1/3} = \left[\frac{(1.99\times10^6)(249)(38)(2.068\times10^5)}{(2\times 21\,256.3)}\right]^{1/3}$$
$$= 4\,507.7\text{ mm}$$
$$\ell_3 = 4.5 \text{ m}$$

(2) Requirement 8.4.12.2.2. Force normal to *y-y* axis of rail

$$\ell_4 = \left(\frac{I_y 498\Delta E}{2F_p}\right)^{1/3} = \left[\frac{(2.29\times10^6)(498)(38)(2.068\times10^5)}{(2\times 21\,256.3)}\right]^{1/3}$$
$$= 5\,951.5\text{ mm}$$
$$\ell_4 = 6.0 \text{ m}$$

Per seismic requirements, ℓ_1 controls, and maximum allowable rail bracket spacing is 3.5 m. This same spacing can be found using A17.1/B44, Fig. 8.4.8.2-4 (see Fig. 3-3.2.4-1).

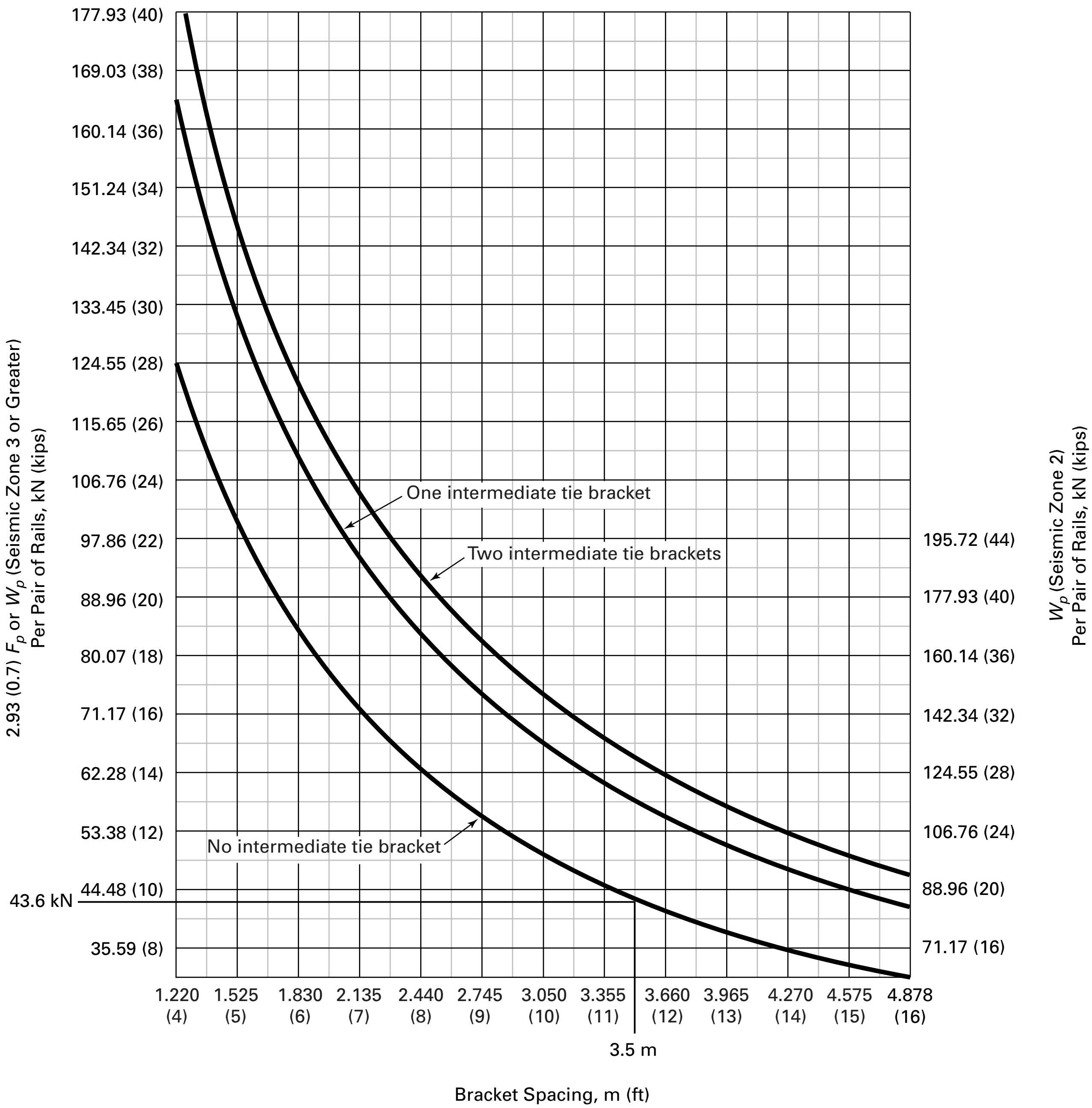
3-3.2.5 Comparison of Car Rail Bracket Spacing Based on Part 2 Rail Requirements (Section 2.23). A17.1/B44, Part 2 rail requirements must also be checked against safety loading.

The shortest rail bracket spacing result from Section 8.4 and Section 2.23 would control the design.

(a) Per requirement 2.23.4.1

$$\text{total load on safety, } W_{safety} = \text{car weight} + \text{capacity} + \text{traveling cable weight} + \text{compensation weight}$$

Fig. 3-3.2.4-1 A17.1/B44, Fig. 8.4.8.2-4, 22.5 kg/m (15 lb/ft) Guide-Rail Bracket Spacing (Marked for Sample Calculation 3b)



$W_{\text{safety}} = 38\,426 + 15\,575 + 2\,100 + 4\,619 = 60\,720$ N or 6 192 kg

The allowed bracket spacing is interpolated from A17.1/B44, Fig. 2.23.4.1-1 (see Fig. 3-3.1.5.1).

For 22.5 kg/m rail
7 000 kg safety load has maximum bracket spacing of 3 m
5 443 kg safety load has maximum bracket spacing of 4.3 m

$$\left(\frac{6\,192\text{ kg} - 7\,000\text{ kg}}{5\,443\text{ kg} - 7\,000\text{ kg}}\right) = \left(\frac{\ell_{\text{Section 2.23}} - 3\text{ m}}{4.3\text{ m} - 3\text{ m}}\right)$$

$$\ell_{\text{Section 2.23}} = 3.675\text{ m} > \ell_1 = 3.5\text{ m}$$

Therefore, Section 8.4 bracket spacing controls and maximum bracket spacing allowed is 3.5 m. This same spacing can be found can be found using A17.1/B44, Fig. 2.23.4.1-1 (see Fig. 3-3.1.5.1).

3-3.2.6 Section 2.23 Versus Section 8.4 Control of Design – Additional Example. For comparison, the bracket spacing for the minimum F_p force will be found

$$\text{for } F_p = 0.309\ W_p = 13\,798.7\text{ N}$$

(a) Requirement 8.4.12.1, Maximum Weight Per Pair of Guide Rails

(1) Requirement 8.4.12.1.1(a)(1). Force normal to *x-x* axis of rail (no intermediate tie brackets)

$$\ell_1 = 4\,948\left[\frac{Z_x}{2.93(0.7F_p)}\right] = 4\,948\left[\frac{3.10\times10^4}{2.93(0.7\times13\,798.7\text{ N})}\right] = 5\,419.9\text{ mm}$$

$$\ell_1 = 5.4\text{ m} \leftarrow \text{maximum length}$$

(2) Requirement 8.4.12.1.2(a)(1). Force normal to *y-y* axis of rail (no intermediate tie brackets)

$$\ell_2 = 9\,896\left[\frac{Z_y}{2.93(0.7F_p)}\right] = 9\,896\left[\frac{3.62\times10^4}{2.93(0.7\times13\,798.7\text{ N})}\right] = 12\,658\text{ mm}$$

$$\ell_2 = 12.7\text{ m}$$

(b) Requirement 8.4.12.2, Required Moment of Inertia of Guide Rails

(1) Force normal to *x-x* axis of rail

$$\ell_3 = \left(\frac{I_x 249\Delta E}{2F_p}\right)^{1/3} = \left[\frac{(1.99\times10^6)(249)(38)(2.068\times10^5)}{(2\times13\,798.7)}\right]^{1/3} = 5\,206\text{ mm}$$

$$\ell_3 = 5.2\text{ m}$$

(2) Force normal to *y-y* axis of rail

$$\ell_4 = \left(\frac{I_y 498\Delta E}{2F_p}\right)^{1/3} = \left[\frac{(2.29\times10^6)(249)(38)(2.068\times10^5)}{(2\times13\,798.7)}\right]^{1/3} = 6\,873.5\text{ mm}$$

$$\ell_4 = 6.9\text{ m}$$

Per seismic requirements, maximum rail bracket spacing will be 5.2 m. Comparing this to the bracket spacing found for Part 2

$$\ell_{\text{Section 2.23}} = 3.675\text{ m} < \ell_1 = 5.2\text{ m}$$

For the minimum F_p, the bracket spacing found in Section 2.23 controls the design.

3-4 SAMPLE CALCULATION(S) 4: GUIDE RAIL BRACKET STRENGTH AND DESIGN (SI UNITS)

The applicable A17.1/B44 code requirements are 8.4(a), 8.4(b), 8.4.8.7, 8.4.12, 8.4.14, and 8.4.15.

