Industrial Building Design — Seismic Issues

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The current seismic design provisions in the United States and Canada were written predominantly to address commercial and institutional buildings. Industrial buildings have geometries, framing systems, mass characteristics, load types and magnitudes, and stiffness properties that may vary significantly from those of typical commercial or institutional buildings. These characteristics may influence the expected performance of these buildings when subject to a seismic event. The normal design procedures presented in the building codes for both the United States and Canada do not fully acknowledge these differences, creating uncertainty and problems for the engineer designing these facilities.

This paper discusses current seismic provisions for the design and construction of steel-framed industrial buildings. Also discussed are current design codes for, and the design of, nonbuilding structures often contained within these facilities.

Part One of this paper will focus on seismic design for industrial buildings. This will include:

- A brief description of current seismic building design procedures in the United States and Canada. This discussion will be primarily a philosophical description of these design procedures. It is assumed that the reader is somewhat familiar with the seismic design requirements in the referenced codes; therefore, this discussion will not be exhaustive.
- A discussion of the previously mentioned characteristics of industrial buildings and how they might affect the performance of the primary structural steel system in a seismic event.
- A listing of publications that the authors have found regarding or related to seismic design of industrial buildings.
- Strategies for designing steel-framed industrial buildings using the most current edition of the predominant building code in the United States (IBC 2003) and the Canadian Building Code (NBCC 1995). IBC was recently updated, and a 2006 edition is now available. The NBCC has been updated, with a 2005 edition recently made available to the general public. Relevant changes included in these updates, which affect these strategies, will be discussed.
- A brief discussion of current research regarding seismic design for industrial buildings, and a discussion of additional research and development necessary with regard to this topic.

Part Two of this paper will focus on self-supporting, nonbuilding structures often encountered in industrial facilities. This will include silos and bins, elevated tanks and ground supported conveyors. Included in this discussion will be:

- A brief description of current seismic building design procedures for these structures in the United States and Canada. Similar to Part One, the procedures discussed will be those presented in IBC 2003 and NBCC 1995. Again, relevant revisions that are included in IBC 2006 and NBCC 2005 will be mentioned.
- A listing of standards available for the design of these nonbuilding structures (in addition to the provisions provided in IBC and NBCC).

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Part One: Seismic Design for Industrial Buildings

Seismic Requirements per the U.S. Building Code, 2003 Edition

Chapter 16 of the 2003 IBC addresses structural design. Applicable design load combinations, seismic design loads and design criteria are presented in this chapter. In many instances, IBC defers to design requirements provided in ASCE 7-02, *Minimum Design Loads for Buildings and Other Structures*. Seismic design in accordance with ASCE 7-02 is also listed as an accepted alternative.

The seismic design requirements in the 2003 IBC and ASCE 7-02 are based primarily upon the 2000 edition of the National Earthquake Hazards Reduction Program's (NEHRP) *Recommended Provisions for Seismic Regulations for New Buildings and Other Structures* (FEMA 368, Commentary is FEMA 369). The start of discussion on seismic design requirements as presented in the International Building Code and in FEMA 368 should start with the purpose of these provisions. Specifically, as stated in FEMA 368 and in the Commentary to IBC 2003, the purpose of the provisions is:

1. To protect the health, safety and welfare of the general public by minimizing the earthquake-related risk to life.
2. To improve the capability of essential facilities and structures containing substantial quantities of hazardous materials to function during and after design earthquakes.

For most steel structures, inelastic behavior is expected if the building is subject to a design-level earthquake. The ability of the structure to withstand this inelastic behavior without collapse is the premise for the majority of the design criteria presented in these references. Essential facilities are designed for higher forces and may have more stringent design requirements. Therefore, the expected level of damage due to a design-level earthquake for these facilities should be less and allow for the continued operation of that facility. It is very possible that it may not be economically feasible to repair a building after a design-level earthquake.

Understanding the premise that inelastic behavior is expected in a structure designed to these provisions is paramount to understanding the intent of the design requirements. This behavior is acknowledged in the various analysis approaches prescribed to predict earthquake forces in the building structure. The most direct analysis approach for estimating both the dynamic and inelastic responses of a building structure is to perform a dynamic analysis (time history analysis), accounting for nonlinear or inelastic effects. Although this may be the most direct approach to predicting the expected behavior of a structure to an earthquake event, most engineers do not possess the analytical tools and knowledge to perform such an analysis. Therefore, the most common approach is to use an Equivalent Lateral Force Analysis. In this approach, an elastic model of the building structure is used, and the base shear associated with the design level earthquake (V) is approximated as:

$$V = C_s W \qquad \text{(Eq. 1)}^3$$

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I}\right)} \leq \frac{S_{D1}}{T\left(\frac{R}{I}\right)}$$

$$\text{(but not less than } 0.44\, S_{DS} I) \qquad \text{(Eq. 2)}^3$$

where

V = earthquake base shear,
W = weight of structure considered during a seismic event,
R = response modification factor,
T = fundamental period of the building structure,
S_{DS} and S_{D1} = earthquake spectral accelerations associated with the building site and
I = importance factor (used to prescribe higher forces for essential facilities).

In Equation 2, the response modification factor, R, represents an adjustment factor used with a linear analysis model to approximate nonlinear dynamic response in the building structure. Therefore, appropriate detailing of the building structure is required to ensure that this approximation is justified. The nature of this "appropriate detailing" is the design criteria included in IBC, ASCE 7 and FEMA 368.