3-4.1 Sample Calculation 4a (SI Units – IBC)

3-4.1.1 Given:

(a) Building installed in jurisdiction where IBC 2006 is in effect.
(b) Latest Safety Code for Elevators and Escalators (ASME A17.1/CSA B44) is also in effect.
(c) $I_p = 1.0$
(d) $S_{DS} = 0.75$
(e) Seismic Design Category D
(f) Counterweight weight = 33361.7 N
(g) Counterweight is two-thirds full
(h) Distance between upper and lower position restraints is greater than rail bracket span, $L > \ell$
(i) Center of gravity of counterweight at its highest point, $z = 61$ m
(j) Average roof height of structure with respect to base, $h = 67$ m

3-4.1.2 Determination of Proper Seismic Requirements and Force Levels

(a) Per requirement 8.4(a)(1)

Seismic Design Category = D

component importance factor, $I_p = 1.0$

Therefore, Section 8.4 requirements are in effect.
(b) Per requirement 8.4(b)(1), building code references Seismic Design Categories. Therefore, force levels per 8.4.14 are to be used.
(c) Per requirement 8.4.14.1(a)

$$F_p = \text{horizontal force based on SD} = \frac{0.4a_pS_{DS}}{\frac{R_p}{I_p}}\left[1+2\left(\frac{z}{h}\right)\right]W_p$$

where

a_p = 1.0
R_p = 2.5
W_p = 33 361.7 N [per requirement 8.4.15(a)]

$$\text{max. } F_p = \frac{0.4(1)(0.75)}{\frac{2.5}{1.0}}\left[1+2\left(\frac{61\text{ m}}{67\text{ m}}\right)\right](33\,361.7)$$
$$= 11\,293.2\text{ N}$$

3-4.1.3 Guide Rail Bracket Design

(a) Per requirement 8.4.8.7 (and Table 8.4.8.7), the guide rail brackets must withstand the seismic loads specified in 8.4.8.2.6. These are summarized, for this case, in Table 8.4.8.7.

(1) To design for deflection, the rail bracket, its fastenings, and any building supports must have a combined deflection of not greater than 6 m with a horizontal seismic load, P, of (see Mandatory Appendix I, Fig. I-8)

$$P = (CB)\frac{2}{3}\left(F_p\right) = (1)\frac{2}{3}(11\,293.2) = 7\,528.8\text{ N}$$

(2) To design for stress, no permanent deformation may result from the combined stresses resulting from the horizontal seismic load, P, of

$$P = (CB)\frac{2}{3}\left(F_p\right) = (0.7)\frac{2}{3}(11\,293.2) = 5\,270.16\text{ N} \qquad (3\text{-}4.1.3\text{-}1)$$

This force is imposed directly on to the counterweight rail bracket.

ANSI/AISC 360-05, Chapter H, H3.2 states (see Note 1 and Mandatory Appendix I, Fig. I-9)

$$\left(\frac{P_r}{P_c}+\frac{M_r}{M_c}\right)+\left(\frac{V_r}{V_c}+\frac{T_r}{T_c}\right)^2 \le 1.0$$

M_c = allowable flexural strength (as defined in Chapter F)
M_r = required flexural strength
P_c = allowable tensile or compressive strength (as defined in Chapter D or E)
P_r = required axial strength (calculated value)
T_c = allowable torsional strength (as defined in Chapter H)
T_r = required torsional strength
V_c = allowable shear strength (as defined in Chapter G)
V_r = required shear strength

NOTE: See A17.1/B44, Table 8.4.8.7, Note 6.

AISC provides equalities, etc., in terms of allowable strength. A17.1 provides requirements in terms of allowable stress. In generic terms

Allowable Strength Design	Allowable Stress Design
$\frac{\text{Force}_{\text{allow}}}{\Omega} \ge \text{Force}_{\text{required}}$ (calculated force)	$\frac{\text{Stress}_{\text{Yield}}}{\Omega} = \text{Stress}_{\text{allowable}}$

Per H3

$$\Omega = 1.67$$

Therefore

Allowable Strength Design	Allowable Stress Design
$0.6\text{Force}_{\text{allow}} \ge \text{Force}_{\text{required}}$	$0.6\text{Stress}_{\text{Yield}} = \text{Stress}_{\text{allowable}}$

Per Table 8.4.8.7, the bracket force was factored by 0.7 [eq.(3-4.1.3-1)].

Allowable Strength Design	Allowable Stress Design
$0.6\text{Force}_{\text{allow}} \ge 0.7\text{Force}_{\text{required}}$	$0.6\text{Stress}_{\text{Yield}} = 0.7\text{Stress}_{\text{allowable}}$

or

$$0.86\text{ Stress}_{\text{Yield}} = \text{Stress}_{\text{allowable}}$$

Note that this is approximately the same stress limit that had been used in previous editions of A17.1 for bending stress in brackets.

3-4.2 Sample Calculation 4b (SI Units – NBCC)

3-4.2.1 Given:

(a) Building installed in jurisdiction where NBCC 2010 is in effect.
(b) Latest Safety Code for Elevators and Escalators (ASME A17.1/CSA B44) is also in effect.
(c) I_E = 1.0
(d) Site Class C
(e) $S_a(0.2)$ = 1.0
(f) F_a = 1 (per NBCC Table 4.1.8.4.B)
(g) Counterweight weight = 33 361.7 N
(h) Counterweight is two-thirds full
(i) Distance between upper and lower position restraints is greater than rail bracket span, $L > \ell$
(j) Center of gravity of counterweight at its highest point, z = 61 m
(k) Average roof height of structure with respect to base, h = 67 m

3-4.2.2 Determination of Proper Seismic Requirements and Force Levels

(a) Per requirement 8.4(a)(3)

$$I_E F_a S_a(0.2) = (1.0)(1)(1) = 1.0 > 0.35$$

Therefore, Section 8.4 requirements are in effect.

(b) Per requirement 8.4(b)(1), building code references $S(0.2)$ values. Therefore, force levels per 8.4.14 are to be used.

(c) Per 8.4.14.1(b) (and NBCC 2010, 4.1.8.18)

$$F_p = \text{horizontal seismic force based on SD} = 0.3F_aS_a(0.2)I_ES_pW_p$$

NOTE: NBCC 2010, 4.1.8.18 lists F_p as V_p. A17.1/B44 uses the F_p term to maintain a common term for similar IBC/NBCC equations.

where

W_p = 33 361.7 N [per requirement 8.4.15(a)]

and F_a, $S_a(0.2)$, and I_E are provided above.

$$S_p = \frac{C_p A_r A_x}{R_p} = \frac{C_p A_r \left(1 + 2\frac{h_x}{h_n}\right)}{R_p}$$

$$\text{calculated max. } S_p = \frac{(1)(1)\left[1 + 2\left(\frac{61}{67}\right)\right]}{2.5} = 1.13$$

←within allowed S_p range of 0.7 through 4

NOTE: Rails and rail brackets are considered rigid components with ductile material and connections. Therefore, C_p = 1.0, A_r = 1.0, and R_p = 2.5 (per NBCC 2010, Table 4.1.8.18, Category 18).

$$\text{max. } F_p = 0.3(1.0)(1.0)(1.0)(1.13)(33\ 361.7) = 11\ 309.6 \text{ N}$$

3-4.2.3 Guide Rail Bracket Design

(a) Per requirement 8.4.8.7 (and Table 8.4.8.7), the guide rail brackets must withstand the seismic loads specified in 8.4.8.2.6. These are summarized, for this case, in Table 8.4.8.7.

(1) To design for deflection, the rail bracket, its fastenings, and any building supports must have a combined deflection of not greater than 0.25 in. with a horizontal seismic load, P, of (see Mandatory Appendix I, Fig. I-8)

$$P = (CB)\frac{2}{3}(F_p) = (1)\frac{2}{3}(11\,309.6) = 7\,539.7 \text{ N}$$

(2) To design for stress, no permanent deformation may result from the combined stresses resulting from the horizontal seismic load, P, of

$$P = (CB)\frac{2}{3}(F_p) = (0.7)\frac{2}{3}(11309.6) = 5\,277.8 \text{ N} \qquad (3\text{-}4.2.3\text{-}1)$$

This force is imposed directly on to the counterweight rail bracket.

ANSI/AISC 360-05, Chapter H, H3.2 states (see Note 1 and Mandatory Appendix I, Fig. I-9)

$$\left(\frac{P_r}{P_c} + \frac{M_r}{M_c}\right) + \left(\frac{V_r}{V_c} + \frac{T_r}{T_c}\right)^2 \le 1.0$$

M_c = allowable flexural strength (as defined in Chapter F)
M_r = required flexural strength
P_c = allowable tensile or compressive strength (as defined in Chapter D or E)
P_r = required axial strength (calculated value)
T_c = allowable torsional strength (as defined in Chapter H)
T_r = required torsional strength
V_c = allowable shear strength (as defined in Chapter G)
V_r = required shear strength

NOTE: See A17.1/B44, Table 8.4.8.7, Note 6.

AISC provides equalities, etc., in terms of allowable strength. A17.1 provides requirements in terms of allowable stress. In generic terms

Allowable Strength Design	Allowable Stress Design
$\frac{\text{Force}_{\text{allow}}}{\Omega} \ge \text{Force}_{\text{required}}$ (calculated force)	$\frac{\text{Stress}_{\text{Yield}}}{\Omega} = \text{Stress}_{\text{allowable}}$

Per H3

$$\Omega = 1.67$$

Therefore

Allowable Strength Design	Allowable Stress Design
$0.6\text{Force}_{\text{allow}} \ge \text{Force}_{\text{required}}$	$0.6\text{Stress}_{\text{Yield}} = \text{Stress}_{\text{allowable}}$

Per Table 8.4.8.7, the bracket force was factored by 0.7 [eq. (3-4.2.3-1)].

Allowable Strength Design	Allowable Stress Design
$0.6\text{Force}_{\text{allow}} \ge 0.7\text{Force}_{\text{required}}$	$0.6\text{Stress}_{\text{Yield}} = 0.7\text{Stress}_{\text{allowable}}$

or

$$0.86\text{Stress}_{\text{Yield}} = \text{Stress}_{\text{allowable}}$$

Note that this is approximately the same stress limit that had been used in previous editions of A17.1 for bending stress in brackets.

3-5 SAMPLE CALCULATION(S) 2: CONTROLLER ANCHORAGE (IMPERIAL UNITS)

The applicable A17.1/B44 code requirements are 8.4(a), 8.4(b), 8.4.14, 8.4.15, and 8.4.2.3.

3-5.1 Sample Calculation 2a (Imperial Units – IBC)

3-5.1.1 Given:

(a) Building installed in jurisdiction where IBC 2006 is in effect.
(b) Latest Safety Code for Elevators and Escalators (ASME A17.1/CSA B44) is also in effect.
(c) I_p = 1.5
(d) S_{DS} = 0.78
(e) Seismic Design Category C
(f) Controller weight = 800 lb
(g) Controller attachment elevation with respect to base, z = 190.5 ft
(h) Average roof height of structure with respect to base, h = 200.5 ft

3-5.1.2 Determination of Proper Seismic Requirements and Force Levels

(a) Per requirement 8.4(a)(1)

Seismic Design Category = C
component importance factor, $I_p = 1.5$

Therefore, Section 8.4 requirements are in effect.