The response modification factor, R, incorporates two effects: an overstrength factor and a ductility (or ductility reduction) factor. The overstrength factor accounts for the difference in the force level required to collapse a frame and the seismic design force level for that frame. This overstrength can be attributed to the following:

1. Design efficiency — in general, members are designed with capacities that are equal to or in excess of their design loads.
2. Drift limits for the building, imposed by seismic design criteria and/or serviceability limit states, result in larger member

sizes than required for strength limit states.
3. The nominal member strengths are larger than design strengths, due to factors of safety or resistance factors (Φ) and the fact that actual steel yield strengths are typically higher than published for a given grade of steel.
4. The building design may be governed by other load combinations, especially where wind loads, gravity loads, and crane loads (both vertical and horizontal) are high.
5. Elastic design methodologies define the strength of a frame by the development of the strength of the weakest element (as compared to the design force) in the frame. After the failure (flexural hinging, yielding, buckling, etc.) of this element, most frames have additional capacity and will continue to resist load until enough members have failed that the structure becomes unstable and collapses. This analysis is similar to plastic analysis methods that have been used for years. The excess strength is expressed as the difference between this collapse load and the load generating the first failure in an individual element (hinging, yielding or buckling).

The second effect included in the R factor is a ductility or ductility reduction factor. This effect is associated with the following:

1. As the structure begins to yield and deform inelastically, the natural period of the building will increase. This increase in period will result in decreased member forces for most buildings and will prevent or reduce a resonant response in the building structure.
2. Inelastic action in members dissipates energy. This is often referred to as hysteretic damping in the structure (whereas damping in the elastic model would be considered viscous damping).

The combination of these two effects was considered in developing the R values that are used today in the United States. The R values currently used are based predominantly on engineering judgment (related principally to commercial and institutional buildings and framing systems common to these buildings) and the performance of various material and systems in past earthquakes. There has been discussion about quantifying these two effects individually and combining these two calculated values to establish a more refined approximation for an R value for a specific building. However, the Commentary to FEMA 368 states that insufficient research has been performed to allow implementation of this type of philosophy, and the resulting complexity of design would be problematic.

For steel buildings, Chapter 22 of IBC 2003 requires that all buildings in Seismic Design Category D, E or F adhere to the requirements of the 2002 AISC Seismic Provisions for Structural Steel Buildings (ANSI/AISC 341-02). For steel buildings in Seismic Design Category A, B or C, the engineer is given the choice of using an R value of 3 and designing per the basic steel specification, or designing with the higher R values provided in the seismic section of Chapter 16 and adhering to the requirements of ANSI/AISC 341-02. AISC has typically advised the use of the former procedure, since seismic loads (even using the decreased R value of 3) will oftentimes be smaller than lateral wind loads on the building structures in the moderate or low seismic areas defined by these seismic design categories. In addition, the increased complexity of design, fabrication and erection associated with the seismic provisions will oftentimes offset any material savings obtained by the use of the higher R values.

Regarding changes included in the 2006 IBC, the authors are aware of the following major changes:

1. The 2006 IBC will completely defer to ASCE 7-05 for seismic design requirements.
2. ASCE 7-05 will incorporate changes included in the 2003 edition of FEMA 368.
3. The 2005 AISC Seismic Provisions for Structural Steel Buildings will be referenced.

Although each of these changes may affect individual design requirements, the basic design philosophy previously described has not changed.

Seismic Design Requirements per the National Building Code of Canada, 1995 Edition

The basic design philosophy included in the 1995 and 2005 National Building Code of Canada (NBCC) parallels that used in the United States as discussed above, with a few differences, as explained below.

The 1995 edition of this code approximates earthquake loads using the following two equations:

$$V = \left(\frac{V_e}{R}\right)U \quad \text{(Eq. 3)}$$

$$V_e = (v)(S)(I)(F)(W)$$

(Eq. 4)

where

V = earthquake base shear,
R = force modification factor,
U = factor representing level of protection, based on experience,
v = zonal velocity ratio,
S = seismic response factor,
I = importance factor,
F = foundation factor and
W = weight of structure considered during a seismic event.

In this equation, S is period-dependent and reflects spectral amplification of motion in the building structure. R is oftentimes referred to as the "ductility factor" and is intended to reflect the capability of the structure to dissipate energy through inelastic behavior. The U factor incorporates overstrength and observations on performance of various material and systems in past earthquakes.

The 2005 edition of the NBCC incorporates many changes, bringing the approaches of Canada and the United States closer together. The revised equation used to determine the earthquake base shear is as follows:

$$V = \frac{S(T)M_v I_E W}{(R_d R_o)}$$

(Eq. 5)

where

V = earthquake base shear,
$S(T) = F_a S_a(T)$ or $F_v S_a(T)$,
$S_a(T)$ = mapped spectral acceleration for building location,
F_a and F_v = functions of site class (soil type) and intensity of ground motion,
M_v = a function of T, type of system and shape of response spectra curve, estimating higher mode effects in building structures,
I_E = importance factor,
R_d = response modification factor associated with ductility of frame and
R_o = response modification factor associated with estimated overstrength of frame.