(b) Per requirement 8.4(b)(1), building code references Seismic Design Categories. Therefore, force levels per 8.4.14 are to be used.

(c) Per requirement 8.4.14.1(a)

$$F_p = \text{horizontal force based on SD}$$
$$= \frac{0.4a_pS_{DS}}{\dfrac{R_p}{I_p}}\left[1+2\left(\frac{z}{h}\right)\right]W_p$$

where

a_p = 1.0 (also reference ASCE 7-10, Table 13.6-1)
R_p = 2.5 (also reference ASCE 7-10, Table 13.6-1)
W_p = 800 lbf [per requirement 8.4.15(a)]

$$F_p = \frac{0.4(1)(0.78)}{\dfrac{2.5}{1.5}}\left[1+2\left(\frac{190.5\text{ ft}}{200.5\text{ ft}}\right)\right](800\text{ lbf}) = 434.3\text{ lb}$$

F_p is not required to be greater than

$$F_p = 1.6S_{DS}I_pW_p = 1.6(0.78)800\text{ lbf} = 998\text{ lbf}$$

F_p shall not be taken as less than

$$F_p = 0.3S_{DS}I_pW_p = 0.3(0.78)(1.5)800\text{ lbf} = 281\text{ lbf}$$

Therefore, $F_p = 434.3$ lbf is acceptable.

(d) Per requirement 8.4.14.1.1(a)

$$F_v = \text{vertical force} = \pm 0.2S_{DS}W_p = \pm 0.2(0.78)800\text{ lbf} = \pm 125\text{ lbf}$$

3-5.1.3 Determination of Proper Seismic Loading

Per requirement 8.4.14.1.2, IBC/ASCE 7 basic load combinations for ASD are

$0.6D + 0.7E$ [load combination 8.4.14.1.2(b)]

or

$D + 0.7E$ [load combination 8.4.14.1.2(a)]

whichever is more stringent, where

D = dead load = W_p for this application
E = earthquake load = $F_p + F_v$

The seismic loading to be used will be the most stringent of the four cases outlined in Mandatory Appendix I, Figs. I-1 through I-4.

Maximum tension on the controller anchors will be generated with Case 1 (see Mandatory Appendix I, Fig. I-1).

It is not the purpose of this example to design a specific anchorage of the controller to its supports. Depending on the medium to which the controller is attached, design guidance is given in requirements 8.4.2.3.3(a) through (d). The analysis of the fastening will be based on best engineering practice.

NOTE: ASCE 7-10, Section 13.4.2, Anchors in Concrete or Masonry: Anchors embedded in concrete or masonry shall be proportioned to carry the least of the following:

(a) 1.3 times the force in the component and its supports due to the prescribed forces.

(b) the maximum force that can be transferred to the anchor by the component and its supports. The value of R_p used in Section 13.3.1 to determine the forces in the connected part shall not exceed 1.5 unless

(1) the component anchorage is designed to be governed by the strength of a ductile steel element

(2) the design of the post-installed anchors in concrete used for component anchorage is prequalified for seismic applications in accordance with ACI 355.2

(3) the anchor is designed in accordance with Section 14.2.2.14

3-5.2 Sample Calculation 2b (Imperial Units – NBCC)

3-5.2.1 Given:

(a) Building installed in jurisdiction where NBCC 2005 is in effect.

(b) Latest Safety Code for Elevators and Escalators (ASME A17.1/CSA B44) is also in effect.

(c) $I_E = 1.5$

(d) Site Class C

(e) $S_a(0.2) = 0.98$

(f) $F_a = 1$ (per NBCC Table 4.1.8.4.B)

(g) Controller weight = 800 lb

(h) Controller attachment elevation with respect to base, $z = 190.5$ ft

(i) Average roof height of structure with respect to base, $h = 200.5$ ft

3-5.2.2 Determination of Proper Seismic Requirements and Force Levels

(a) Per requirement 8.4(a)(3)

$$I_EF_aS_a(0.2) = (1.5)(1)(0.98) = 1.47 > 0.35$$

Therefore, Section 8.4 requirements are in effect.

(b) Per requirement 8.4(b)(1), building code references $S_a(0.2)$. Therefore, force levels per 8.4.14 are to be used.

(c) Per requirement 8.4.14.1(b) (and NBCC 2005, 4.1.8.17)

$$F_p = \text{horizontal force based on SD} = 0.3F_aS_a(0.2)I_ES_pW_p$$

NOTE: NBCC 2005, 4.1.8.17 lists F_p as V_p. ASME A171.1/B44 uses the F_p term to maintain a common term for similar IBC/NBCC equations.

(1) Per NBCC 2005, Table 4.1.8.17, Category 11

A_r = 1.0
C_p = 1.0
R_p = 1.25

NOTE: Controllers can be considered machinery that are rigid and rigidly connected. See Note (3) from requirement 8.4.14.1(b).

$$S_p = \frac{C_p A_r A_x}{R_p} = \frac{C_p A_r \left(1+2\frac{h_x}{h_n}\right)}{R_p} = \frac{(1)(1)\left(1+2\frac{190.5}{200.5}\right)}{1.25} = 2.32$$

where S_p minimum allowed = 0.7 and S_p maximum need not be more than 4. Calculated S_p falls within the acceptable range and will be used.

(2) Per requirement 8.4.15(a)

$$W_p = 800 \text{ lbf}$$

Therefore

$$F_p = 0.3F_a S_a(0.2) I_E S_p W_p = 0.3(1)(0.98)(1.5)(2.32)(800 \text{ lbf}) = 818.5 \text{ lbf}$$

(d) Per requirement 8.4.14.1.1(b)

$$F_v = \pm 0.2\left[\frac{2}{3}F_a S_a(0.2)\right]W_p = \pm 0.2\left[\frac{2}{3}(1)(0.98)\right](800 \text{ lbf}) = \pm 104.5 \text{ lbf}$$

3-5.2.3 Determination of Proper Seismic Loading. Per requirement 8.4.14.1.2, IBC/ASCE 7 basic load combinations for ASD are

$0.6D + 0.7E$ [load combination 8.4.14.1.2(b)]

or

$D + 0.7E$ [load combination 8.4.14.1.2(a)]

whichever is more stringent, where

D = dead load = W_p for this application
E = earthquake load = $F_p + F_v$

The seismic loading to be used will be the most stringent of the four cases outlined in Mandatory Appendix I, Figs. I-1 through I-4.

Maximum tension on the controller anchors will be generated with Case 1 (see Mandatory Appendix I, Fig. I-1).

It is not the purpose of this example to design a specific anchorage of the controller to its supports. Depending on the medium to which the controller is attached, design guidance is given in requirements 8.4.2.3.3(a) through (d). The analysis of the fastening will be based on best engineering practice.

3-6 SAMPLE CALCULATION(S) 3: GUIDE RAIL BRACKET SPACING (IMPERIAL UNITS)

The applicable A17.1/B44 code requirements are 8.4(a), 8.4(b), 8.4.8.9, 8.4.12, 8.4.14, and 8.4.15.

3-6.1 Sample Calculation 3a (Imperial Units – IBC)

3-6.1.1 Given:

(a) Building installed in jurisdiction where IBC 2006 is in effect.

(b) Latest Safety Code for Elevators and Escalators (ASME A17.1/CSA B44) is also in effect.

(c) I_p = 1.5

(d) S_{DS} = 0.78

(e) Seismic Design Category C

(f) A standard overhead traction elevator system with

(1) 8,634 lb car weight
(2) 3,500 lb capacity
(3) 472 lb traveling cable weight
(4) 1,038 lb compensation weight
(5) 16 ft car guide overall height (CL lower to CL upper guide)
(6) Overall building height = 200.5 ft
(7) Center of gravity of car at its highest point = 187.5 ft
(8) Center of gravity of car at its lowest point = 7 ft
(9) Center of gravity is located one-third above lower car position restraints.
(10) 15 lb steel car guide rails to be used

3-6.1.2 Determination of Proper Seismic Requirements and Force Levels

(a) Per requirement 8.4(a)(1)

Seismic Design Category C

$$I_p = 1.5$$

Therefore, Section 8.4 requirements are in effect.

(b) Per requirement 8.4(b)(1), building code references Seismic Design Categories. Therefore, force levels per 8.4.14 are to be used.

(c) Per 8.4.14.1(a)

$$F_p = \text{horizontal seismic force based on SD} = \frac{0.4a_p S_{DS}}{\frac{R_p}{I_p}}\left(1+2\frac{z}{h}\right)W_p$$

where

a_p = 1.0 (also reference ASCE 7-10, Table 13.6-1)
R_p = 2.5 (also reference ASCE 7-10, Table 13.6-1)

$$\text{max. } F_p = \frac{0.4(1)(0.78)}{\frac{(2.5)}{(1.5)}}\left(1+2\frac{187.5}{200.5}\right)W_p = 0.537W_p$$

$$\text{min. } F_p = \frac{0.4(1)(0.78)}{\frac{(2.5)}{(1.5)}}\left(1+2\frac{7}{200.5}\right)W_p = 0.200W_p$$

F_p is not required to be greater than

$$F_p = 1.6S_{DS}I_pW_p = 1.6(0.78)(1.5)W_p = 1.872W_p$$

F_p shall not be taken as less than

$$F_p = 0.3S_{DS}I_pW_p = 0.3(0.78)(1.5)W_p = 0.351W_p$$

Then maximum $F_p = 0.537W_p$ is acceptable (within minimum/maximum F_p range). Minimum $F_p = 0.200W_p$ is not acceptable. A minimum $F_p = 0.351W_p$ must be used.

Therefore

$$\text{max. } F_p = 0.506W_p$$

$$\text{min. } F_p = 0.351W_p$$

NOTE: Equating F_p formula with minimum allowed F_p and solving for z will indicate highest point where minimum allowed z will be used.

$$F_p = \frac{0.4(1)(0.78)}{\frac{(2.5)}{(1.5)}}\left(1+2\frac{z}{200.5}\right)W_p = 0.351\,W_p$$

or

$$z = 87.7 \text{ ft}$$

Therefore, the minimum F_p value, $0.351W_p$, will be used for all heights up to 87.7 ft (see Mandatory Appendix I, Fig. I-5). At heights above 87.7 ft, the calculated F_p based on z will be used. The calculated F_p will continue to be used until $z = h$ or the maximum F_p value is reached. (Maximum F_p is not reached in this calculation.)