An explanation of many of the major changes in the 2005 edition is as follows:

1. Revised approach to incorporate newly mapped spectral accelerations, S_{DS} and S_{D1} (same values as used in the United States). These spectral accelerations define the shape of the site-specific, period-dependent response spectra for a given site.
2. Changed the basis for a design-level event to a probability of exceedance of 2 percent in 50 years (versus 10 percent in 50 years in 1995 edition). This is similar to changes introduced in the 2000 IBC in the United States and is based on the goal of providing a more uniform basis for the factor of safety against collapse. This often results in higher seismic design forces in areas where significant earthquakes have happened but are infrequent.
3. A more refined approach to evaluation of the site parameter (foundation factor) that takes into account the nature of the supporting foundation material and the mapped spectral accelerations. This is similar to the approach used in the United States.
4. Delineation of the effects of overstrength and ductility in developing a new expression for the force or response modification factor. This is where the Canadian code varies from the United States code. The Canadian approach independently quantifies both of these effects and uses these independent values to calculate a job-specific R value.
5. An elastic or inelastic dynamic analysis is required, except that an equivalent lateral force procedure is allowed for any of the following cases:

   a. The building has a low seismic hazard as defined by the magnitude of S_{DS}.
   b. Regular structures less than 60 m (approximately 197 feet) in height.
   c. Certain irregular structures less than 20 m (approximately 66 feet) in height.

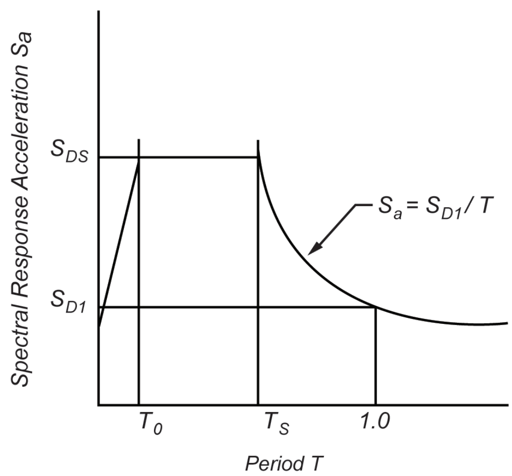
The rationale for this criterion is that dynamic analyses (especially linear, modal analyses) are rather straightforward with current software used in the industry, and these analyses provide a much better approximation of the behavior of the structure in an earthquake event. With regard to industrial buildings, the majority of these buildings should be of a geometry that would allow the use of an equivalent lateral force procedure.

Since it has been demonstrated that the seismic design philosophies in the United States and Canada are very similar, and for the sake of brevity, the remaining discussion will reference only the United States design requirements.

Characteristics of Industrial Steel-framed Buildings and How They Might Affect the Seismic Performance of These Buildings

The following characteristics of a steel-framed industrial building distinguish it from commercial and institutional buildings and affect

Figure 1



Typical earthquake response spectrum curve.

the expected and/or required response of these structures to a design-level earthquake.

- Mass and stiffness properties of the building frames.
- Building geometries.
- Framing systems.
- Bracing arrangements.
- Loading considerations.
- Lack of rigid diaphragm.
- Lack of public exposure (sometimes).
- Presence of hazardous material (sometimes).
- Type of facade.

Each of these characteristics and the effect it has on the expected and/or required response of the structure to a design-level earthquake is provided below.

Mass and Stiffness Properties of the Building Frame — Two types of industrial buildings are considered as follows. The first type is an industrial building without heavy process loads supported on the building frame. These buildings commonly have light metal wall and roof panel systems and are therefore relatively light. Examples of such buildings are typical "pre-engineered" metal buildings. Therefore, these types of industrial buildings have significantly less weight or mass (per unit volume) to be considered in a seismic event as compared to commercial or industrial buildings with similar footprints. Moreover, these buildings may be considerably more flexible than most commercial or institutional buildings, because the facade (metal skin) is less sensitive to building movement than typical architectural facades. The net effect of these two differences on the expected and/or required response of the structure to a design-level earthquake is:

1. The calculated magnitude of seismic base shears is smaller for these types of buildings. This is due to the relatively low seismic weight of the building and the higher fundamental building period, positioning the building structure on the low end of the typical response spectra curve, as illustrated in Figure 1. Both of these effects are acknowledged in the current seismic codes.
2. Seismic loads may still be overstated and/or seismic design criteria may be overly conservative for these types of structures. The rationale for this is, since the seismic loads are low, member sizes in these frames are often governed by live and wind load combinations; therefore, the overstrength component of the response modification factor may be significantly higher than that associated with a heavier building that would have higher seismic shears. In addition, the fundamental period of these buildings should typically be larger than commercial or institutional buildings due to the higher flexibility of these buildings. Building code requirements provide an estimated value for the fundamental period of the building, T_a, based on empirical studies and "typical" building mass and stiffness characteristics. The calculated period, T, to be used in design for strength, is limited to T_a multiplied by a factor (C_u in IBC and 1.5 in NBCC). This upper limit on T for strength calculations is based on the code committee's desire to provide a conservative approximation of the period. For industrial buildings, it could be argued that this limit is overly conservative (see Figure 1 for a typical response spectrum curve and variation of seismic force with building period). It should be acknowledged that second-order effects are required to be included in the design of the building structure. It should also be noted that seismic drift limits provided in IBC 2003 (Table 1617.3.1) do not apply to single-story buildings if building facade and components have been designed or detailed to accommodate the calculated drift.

The second type is an industrial building with potentially high process loads on the building structure. An example would be a fossil-fueled power plant with a large boiler hung from the top deck of the structure. Another example would be a building supporting heavy cranes, where the weight of the crane(s) is significant. In both of these cases, there is a large, relatively concentrated weight or mass positioned high on the structure. This weight or mass may exceed the weight of the supporting building frame and facade. In this case, the predicted vertical distribution of the seismic shear over the height of the building may be affected. This is really a mass irregularity. For example, in a building supporting heavy cranes with metal wall panels, the two predominant mass elements on the building structure are the cranes and the roof. The relative height and magnitude of these two masses should influence the distribution of the dynamic earthquake forces over the height of the building structure. The equations used in the building code to predict this distribution were developed for typical mass and stiffness characteristics in commercial and institutional buildings and may not be accurate for this type of building. A dynamic analysis may be warranted to more accurately predict this distribution of forces.

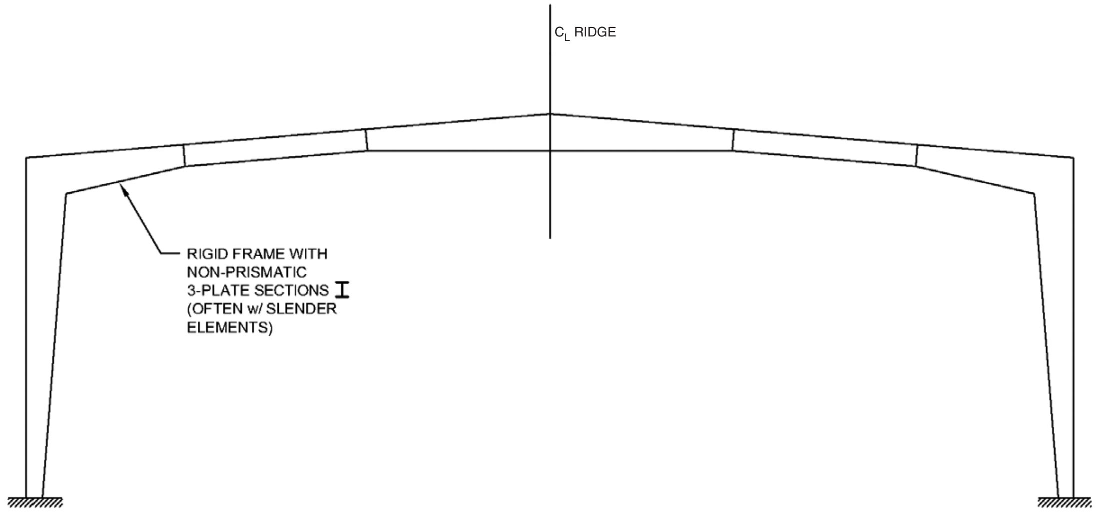
Building Geometry — The primary differences in geometry for industrial buildings discussed in this section are the high floor-to-floor and floor-to-roof heights and the long roof spans that are often encountered in industrial buildings. These geometries are driven by process requirements in these facilities. The effect of this on the expected and/or required response of the structure to a design-level earthquake is:

1. Height requirements for various types of industrial buildings will often exceed height limits imposed by the building codes for common, less-expensive framing systems (ordinary moment frame (OMF) systems and ordinary concentric braced frame (OCBF) systems). Whether this consequence is warranted is questionable. Commentary on the source of these height requirements is not readily available, but is suggested to be based on engineering judgment of the code writers. The mass and stiffness characteristics of a one-story, 60-foot-tall industrial building as compared to a five-story, 60-foot-tall commercial or institutional building are considerably different. Therefore, review of these height limits for industrial buildings is warranted.
2. Many industrial buildings have long roof spans that, for economical reasons, are framed with truss framing. The only moment frame system utilizing a truss that is currently acknowledged by the building code is a special truss moment frame, which utilizes a special interior veirendeel or x-braced panel designed and constructed to force all inelastic behavior into this panel (the fuse in this system), with the surrounding truss elements not subject to inelastic behavior. The drawback to this system is that it is limited to trusses with spans less than or equal to 65 feet and depths less than or equal to 6 feet. This still does not address geometry requirements for many industrial buildings. Therefore, the engineer is left with the option of making the building a braced building (may be difficult depending on the aspect ratio of the building and bracing restrictions) or developing some other form of lateral load-resisting system that meets the intent of the code, and then explaining and defending that concept with local building officials.

Framing Systems — As discussed in the previous section, process requirements within an industrial building structure will typically drive the geometry of the building structure and may also drive the type of framing system used in the building structure. If the process requirements and/or building geometry do not allow for the practical use of discreet bracing (either in plan and/or elevation), some form of rigid frame structure is often necessary to resist lateral loads in the building structure. These rigid frame structures have certain characteristics that make them different from rigid frame structures typically used in commercial and institutional buildings. Examples are as follows:

1. The previously referenced "pre-engineered" metal building system, used in many lighter industrial applications, typically utilizes nonprismatic rigid frame building structures using members with slender elements (see Figure 2). The expected behavior of these types of rigid frames when subject to a design-level earthquake does not necessarily match that of conventional steel rigid frames used in commercial and institutional buildings. Slender cross-section elements may buckle prior to developing full yield strengths of cross-sections, and, with non-prismatic frame profiles, expected locations of flexural hinges in the frame may not be obvious (similar to expected behavior for reduced beam sections in moment frames). IBC 2003 precludes the use of slender elements in special moment frames and requires tested connections for both special and intermediate moment frames. Since the frame profiles and member sizes vary considerably for the myriad of building geometries and loadings included in metal building systems, it is not practical to use tested connections in these structures. Therefore, typical design methodology used for buildings in Seismic Design Category D, E or F (where AISC Seismic