3-6.1.3 Determination of Seismic Forces for Layouts

(a) Per requirement 8.4.15(b)

$$W_p = \text{car weight} + 40\% \text{ capacity}$$

Insert given values

$$W_p = [8{,}634 + (0.4)(3{,}500)] = 10{,}034 \text{ lbf}$$

Updating maximum and minimum F_p calculated above yields

$$\text{max. } F_p = (0.537)(10{,}034) = 5{,}388.3 \text{ lbf}$$

$$\text{min. } F_p = (0.351)(10{,}034) = 3{,}521.9 \text{ lbf}$$

Therefore

$$\text{max. } F_p = 5{,}388.3 \text{ lbf}$$

$$\text{min. } F_p = 3{,}521.9 \text{ lbf}$$

(b) Per requirement 8.4.8.9, the following force levels are to be shown on elevator layouts (see Mandatory Appendix I, Fig. I-6).

3-6.1.3.1 Requirement 8.4.8.9.1(a). Maximum guide rail force normal to *x-x* axis of guide rail, $F_{x\text{-}x}$

$$F_{x-x} = \frac{2F_p}{3} = \frac{2(5{,}388.3)}{3} = 3{,}592.2 \text{ lbf}$$

3-6.1.3.2 Requirement 8.4.8.9.2(a). Maximum guide rail force normal to *y-y* axis of guide rail, $F_{y\text{-}y}$

$$F_{y-y} = \frac{F_p}{3} = \frac{5{,}388.3}{3} = 1{,}796.1 \text{ lbf}$$

3-6.1.4 Determination of Car Rail Bracket Spacing Based on Seismic Requirements (Section 8.4). These force levels calculated in 3-6.1.3(b) are based on SD. To convert to ASD, IBC allows a factored load, $0.7F_p$ to be used.

A17.1/B44 has already accounted for this factored value as $(0.7F_p)$.

The factored value is used when sizing equipment and determining spacing of rail brackets (in stress calculations). See bending stress calculation section under A17.1/B44, requirement 8.4.12.1 and 3-7, Sample Calculation 7.

(a) *Nomenclature*

- E = modulus of elasticity for steel, $E = 30 \times 10^6$ psi
- F_p = horizontal seismic rail force (strength level)
- I = moment of inertia, in.^{4}
- ℓ = distance between car guide rail brackets, in.
- Z = elastic section modulus, in.^{3}
- Δ = maximum allowable deflection at center of rail span, in. (based on A17.1/B44, Table 8.4.12.2.2, reproduced in Mandatory Appendix I, Table I-1)

(1) Rail Section Properties for 15 lb Rail (See Mandatory Appendix I, Fig. I-7)

$$I_x = 4.78 \text{ in.}^4 \qquad I_y = 5.51 \text{ in.}^4$$

$$Z_x = 1.89 \text{ in.}^3 \qquad Z_y = 2.21 \text{ in.}^3$$

(2) Maximum Allowable Deflection, 15 lb Rail (See Mandatory Appendix I, Table I-1)

$$\Delta = 1.50 \text{ in.}$$

(b) Requirement 8.4.12.1, Maximum Weight Per Pair of Guide Rails

(1) Requirement 8.4.12.1.1(a)(1). Force normal to *x-x* axis of rail (no intermediate tie brackets)

NOTE: ℓ_1 can also be obtained from Fig. 3-6.1.4-1 with $2.93(0.7F_p) = 11{,}051.4$ lb.

$$\ell_1 = 717{,}671\left[\frac{Z_x}{2.93(0.7F_p)}\right] = 717{,}671\left[\frac{1.89}{2.93(0.7\times 5{,}388.3)}\right] = 122.7 \text{ in.}$$

$$\ell_1 = 10.2 \text{ ft} \leftarrow \text{maximum length}$$

(2) Requirement 8.4.12.1.2(a)(1). Force normal to *y-y* axis of rail (no intermediate tie brackets)

$$\ell_2 = 1{,}435{,}342\left[\frac{Z_y}{2.93(0.7F_p)}\right] = 1{,}435{,}342\left[\frac{2.21}{2.93(0.7\times 5{,}388.3)}\right] = 287.0 \text{ in.}$$

$$\ell_2 = 23.9 \text{ ft}$$

(c) Requirement 8.4.12.2, Required Moment of Inertia of Guide Rails

(1) *Requirement 8.4.12.2.1*. Force normal to *x-x* axis of rail

$$\ell_3 = \left(\frac{I_x 249\Delta E}{2F_p}\right)^{1/3} = \left[\frac{(4.78)(249)(1.5)\left(30\times10^6\right)}{(2\times5{,}388.3)}\right]^{1/3}$$
$$= 170.7 \text{ in.}$$

$$\ell_3 = 14.2 \text{ ft}$$

(2) *Requirement 8.4.12.2.2*. Force normal to *y-y* axis of rail

$$\ell_4 = \left(\frac{I_y 498\Delta E}{2F_p}\right)^{1/3} = \left[\frac{(5.51)(498)(1.5)\left(30\times10^6\right)}{(2\times5{,}388.3)}\right]^{1/3}$$
$$= 225.4 \text{ in.}$$

$$\ell_4 = 18.8 \text{ ft}$$

Per seismic requirements, ℓ_1 controls and maximum allowable rail bracket spacing will be 10.2 ft. This same spacing can be found using A17.1/B44, Fig. 8.4.8.2-4 (see Fig. 3-6.1.4-1).

3-6.1.5 Comparison of Car Rail Bracket Spacing Based on Part 2 Rail Requirements (Section 2.23). A17.1/B44, Part 2 rail requirements must also be checked against safety loading.

The shortest rail bracket spacing result from Section 8.4 and Section 2.23 would control the design.

(a) Per requirement 2.23.4.1

total load on safety, W_{safety} = car weight + capacity + traveling cable weight + compensation weight

$$W_{\text{safety}} = 8{,}634 + 3{,}500 + 472 + 1{,}038 = 13{,}644 \text{ lb}$$

The allowed bracket spacing is interpolated from Fig. 3-6.1.5-1.

For 15 lb Rail
15,419 lb safety load has maximum bracket spacing of 9.84 ft
11,989 lb safety load has maximum bracket spacing of 14.104 ft

$$\left(\frac{13{,}644 \text{ lb} - 15{,}419 \text{ lb}}{11{,}989 \text{ lb} - 15{,}419 \text{ lb}}\right) = \left(\frac{\ell_{\text{Section 2.23}} - 9.84 \text{ ft}}{14.104 \text{ ft} - 9.84 \text{ ft}}\right)$$

$$\ell_{\text{Section 2.23}} = 12.05 \text{ ft} > \ell_1 = 10.2 \text{ ft}$$

Per Section 2.23, maximum allowable rail spacing is 12.05 ft.

Therefore, Section 8.4 bracket spacing controls and maximum bracket spacing allowed is 10.2 ft. This same spacing can be found using A17.1/B44, Fig. 2.23.4.1-1 (see Fig. 3-6.1.5-1).

3-6.1.6 Section 2.23 Versus Section 8.4 Control of Design: Additional Example. For comparison, the bracket spacing for the minimum F_p force will be found

$$\text{for } F_p = 0.351W_p = 3{,}521.9 \text{ lb}$$

(a) Requirement 8.4.12.1, Maximum Weight Per Pair of Guide Rails

(1) *Requirement 8.4.12.1.1(a)(1)*. Force normal to *x-x* axis of rail (no intermediate tie brackets)

$$\ell_1 = 717{,}671\left[\frac{Z_x}{2.93\left(0.7F_p\right)}\right] = 717{,}671\left[\frac{1.89}{2.93(0.7\times3{,}521.9)}\right]$$
$$= 187.8 \text{ in.}$$
$$\ell_1 = 15.6 \text{ ft} \leftarrow \text{maximum length}$$

(2) *Requirement 8.4.12.1.2(a)(1)*. Force normal to *y-y* axis of rail (no intermediate tie brackets)

$$\ell_2 = 1{,}435{,}342\left[\frac{Z_y}{2.93\left(0.7F_p\right)}\right] = 1{,}435{,}342\left[\frac{2.21}{2.93(0.7\times3{,}521.9)}\right]$$
$$= 439.1 \text{ in.}$$
$$\ell_2 = 36.6 \text{ ft}$$

(b) Requirement 8.4.12.2, Required Moment of Inertia of Guide Rails

(1) *Requirement 8.4.12.2.1*. Force normal to *x-x* axis of rail

$$\ell_3 = \left(\frac{I_x 249\Delta E}{2F_p}\right)^{1/3} = \left[\frac{(4.78)(249)(1.5)\left(30\times10^6\right)}{(2\times3{,}521.9)}\right]^{1/3}$$
$$= 196.6 \text{ in.}$$

$$\ell_3 = 16.4 \text{ ft}$$

(2) *Requirement 8.4.12.2.2*. Force normal to *y-y* axis of rail

$$\ell_4 = \left(\frac{I_y 498\Delta E}{2F_p}\right)^{1/3} = \left[\frac{(5.51)(498)(1.5)\left(30\times10^6\right)}{(2\times3{,}521.9)}\right]^{1/3}$$
$$= 259.8 \text{ in.}$$

$$\ell_4 = 21.6 \text{ ft}$$

Per seismic requirements, maximum rail bracket spacing will be 15.6 ft. Comparing this to the bracket spacing found for Section 2.23

$$\ell_{\text{Section 2.23}} = 12.05 \text{ ft} < \ell_1 = 15.6 \text{ ft}$$

For the minimum F_p, the bracket spacing found in Section 2.23 controls the design.