Figure 2

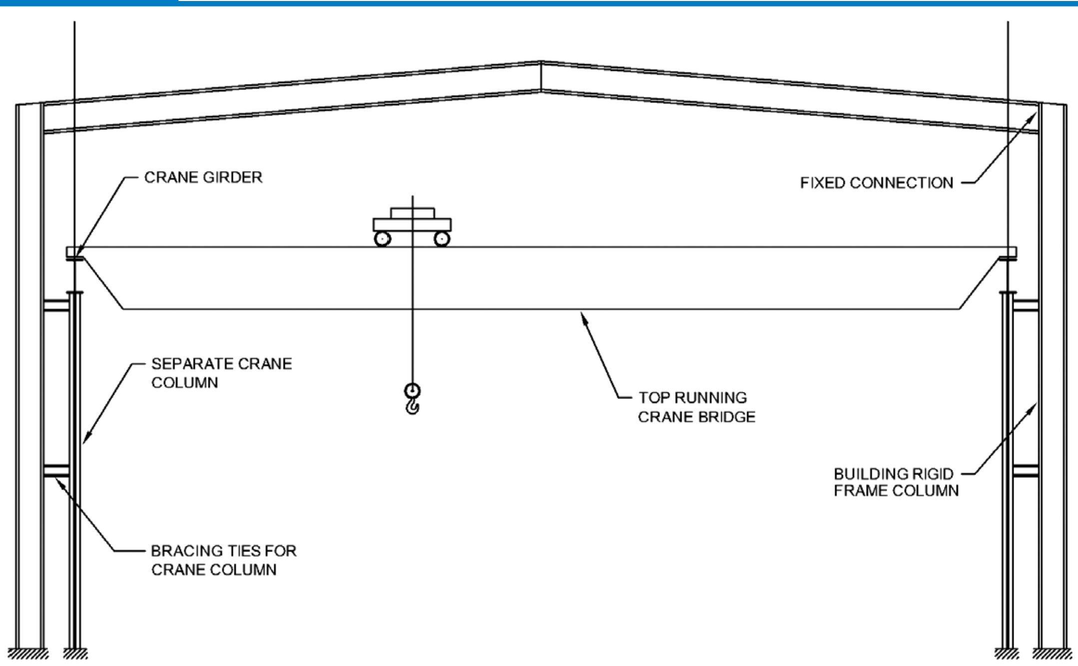


Typical metal building rigid frame.

Provisions must be used) is to design these frames as ordinary moment frames. With this approach, the building height and roof dead load are limited by the restrictions applied to ordinary moment frames.

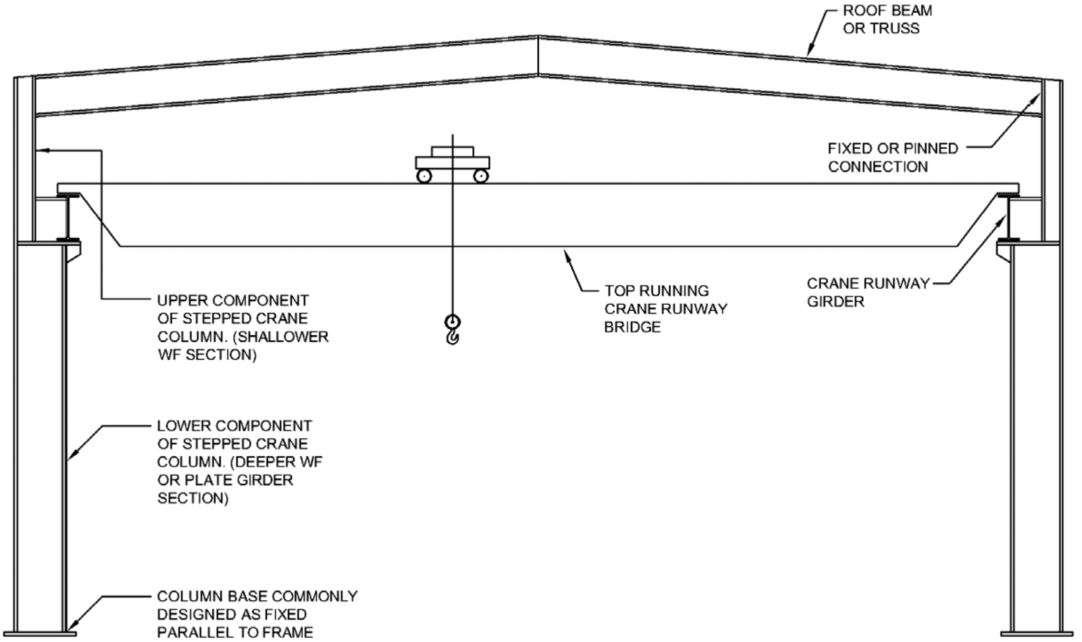
2. Crane runway systems are often included in industrial building structures for use in material handling. In these structures, the geometry of the building is commonly defined by the dimensional characteristics of the crane. Examples of typical building geometries for crane buildings are shown in Figures 3–5. When the mass of the crane is supported on separate crane columns that are independent of the lateral load-resisting system (see Figure 3), the design of the rigid frame is relatively straightforward, assuming that a rational approach is provided for distributing the lateral shear forces over the height of the structure (see discussion in "Mass and Stiffness Properties of the Building Frame," above). When the crane vertical support system is integrated with the rigid frame system, as shown in Figures 4 and 5, the column cross-section, stiffness and flexural strength vary

Figure 3



Crane-supporting building with rigid frame and separate crane columns.

Figure 4

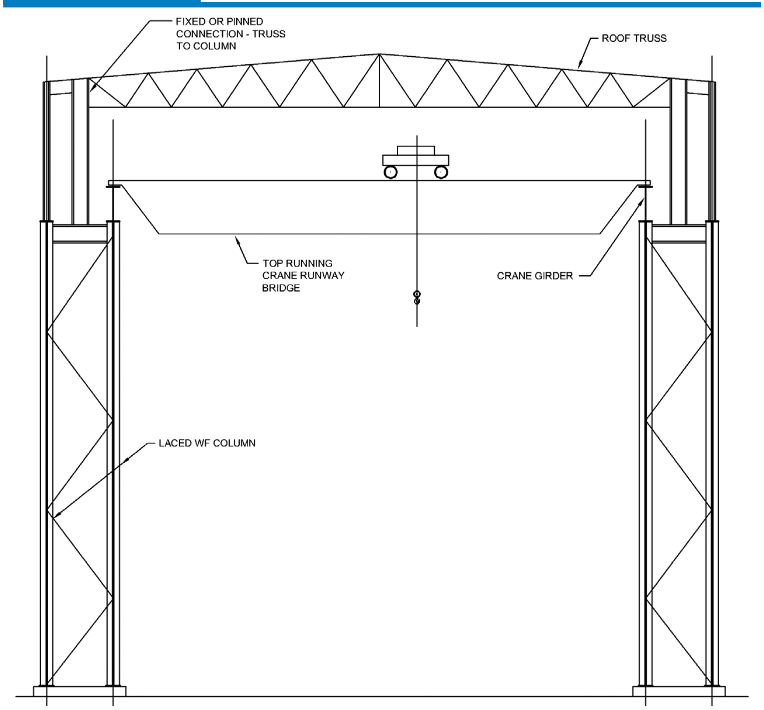


Crane-supporting building with stepped crane columns.

significantly both below and above the crane runway. In these cases, the dynamic response of the structure will be affected by the column properties, and this should be considered in the design of the rigid frame. In heavy crane buildings, as shown in Figure 5, it is not uncommon for the lateral load resistance to come solely from the cantilevered, laced columns. In first reviewing the various lateral load-resisting system classifications contained in IBC 2003 (Table 1617.6.2), it would seem logical to classify this system as an inverted pendulum, cantilevered column system, having a relatively low R value of 2.5. The resulting seismic forces in the building structure may be significant (because of low R value), and IBC and the AISC Seismic Provisions are not clear on detailing requirements for this type of structure.

An additional consideration for crane-supporting buildings is the presence of the crane bridge and its effect on the response of the building structure. Specifically, beneficial or not, the crane bridge serves as an axial tie between the supporting column lines that is limited by frictional capacity of the crane wheel connection to the supporting rail and types of crane wheels used, single-flange or double-flanged. (Single-flange wheels will restrict relative movement in one direction, whereas double-flanged wheels will restrict relative movement in two directions.) The presence of this axial tie in the building structure may affect the dynamic response of the building structure to lateral seismic forces. The design engineer should consider this in the design of the building structure and, perhaps, if these forces are significant, communicate these forces to the crane supplier responsible for the design of the crane.

Figure 5



Heavy crane–supporting building with laced column system.

Bracing Arrangements — As mentioned earlier, process layout within industrial buildings typically dictates the framing geometry and systems used in these structures. Similarly, this layout affects where bracing can and cannot occur. In multilevel industrial complexes (e.g., boiler buildings in power plants), acceptable bracing locations and configurations (both in elevation and plan) may vary from floor to floor, resulting in multiple bracing offsets and potentially other irregularities in the braced frame lateral load-resisting system for the building structure. In most commercial and institutional buildings, it is possible to avoid or limit the extent of these irregularities, whereas in industrial complexes, this may be difficult if not impossible. In designing these types of facilities, the design engineer needs to be cognizant of these potential irregularities, try to avoid them if possible, and address them if it is not possible to avoid.

Loading Considerations — When designing industrial complexes for seismic loads, two loading issues need to be considered. The first issue is what masses or weights should be included in W, the effective seismic weight of the building structure used to calculate the seismic shear for the building structure. As defined in IBC 2003, the seismic weight is to include the weight of the building structure, 25 percent of storage live loads, the weight of permanent equipment, and 20 percent of the flat roof snow load where this load exceeds 30 psf. Some judgment is required from the design engineer regarding how much process loading should be included in the effective seismic weight and what crane loads should be considered for single-aisle or multiple-crane-aisle conditions. The second issue is what load combinations are pertinent for these building structures. In this context, process loadings are typically considered to be live loads, and IBC specifies load combinations that are to be considered. For live loads less than 100 psf, IBC allows a reduction of live load when considered in conjunction with earthquake loads. If the process loading is well-defined and is often present on the building structure, this reduction is not warranted. With regard to crane loads in combination with earthquake loads, IBC does not provide clear direction.