Fig. 3-6.1.4-1 A17.1/B44, Fig. 8.4.8.2-4, 22.5 kg/m (15 lb/ft) Guide-Rail Bracket Spacing (Marked for Sample Calculation 3a)

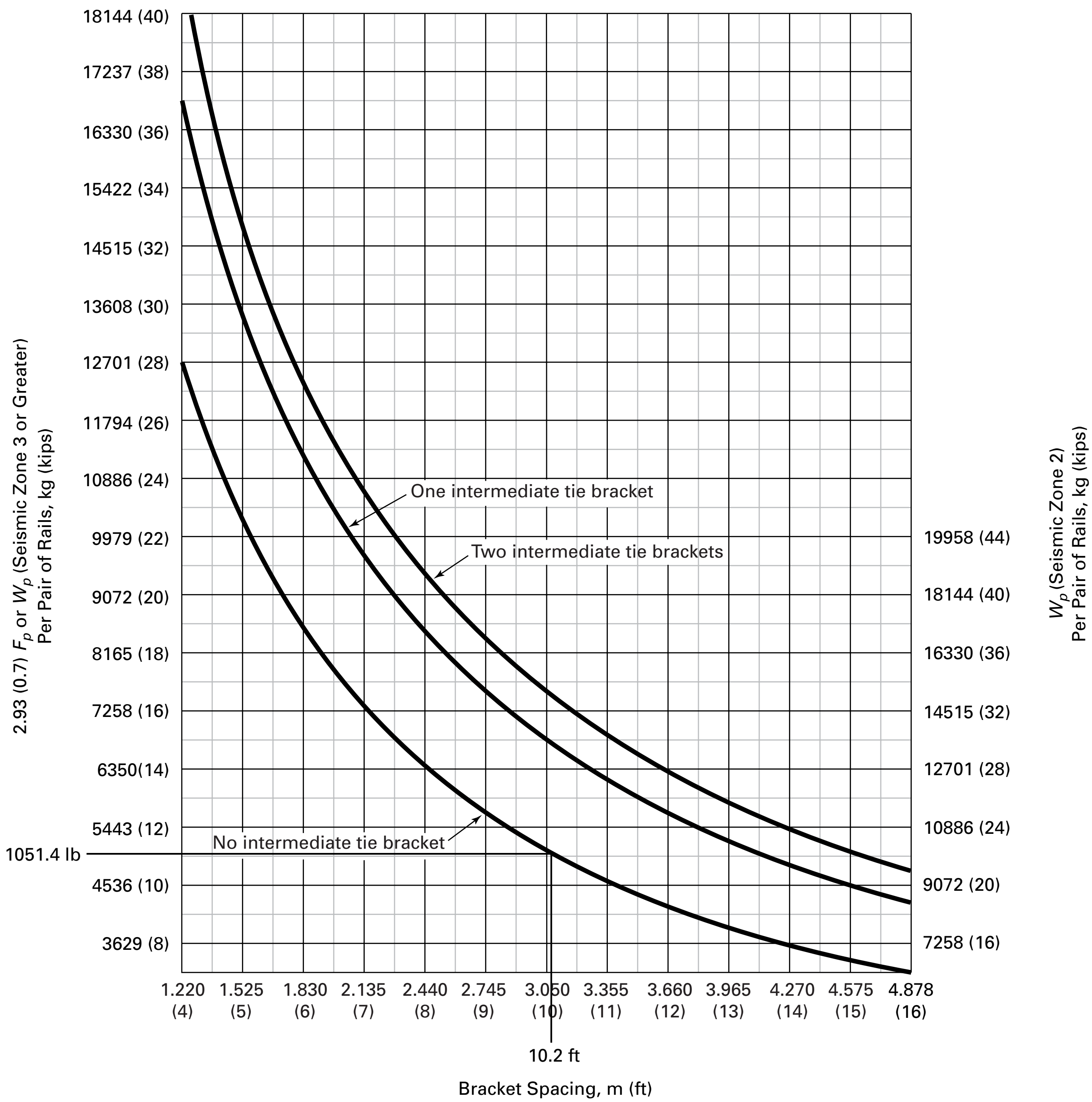
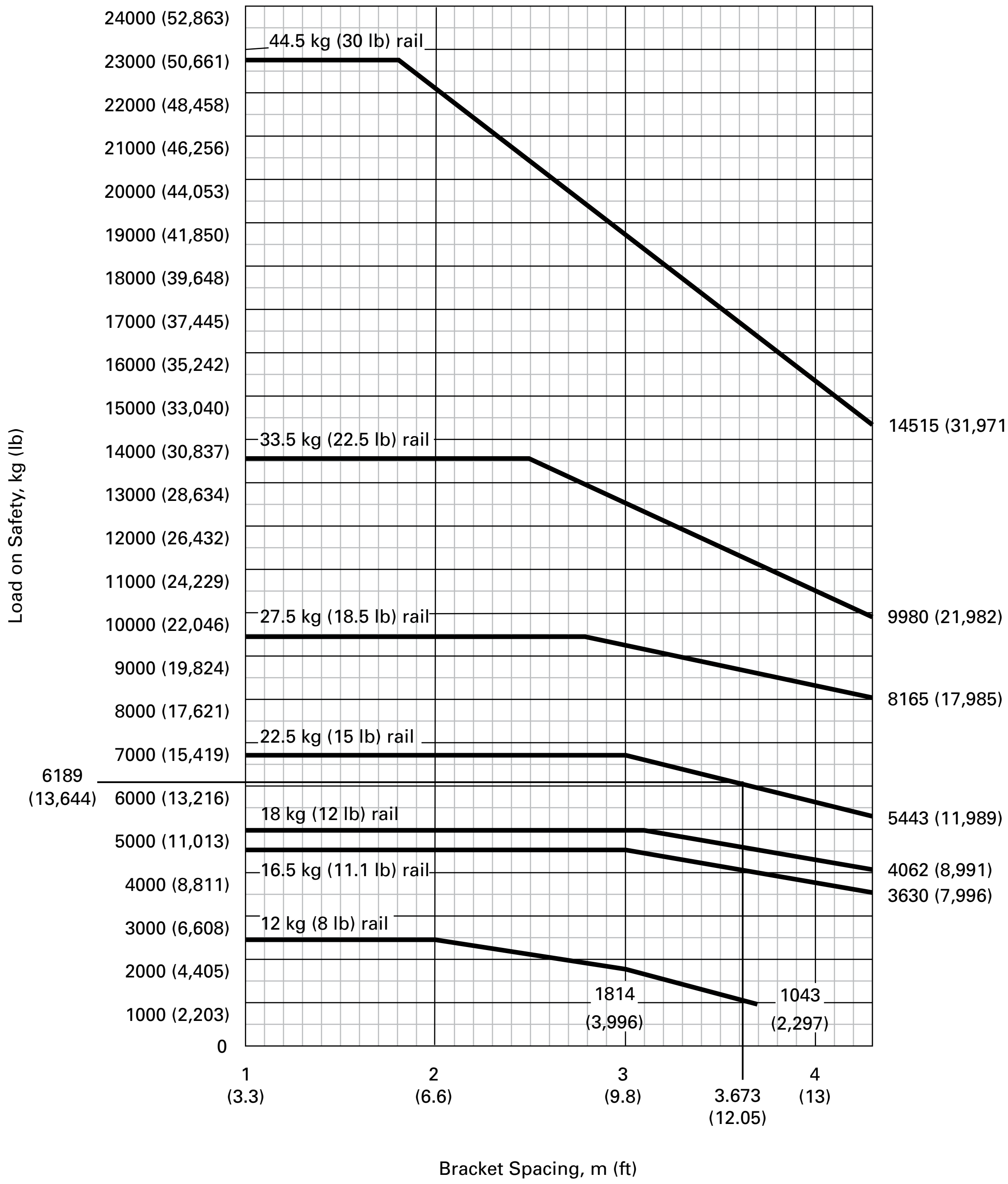


Fig. 3-6.1.5-1 A17.1/B44, Fig. 2.23.4.1-1 (Marked for Sample Calculation 3a)



3-6.2 Sample Calculation 3b (Imperial Units – NBCC)

3-6.2.1 Given:

(a) Building installed in jurisdiction where NBCC 2005 is in effect.

(b) Latest Safety Code for Elevators and Escalators (ASME A17.1/CSA B44) is also in effect.

(c) $I_E = 1.5$

(d) Site Class C

(e) $S_a(0.2) = 0.98$

(f) $F_a = 1$ (per NBCC Table 4.1.8.4.B)

(g) A standard overhead traction elevator system with

(1) 8,634 lb car weight

(2) 3,500 lb capacity

(3) 472 lb traveling cable weight

(4) 1,038 lb compensation weight

(5) 16 ft car guide overall height (CL lower to CL upper guide)

(6) Overall building height = 200.5 ft

(7) Center of gravity of car at its highest point = 187.5 ft

(8) Center of gravity of car at its lowest point = 7 ft

(9) Center of gravity is located one-third above lower car position restraints

(10) 15 lb/ft steel car guide rails to be used

3-6.2.2 Determination of Proper Seismic Requirements and Force Levels

(a) Per requirement 8.4(a)(3)

$$I_E F_a S_a(0.2) = (1.5)(1)(0.98) = 1.47 > 0.35$$

Therefore, Section 8.4 requirements are in effect.

(b) Per requirement 8.4(b)(1), building code references $S(0.2)$ values. Therefore, force levels per 8.4.14 are to be used.

(c) Per 8.4.14.1(b) (and NBCC 2005, 4.1.8.17)

$$\begin{aligned} F_p &= \text{horizontal seismic force based on SD} \\ &= 0.3F_a S_a(0.2) I_E S_p W_p \end{aligned}$$

NOTE: NBCC 2010, 4.1.8.18 lists F_p as V_p. ASME A171.1/B44 uses the F_p term to maintain a common term for similar IBC/NBCC equations.

(1) Per NBCC 2005, Table 4.1.8.17, Category 18

$A_r = 1.0$
$C_p = 1.0$
$R_p = 2.5$

NOTE: Rails and rail brackets are considered rigid components with ductile material and connections.

$$S_p = \frac{C_p A_r A_x}{R_p} = \frac{C_p A_r \left(1 + 2\dfrac{h_x}{h_n}\right)}{R_p}$$

where S_p minimum allowed = 0.7 and S_p maximum need not be more than 4.

Maximum S_p will be taken at the highest car position.