Lack of Rigid Diaphragm — Most one-story industrial buildings have some form of metal roof deck (either a metal deck cladding or a metal roof deck supporting a weather-resistant membrane or topping). With the exception of a standing seam metal roof cladding, most of these decks have diaphragm capability, with the strength and stiffness of the diaphragm dependent upon the deck profile and gauge, support spacing, and type and number of fasteners. Lateral deflections of these diaphragms may be appreciable; therefore, an often-questioned aspect of design is whether these diaphragms are rigid or flexible. From the definition of a rigid diaphragm, the deflections of the diaphragm are not considered significant in comparison to the deflections of the vertical lateral load-resisting system. A flexible diaphragm is defined as having deflections that are considered significant in comparison to the deflections of the vertical lateral load-resisting system. IBC 2003 defines a flexible diaphragm as having a lateral deflection of more than two times the average story drift of the vertical elements of the lateral load-resisting system. For diaphragms categorized as rigid, the diaphragm is considered rigid for the purpose of distribution of lateral story shears and torsional moments. For diaphragms categorized as flexible, story shears are distributed to vertical lateral load-resisting elements, assuming that the diaphragm is flexible and therefore a simple span horizontal element between vertical lateral load-resisting elements.

Standing seam roofs are very popular in pre-engineered metal buildings and, in some cases, rehabilitation work, where they are applied over existing deteriorated roofs. These roofs are a special type of metal deck cladding using formed metal sheets with sidelaps joined together and supported on clips connected to the supporting roof structure below. The nature of this system allows for independent movement parallel to the sheet between each sheet and between the sheets and supporting clips. This independent movement is advantageous in that it allows for differential thermal movement between the roof panel and supporting structure. Due to the nature of this system and the connectivity described above, this roof does not typically provide appreciable diaphragm strength or stiffness, and a separate discreet bracing system is required to transfer lateral loads to the vertical lateral load-resisting systems in the building. In most instances, these roofs are assumed to have sufficient subdiaphragm strength to span to loading points for the discreet bracing system.

Lack of Public Exposure (Sometimes) — Industrial buildings are often located in industrial parks, and the population of these buildings may be considerably smaller than expected in a commercial or institutional building. Therefore, a legitimate question is whether a reduced level of design is warranted in these structures. Using the design approach currently in building codes, this would be a potential argument for a reduced importance factor for these types of buildings. This would obviously be associated with number of occupants in the building structure, presence of hazardous materials in the building structure, and whether the particular facility would be associated with an essential operation after an earthquake event.

Presence of Hazardous Materials — As discussed in the previous section, the presence of hazardous materials in a facility is always a consideration in design. The current building code requirements assign more stringent design requirements (higher loads by virtue of higher importance factor) to facilities containing such substances. Chemicals and waste products generated in these facilities are likely candidates for hazardous material. Section 307 of IBC 2003 defines what materials are considered hazardous.

Type of Facade — Industrial buildings commonly use metal wall panel systems that, when subject to high distortions due to large drifts, do not present a life safety hazard due to broken and falling debris. Excessive drift may result in elongated holes in the wall system and failure of some fasteners, but this would not typically pose a life safety problem. Conversely, architectural facades — including concrete, masonry and/or glass — may deteriorate if the supporting building structure deforms too much, and failure of these systems may pose a life safety hazard to people in or near the building structure.

Existing Publications Regarding Seismic Design of Industrial Buildings

Previous sections of this paper have referenced design codes and standards, including IBC 2003, ASCE 7-02, ASCE 7-05, FEMA 368 and 369, ANSI/AISC 341-02, ANSI/AISC 341-05, NBCC-1995, NBCC-2005, CSA and CISC (specifically CSA Standard S16-01, which addresses limit state design of steel structures, including seismic design requirements). Additional references on seismic design of industrial buildings or related facilities that the authors have investigated include:

- *Guidelines for Seismic Evaluation and Design of Petrochemical Facilities,* published by ASCE, 1997.
- *Guidelines to Improved Earthquake Performance of Electric Power Systems,* published by ASCE, 1999.
- *Seismic Design Guide for Metal Building Systems,* published by the Metal Building Manufacturers Association (MBMA), 2000 IBC edition.
- *Seismic Codes and Standard of Energy Supply Systems,* published by the Energy Commission, Ministry of Economic Affairs, Taipei, China.
- *Technical Report 13, Guide for the Design and Construction of Mill Buildings,* published by the Association for Iron & Steel Technology, 2003.

Strategies for Designing Steel-framed Industrial Buildings Using Current Building Codes

As previously noted, the existing seismic design requirements were written predominantly to address commercial and institutional building structures. Previous discussions have focused on characteristics of industrial buildings that make the design of these structures different for earthquake forces. Based on these differences, it is clear that these provisions need to be amended to address these types of building structures. Recognizing that these amendments are currently not available and design engineers are faced with the dilemma of trying to apply the current seismic design requirements to industrial buildings, the following strategies and thoughts are provided.

Characteristic 1 — Mass and Stiffness Properties of Industrial Buildings — The discussion relative to R values and whether they are appropriate to lighter industrial buildings (see earlier discussion) would need to be addressed by future research and study. Since these structures are often more flexible than commercial or institutional structures, it would make sense to calculate a building period (T), as opposed to using only the approximate period (T_a) as calculated by the equations in the building code. For a simple one-story building with the majority of the mass at the roof level, the building period can be calculated as:

$$T = 2\pi\sqrt{\frac{m}{k}} \tag{Eq. 6}$$

where

m = mass at roof level = W/g (kips-sec.^{2}/in.),
g = acceleration due to gravity (386.4 in./sec.^{2}) and
k = frame lateral stiffness (kips/in.).

Further research and study may also focus on whether the code-prescribed limit on T ($T_{max} = C_u T_a$) is necessary for these types of structures.