Minimum S_p will be taken at the lowest car position.

$$\text{calculated max. } S_p = \frac{(1)(1)\left[1 + 2\left(\dfrac{187.5}{200.5}\right)\right]}{2.5}$$

$$= 1.15 \leftarrow \text{within allowed } S_p \text{ range}$$

$$\text{calculated min. } S_p = \frac{(1)(1)\left[1 + 2\left(\dfrac{7}{200.5}\right)\right]}{2.5}$$

$$= 0.43 \leftarrow \text{outside allowed } S_p \text{ range}$$

Calculated max. S_p is within allowed range

Calculated min. S_p is below minimum allowed.

Therefore, minimum S_p used will be 0.7.

NOTE: Equating S_p formula with minimum allowed S_p and solving for h_x will indicate the highest point where minimum allowed S_p will be used.

$$\frac{(1)(1)\left[1 + 2\left(\dfrac{h_x}{h_n}\right)\right]}{2.5} = 0.7$$

or

$$h_x = 0.375 h_n$$

(This constraint is true for all rigid components with ductile material.)

$$h_x = 0.375(200.5 \text{ ft}) = 75.19 \text{ ft}$$

Then the minimum S_p will be used for all heights up to 75.19 ft.

Inserting values for F_a, $S_a(0.2)$, I_E, and min./max. F_p yields

$$\text{max. } F_p = 0.3(1)(0.98)(1.5)(1.15)W_p = 0.507W_p$$

$$\text{min. } F_p = 0.3(1)(0.98)(1.5)(0.7)W_p = 0.309W_p$$

Therefore

$$\text{max. } F_p = 0.507W_p$$
$$\text{min. } F_p = 0.309W_p$$

3-6.2.3 Determination of Seismic Forces for Layouts

(a) Per requirement 8.4.15(b)

$$W_p = \text{car weight} + 40\% \text{ capacity}$$

Insert given values

$$W_p = [8{,}634 + (0.4)(3{,}500)] = 10{,}034 \text{ lbf}$$

Updating maximum and minimum F_p yields

$$\text{max. } F_p = (0.507)(10{,}034) = 5{,}087.2 \text{ lb}$$

$$\text{min. } F_p = (0.309)(10{,}034) = 3{,}100.5 \text{ lb}$$

Therefore

$$\text{max. } F_p = 5{,}087.2 \text{ lb}$$

$$\text{min. } F_p = 3{,}100.5 \text{ lb}$$

(b) Per requirement 8.4.8.9, the following force levels are to be shown on elevator layouts (see Mandatory Appendix I, Fig. I-6).

(1) Requirement 8.4.8.9.1(a). Maximum guide rail force normal to *x-x* axis of guide rail, $F_{x\text{-}x}$

$$F_{x\text{-}x} = \frac{2F_p}{3} = \frac{2(5{,}087.2)}{3} = 3{,}391.5 \text{ lbf}$$

(2) Requirement 8.4.8.9.2(a). Maximum guide rail force normal to *y-y* axis of guide rail, $F_{y\text{-}y}$

$$F_{y\text{-}y} = \frac{F_p}{3} = \frac{(5{,}087.2)}{3} = 1{,}695.7 \text{ lbf}$$

Therefore

$$F_{x\text{-}x} = 3{,}391.5 \text{ lbf}$$

$$F_{y\text{-}y} = 1{,}695.7 \text{ lbf}$$

3-6.2.4 Determination of Car Rail Bracket Spacing Based on Seismic Requirements (Section 8.4). The force levels calculated in 3-6.2.7 are based on SD. To convert to ASD, IBC allows a factored load, $0.7F_p$ to be used. This same factored load will be used for NBCC to convert to ASD.

A17.1/B44 has already accounted for this factored value as ($0.7F_p$).

The factored value is used when sizing equipment and determining spacing of rail brackets (in stress calculations). See bending stress calculation section under A17.1/B44, requirement 8.4.12.1 and 3-7, Sample Calculation 4.

(a) Nomenclature

E = modulus of elasticity for steel, $E = 30 \times 10^6$ psi
F_p = horizontal seismic rail force (strength level)
I = moment of inertia, in.^{4}
ℓ = distance between car guide rail brackets, in.
Z = elastic section modulus, in.^{3}
Δ = maximum allowable deflection at center of rail span, in. (based on Table 8.4.12.2.2)

(1) Rail Section Properties for 15 lb/lb Rail (See Mandatory Appendix I, Fig. I-7)

$I_x = 4.78$ in.^{4} $\qquad$ $I_y = 5.51$ in.^{4}

$Z_x = 1.89$ in.^{3} $\qquad$ $Z_y = 2.21$ in.^{3}

(2) Maximum Allowable Deflection, 15 lb Rail (See Mandatory Appendix I, Table I-1)

$$\Delta = 1.50 \text{ in.}$$

(b) Requirement 8.4.12.1, Maximum Weight Per Pair of Guide Rails

(1) Requirement 8.4.12.1.1(a)(1). Force normal to *x-x* axis of rail (no intermediate tie brackets)

NOTE: ℓ_1 can also be obtained from A17.1/B44, Fig. 8.4.8.2-4 with $2.93(0.7F_p)$. See Fig. 3-3.2.4.1.

$$\ell_1 = 717{,}671\left[\frac{Z_x}{2.93\left(0.7F_p\right)}\right]$$

$$= 717{,}671\left[\frac{1.89}{2.93(0.7\times 5{,}087.2)}\right] = 130 \text{ in.}$$

$$\ell_1 = 10.8 \text{ ft} \leftarrow \text{maximum length}$$

(2) Requirement 8.4.12.1.2(a)(1). Force normal to *y-y* axis of rail (no intermediate tie brackets)

$$\ell_2 = 1{,}435{,}342\left[\frac{Z_y}{2.93\left(0.7F_p\right)}\right]$$

$$= 1{,}435{,}342\left[\frac{2.21}{2.93(0.7\times 5{,}087.2)}\right] = 304 \text{ in.}$$

$$\ell_2 = 25.3 \text{ ft}$$

(c) Requirement 8.4.12.2, Required Moment of Inertia of Guide Rails

(1) Requirement 8.4.12.2.1. Force normal to *x-x* axis of rail

$$\ell_3 = \left(\frac{I_x 249\Delta E}{2F_p}\right)^{1/3} = \left[\frac{(4.78)(249)(1.5)\left(30\times 10^6\right)}{(2\times 5{,}087.2)}\right]^{1/3} = 174 \text{ in.}$$

$$\ell_3 = 14.5 \text{ ft}$$

(2) Requirement 8.4.12.2.2. Force normal to *y-y* axis of rail

$$\ell_4 = \left(\frac{I_y 498\Delta E}{2F_p}\right)^{1/3} = \left[\frac{(5.51)(498)(1.5)\left(30\times 10^6\right)}{(2\times 5{,}087.2)}\right]^{1/3}$$

$$= 229.8 \text{ in.}$$

$$\ell_4 = 19.2 \text{ ft}$$

Per seismic requirements, ℓ_1 controls, maximum allowable rail bracket spacing is 10.8 ft. This same spacing can be found using A17.1/B44, Fig. 8.4.8.2-4 (see Fig. 3-3.2.4.1).

3-6.2.5 Comparison of Car Rail Bracket Spacing Based on Part 2 Rail Requirements (Section 2.23). A17.1/B44, Part 2 rail requirements must also be checked against safety loading.

The shortest rail bracket spacing result from Section 8.4 and Section 2.23 would control the design.

(a) Per requirement 2.23.4.1

total load on safety, W_{safety} = car weight + capacity + traveling cable weight + compensation weight

$$W_{\text{safety}} = 8{,}634 + 3{,}500 + 472 + 1{,}038 = 13{,}644 \text{ lb}$$

The allowed bracket spacing is interpolated from Fig. 3-3.1.5.1.

For 15 lb/ft Rail
15,419 lb safety load has maximum bracket spacing of 9.84 ft (or 3 m)
11,989 lb safety load has maximum bracket spacing of 14.104 ft (or 4.3 m)

$$\left(\frac{13{,}644\text{ lb}-15{,}419\text{ lb}}{11{,}989\text{ lb}-15{,}419\text{ lb}}\right)=\left(\frac{\ell_{\text{Section 2.23}}-9.84\text{ ft}}{14.104\text{ ft}-9.84\text{ ft}}\right)$$

$$\ell_{\text{Section 2.23}} = 12.05\text{ ft} > \ell_1 = 10.8\text{ ft}$$

Therefore, Section 8.4 bracket spacing controls and maximum bracket spacing allowed is 10.8 ft. This same spacing can be found using A17.1/B44, Fig. 2.23.4.1-1 (see Fig. 3-3.1.5.1).

3-6.2.6 Section 2.23 Versus Section 8.4 Control of Design: Additional Example. For comparison, the bracket spacing for the minimum F_p force will be found

$$\text{for } F_p = 0.309\ W_p = 3{,}100.5\text{ lb}$$

(a) Requirement 8.4.12.1, Maximum Weight Per Pair of Guide Rails

(1) Requirement 8.4.12.1.1(a)(1). Force normal to *x-x* axis of rail (no intermediate tie brackets)

$$\ell_1 = 717{,}671\left[\frac{Z_x}{2.93\left(0.7F_p\right)}\right] = 717{,}671\left[\frac{1.89}{2.93(0.7\times 3{,}100.5)}\right]$$

$$= 213.3\text{ in.}$$

$$\ell_1 = 17.8\text{ ft} \leftarrow \text{maximum length}$$

(2) Requirement 8.4.12.1.2(a)(1). Force normal to *y-y* axis of rail (no intermediate tie brackets)

$$\ell_2 = 1{,}435{,}342\left[\frac{Z_y}{2.93\left(0.7F_p\right)}\right] = 1{,}435{,}342\left[\frac{2.21}{2.93(0.7\times 3{,}100.5)}\right]$$

$$= 498.8\text{ in.}$$

$$\ell_2 = 41.6\text{ ft}$$

(b) Requirement 8.4.12.2, Required Moment of Inertia of Guide Rails

(1) Force normal to *x-x* axis of rail

$$\ell_3 = \left(\frac{I_x 249\Delta E}{2F_p}\right)^{1/3} = \left[\frac{(4.78)(249)(1.5)\left(30\times 10^6\right)}{(2\times 3{,}100.5)}\right]^{1/3}$$

$$= 205.2\text{ in.}$$

$$\ell_3 = 17.1\text{ ft}$$

(2) Force normal to *y-y* axis of rail

$$\ell_4 = \left(\frac{I_y 498\Delta E}{2F_p}\right)^{1/3} = \left[\frac{(5.51)(498)(1.5)\left(30\times 10^6\right)}{(2\times 3{,}100.5)}\right]^{1/3}$$

$$= 271.0\text{ in.}$$

$$\ell_4 = 22.6\text{ ft}$$

Per seismic requirements, maximum rail bracket spacing will be 17.8 ft. Comparing this to the bracket spacing found for Part 2

$$\ell_{\text{Section 2.23}} = 12.05\text{ ft} < \ell_1 = 17.8\text{ ft}$$

For the minimum F_p, the bracket spacing found in Section 2.23 controls the design.

3-7 SAMPLE CALCULATION(S) 4: GUIDE RAIL BRACKET STRENGTH AND DESIGN (IMPERIAL UNITS)

The applicable A17.1/B44 code requirements are 8.4(a), 8.4(b), 8.4.8.7, 8.4.12, 8.4.14, and 8.4.15.

3-7.1 Sample Calculation 4a (Imperial Units – IBC)

3-7.1.1 Given:

(a) Building installed in jurisdiction where IBC 2006 is in effect.

(b) Latest Safety Code for Elevators and Escalators (ASME A17.1/CSA B44) is also in effect.

(c) $I_p = 1.0$

(d) $S_{DS} = 0.75$

(e) Seismic Design Category D

(f) Counterweight weight = 7,500 lb

(g) Counterweight is two-thirds full

(h) Distance between upper and lower position restraints is greater than rail bracket span, $L > \ell$

(i) Center of gravity of counterweight at its highest point, $z = 200$ ft

(j) Average roof height of structure with respect to base, $h = 220$ ft

3-7.1.2 Determination of Proper Seismic Requirements and Force Levels

(a) Per requirement 8.4(a)(1)

$$\text{Seismic Design Category} = \text{D}$$

$$\text{component importance factor, } I_p = 1.0$$

Therefore, Section 8.4 requirements are in effect.

(b) Per requirement 8.4(b)(1), building code references Seismic Design Categories. Therefore, force levels per 8.4.14 are to be used.

(c) Per requirement 8.4.14.1(a)

F_p = horizontal force based on SD

$$= \frac{0.4 a_p S_{DS}}{\frac{R_p}{I_p}} \left[1 + 2\left(\frac{z}{h}\right)\right] W_p$$

where

a_p = 1.0
R_p = 2.5
W_p = 7,500 lbf [per requirement 8.4.15(a)]

$$\text{max. } F_p = \frac{0.4(1)(0.75)}{\frac{2.5}{1.0}} \left[1 + 2\left(\frac{200 \text{ ft}}{220 \text{ ft}}\right)\right](7{,}500 \text{ lbf}) = 2{,}536.4 \text{ lbf}$$

3-7.1.3 Guide Rail Bracket Design

(a) Per requirement 8.4.8.7 (and Table 8.4.8.7), the guide rail brackets must withstand the seismic loads specified in 8.4.8.2.6. These are summarized, for this case, in Table 8.4.8.7.

(1) To design for deflection, the rail bracket, its fastenings, and any building supports must have a combined deflection of not greater than 0.25 in. with a horizontal seismic load, P, of (see Mandatory Appendix I, Fig. I-8)

$$P = (CB)\frac{2}{3}\left(F_p\right) = (1)\frac{2}{3}(2{,}536.4) = 1{,}691 \text{ lbf}$$

(2) To design for stress, no permanent deformation may result from the combined stresses resulting from the horizontal seismic load, P, of

$$P = (CB)\frac{2}{3}\left(F_p\right) = (0.7)\frac{2}{3}(2{,}536.4) = 1{,}183.7 \text{ lbf} \qquad \text{(3-7.1.3-1)}$$

This force is imposed directly on to the counterweight rail bracket.

ANSI/AISC 360-05, Chapter H, H3.2 states (see Note 1 and Mandatory Appendix I, Fig. I-9)

$$\left(\frac{P_r}{P_c} + \frac{M_r}{M_c}\right) + \left(\frac{V_r}{V_c} + \frac{T_r}{T_c}\right)^2 \le 1.0$$

M_c = allowable flexural strength (as defined in Chapter F)
M_r = required flexural strength
P_c = allowable tensile or compressive strength (as defined in Chapter D or E)
P_r = required axial strength (calculated value)
T_c = allowable torsional strength (as defined in Chapter H)
T_r = required torsional strength
V_c = allowable shear strength (as defined in Chapter G)
V_r = required shear strength

NOTE: See A17.1/B44, Table 8.4.8.7, Note 6.

AISC provides equalities, etc., in terms of allowable strength. A17.1 provides requirements in terms of allowable stress. In generic terms

Allowable Strength Design	Allowable Stress Design
$\frac{\text{Force}_{\text{allow}}}{\Omega} \ge$	$\frac{\text{Stress}_{\text{Yield}}}{\Omega} =$
$\text{Force}_{\text{required}}$ (calculated force)	$\text{Stress}_{\text{allowable}}$

Per H3

$$\Omega = 1.67$$

Therefore

Allowable Strength Design	Allowable Stress Design
$0.6\text{Force}_{\text{allow}} \ge \text{Force}_{\text{required}}$	$0.6\text{Stress}_{\text{Yield}} = \text{Stress}_{\text{allowable}}$

Per Table 8.4.8.7, the bracket force was factored by 0.7 [eq. (3-7.1.3-1)].

Allowable Strength Design	Allowable Stress Design
$0.6\text{Force}_{\text{allow}} \ge 0.7\text{Force}_{\text{required}}$	$0.6\text{Stress}_{\text{Yield}} = 0.7\text{Stress}_{\text{allowable}}$

or

$$0.86\text{Stress}_{\text{Yield}} = \text{Stress}_{\text{allowable}}$$

Note that this is approximately the same stress limit that had been used in previous editions of A17.1 for bending stress in brackets.

3-7.2 Sample Calculation 4b (Imperial Units – NBCC)

3-7.2.1 Given:

(a) Building installed in jurisdiction where NBCC 2005 is in effect.

(b) Latest Safety Code for Elevators and Escalators (ASME A17.1/CSA B44) is also in effect.

(c) I_E = 1.0

(d) Site Class C

(e) $S_a(0.2)$ = 1.0

(f) F_a = 1 (per NBCC Table 4.1.8.4.B)

(g) Counterweight weight = 7,500 lb

(h) Counterweight is two-thirds full

(i) Distance between upper and lower position restraints is greater than rail bracket span, $L > \ell$

(j) Center of gravity of counterweight at its highest point, z = 200 ft

(k) Average roof height of structure with respect to base, h = 220 ft

3-7.2.2 Determination of Proper Seismic Requirements and Force Levels

(a) Per requirement 8.4(a)(3)

$$I_E F_a S_a(0.2) = (1.0)(1)(1) = 1.0 > 0.35$$

Therefore, Section 8.4 requirements are in effect.

(b) Per requirement 8.4(b)(1), building code references S(0.2) values. Therefore, force levels per 8.4.14 are to be used.

(c) Per 8.4.14.1(b) (and NBCC 2005,4.1.8.18)

$$F_p = \text{horizontal seismic force based on SD} = 0.3F_a S_a(0.2) I_E S_p W_p$$

NOTE: NBCC 2005, 4.1.8.18 lists F_p as V_p. ASME A171.1/B44 uses the F_p term to maintain a common term for similar IBC/NBCC equations.

where

W_p = 7,500 lbf [per requirement 8.4.15(a)]

and F_a, S_a(0.2), and I_E are provided above.

$$S_p = \frac{C_p A_r A_x}{R_p} = \frac{C_p A_r \left(1 + 2\frac{h_x}{h_n}\right)}{R_p}$$

$$\text{calc. max. } S_p = \frac{(1)(1)\left[1 + 2\left(\frac{200}{220}\right)\right]}{2.5} = 1.13$$

← within allowed S_p range of 0.7 through 4.

NOTE: Rails and rail brackets are considered rigid components with ductile material and connections. Therefore, C_p = 1.0, A_r = 1.0, and R_p = 2.5 (per NBCC 2005, Table 4.1.8.18, Category 18).

$$\text{max. } F_p = 0.3(1.0)(1.0)(1.0)(1.13)(7{,}500) = 2{,}542.5 \text{ lbf}$$

3-7.2.3 Guide Rail Bracket Design

(a) Per requirement 8.4.8.7 (and Table 8.4.8.7), the guide rail brackets must withstand the seismic loads specified in 8.4.8.2.6. These are summarized, for this case, in Table 8.4.8.7.

(1) To design for deflection, the rail bracket, its fastenings, and any building supports must have a combined deflection of not greater than 0.25 in. with a horizontal seismic load, P, of (see Mandatory Appendix I, Fig. I-8)

$$P = (CB)\frac{2}{3}\left(F_p\right) = (1)\frac{2}{3}(2{,}542.5) = 1{,}695 \text{ lbf}$$

(2) To design for stress, no permanent deformation may result from the combined stresses resulting from the horizontal seismic load, P, of

$$P = (CB)\frac{2}{3}\left(F_p\right) = (0.7)\frac{2}{3}(2{,}536.4) = 1{,}183.7 \text{ lbf} \quad \text{(3-7.2.3-1)}$$

This force is imposed directly on to the counterweight rail bracket.

ANSI/AISC 360-05, Chapter H, H3.2 states (see Note 1 and Mandatory Appendix I, Fig. I-9)

$$\left(\frac{P_r}{P_c} + \frac{M_r}{M_c}\right) + \left(\frac{V_r}{V_c} + \frac{T_r}{T_c}\right)^2 \le 1.0$$

M_c = allowable flexural strength (as defined in Chapter F)
M_r = required flexural strength
P_c = allowable tensile or compressive strength (as defined in Chapter D or E)
P_r = required axial strength (calculated value)
T_c = allowable torsional strength (as defined in Chapter H)
T_r = required torsional strength
V_c = allowable shear strength (as defined in Chapter G)
V_r = required shear strength

NOTE: See A17.1/B44, Table 8.4.8.7, Note 6.

AISC provides equalities, etc., in terms of allowable strength. A17.1 provides requirements in terms of allowable stress. In generic terms

Allowable Strength Design	Allowable Stress Design
$\frac{\text{Force}_{\text{allow}}}{\Omega} \ge$	$\frac{\text{Stress}_{\text{Yield}}}{\Omega} =$
$\text{Force}_{\text{required}}$ (calculated force)	$\text{Stress}_{\text{allowable}}$

Per H3

$$\Omega = 1.67$$

Therefore

Allowable Strength Design	Allowable Stress Design
$0.6\text{Force}_{\text{allow}} \ge \text{Force}_{\text{required}}$	$0.6\text{Stress}_{\text{Yield}} = \text{Stress}_{\text{allowable}}$

Per Table 8.4.8.7, the bracket force was factored by 0.7 [eq. (3-7.2.3-1)].

Allowable Strength Design	Allowable Stress Design
$0.6\text{Force}_{\text{allow}} \ge 0.7\text{Force}_{\text{required}}$	$0.6\text{Stress}_{\text{Yield}} = 0.7\text{Stress}_{\text{allowable}}$

or

$$0.86\text{Stress}_{\text{Yield}} = \text{Stress}_{\text{allowable}}$$

Note that this is approximately the same stress limit that had been used in previous editions of A17.1 for bending stress in brackets.

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Mandatory Appendix I
Sample Calculation Figures

This Mandatory Appendix contains a table and figures to be used in conjunction with Part 3 of this Guide.

Fig. I-1 Case 1, Load Eq. 8.4.14.1.2(b): Seismic Loading (F_v in the UP Direction)

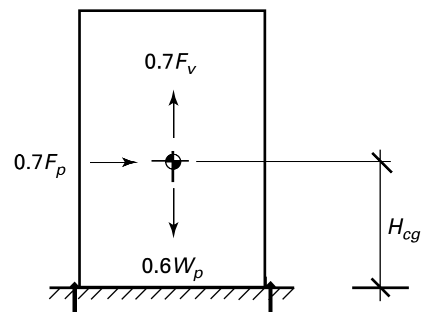

Fig. I-2 Case 2, Load Eq. 8.4.14.1.2(b): Seismic Loading (F_v in the DOWN Direction)

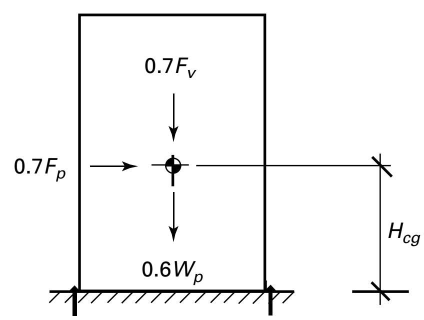

Fig. I-3 Case 3, Load Eq. 8.4.14.1.2(a): Seismic Loading (F_v in the UP Direction)

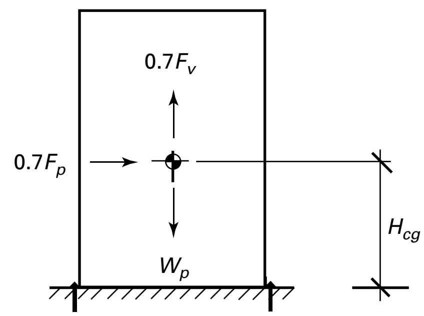

Fig. I-4 Case 4, Load Eq. 8.4.14.1.2(a): Seismic Loading (F_v in the DOWN Direction)

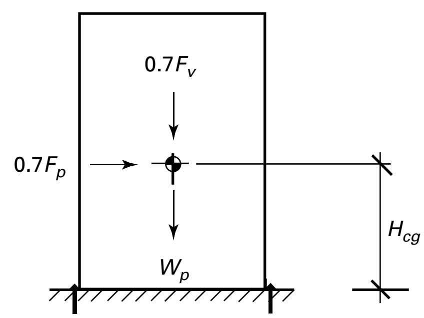

Fig. I-5 Pictorial View of F_p Forces

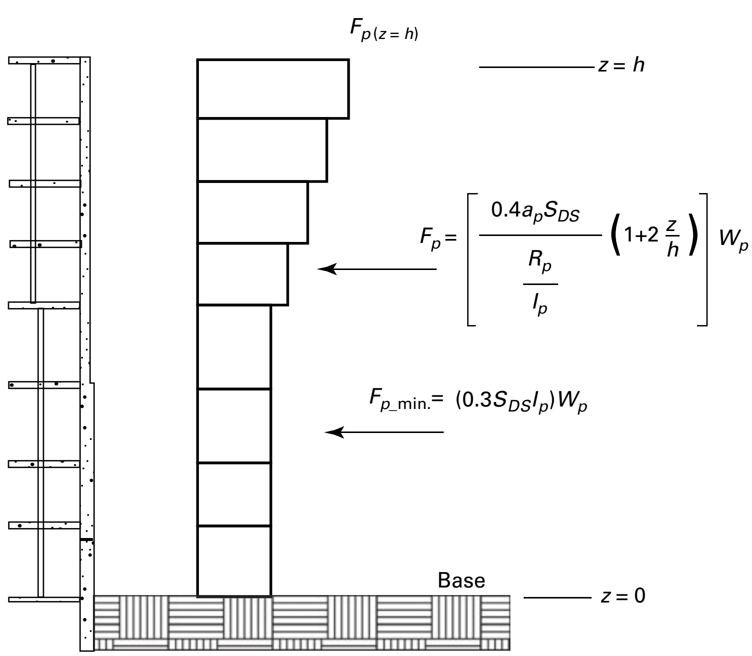

Fig. I-6 Elevator Guide Rail Force Orientations

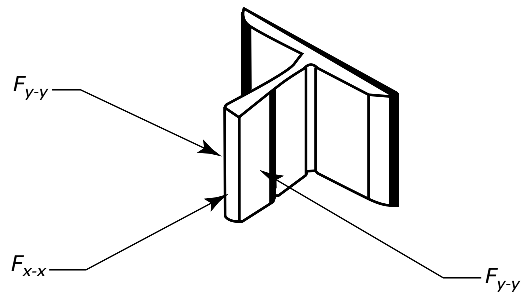

Fig. I-7 A17.1/B44, Fig. 8.4.8.9, Guide Rail Axes

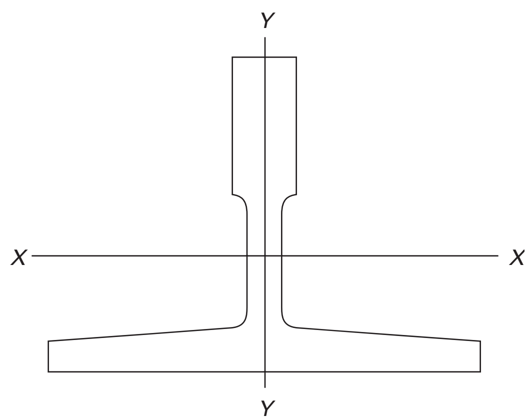

Fig. I-8 Seismic Rail Loading Force for Counterweight

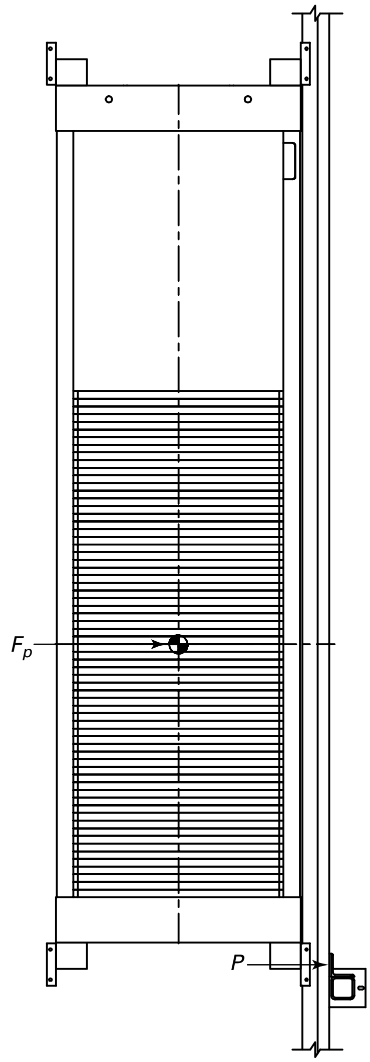

Fig. I-9 Rail Bracket Free Body and Bending Moment Diagram

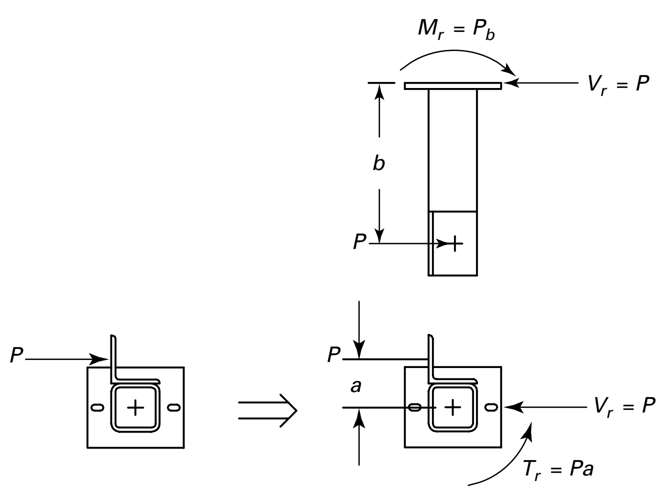

Table I-1 A17.1/B44, Table 8.4.12.2.2, Maximum Allowable Deflection

Rail Size, kg (lb)	Δ, Max., mm (in.)
12.0 (8.0)	20 (0.75)
16.5 (11.0)	25 (1.00)
18.0 (12.0)	32 (1.25)
22.5 (15.0)	38 (1.50)
27.5 (18.5)	38 (1.50)
33.5 (22.5)	38 (1.50)
45.0 (30.0)	45 (1.75)

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ASME A17.1-8.4–2013

ISBN 978-0-7918-6878-2

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