The mass characteristics of an industrial building need to be evaluated to verify that both vertical and plan irregularities do not exist. These irregularities could generate response (to a seismic event) in the building structure that varies significantly from that predicted by the equations presented with the equivalent lateral force procedure in either IBC or ASCE 7. Both of these documents define a vertical weight (mass) irregularity as a structure with an effective mass of any story more than 150 percent of the effective mass on an adjacent story (however, a roof that is lighter than the floor below need not be considered). If mass on an elevated floor or roof structure is significantly heavier in one area of the building (as compared to the other areas), the building structure may be prone to potential torsional irregularity. Both of these irregularities are defined in IBC and ASCE 7, with special analysis and design requirements required if these irregularities exist. Special consideration should be provided to a crane-supporting building structure, where heavy cranes contribute mass at an intermediate height on the column. The equivalent lateral force procedure provided in IBC and ASCE 7 distributes the overall base shear to the various levels as a function of the mass and height associated with each of these levels per the following equations:

$$F_x = C_{vx} V \qquad \text{(Eq. 7)}$$

$$C_{vx} = \frac{w_x h_x^k}{\Sigma w_i h_i^k} \qquad \text{(Eq. 8)}$$

where

V = total base shear,
F_x = force at level x,
w_i and w_x = the portion of the total effective seismic weight of the structure (W) located or assigned to level i or x,
h_i and h_x = the height from the base to level i or x,
k = exponent = 1 for structures with fundamental period (T) less than or equal to 0.5 second, and 2 for structures with fundamental period (T) less than or equal to 2.5 seconds. If T is between 0.5 and 2.5 seconds, k can be taken as 2 or determined by linear interpolation.

Equation 8 may generate erroneous results for mass located at an intermediate level on a column, especially when a stepped or laced nonprismatic column profile is used, as previously discussed and illustrated. A dynamic analysis should be performed in these instances to better determine the distribution of seismic shears over the height of the building structure. After a representative number of models are evaluated, rules of thumb may be developed that would alleviate the need for the dynamic analysis.

Characteristic 2 — Building Geometry — As previously discussed, many one-story industrial buildings exceed height limits allowed for standard OMF and OCBF framing systems in Seismic Design Categories D and E. IBC 2003 and ASCE 7-02 limit the use of OMF systems in these Seismic Design Categories to one-story buildings with building heights less than or equal to 35 feet, and where the dead load of the walls and roof do not exceed 15 psf. These documents allow an increase in building height to 60 feet when field-constructed moment joints use bolted end plates, the roof dead load remains limited to 15 psf, and the wall weights more than 35 feet above the base of the building do not exceed 15 psf. This increased height allowance would appear to be in deference to the metal building industry, where bolted end plates are commonly used for moment connections. Similarly, OCBF systems are limited in height to 35 feet, but this height limit is increased to 60 feet for one-story buildings with roof dead loads less than or equal to 15 psf. Large industrial buildings may exceed these height limits. In these instances, the designer is left with a dilemma of using a special steel moment frame or special concentric braced frame lateral load-resisting system. This may be problematic because special steel moment frames require the use of tested connections. Deeper column profiles could very well be required due to strength and drift requirements and the tall floor-to-floor heights. Tested connections for these deeper column profiles are not readily available. The use of a special concentric braced frame may also be problematic due to poor building aspect ratios and inability to locate bracing along interior column lines (due to process requirements). Large bracing connections requiring localized reinforcement of the bracing member in the vicinity of the connection are typical for an SCBF system. These connections are costly. As previously explained, for tall, one-story buildings, seismic base shears may be considerably less than wind loads, and the available overstrength in the building when subject to design-level earthquake forces may be significantly larger than the same height multistory commercial or institutional building (see previous discussion on components of R values). Therefore, it may be argued that seismic detailing requirements are overstated for these buildings.

Another characteristic of industrial buildings included in this category is long roof spans that, for economical reasons, are framed with truss framing. If the building geometry and construction dictate that these frame systems be designed as moment frame systems, the current building code requirements are not explicit on how to address these types of frames.

A number of strategies may be applied when dealing with these issues within the constraints of existing published design criteria. A few of these strategies might be as follows:

1. For buildings in Seismic Design Categories A, B or C, no height limits exist for steel framing systems if the designer chooses to use an R value of 3, categorizing the system as a "steel system not specifically detailed for seismic resistance." As previously stated, it is typically recommended that this approach be followed when applicable.
2. ASCE 7-05 has changed height limit requirements for OMF systems in Seismic Design Categories D and E to 65 feet for a one-story building when the roof dead load is less than 20 psf. No restriction on type of moment connection is included in this height limit extension. Although ASCE 7-05 will not be enforced until IBC 2006 is adopted, this document has been balloted and accepted by the authorities responsible for these documents, and building officials may be willing to accept variances based on this document. This increased height limit, coupled with the increased dead load allowance, should encompass the geometry of a large majority of industrial buildings.
3. ASCE 7-05 has amended the height limit requirements for OCBF systems in one-story buildings in Seismic Design Category D or E to extend to 60 feet (same as for ASCE 7-02), but with dead load allowance increased to 20 psf. As stated in the previous paragraph, the use of this provision is not currently an adopted building code provision, but the building official may be willing to accept a variance consistent with this provision based on this document.
4. The 2005 edition of the NBCC lists height limits for various systems. However, the Commentary to this document indicates that certain one-story buildings are excluded from height limits. Outside of Canada, this could be used only as a basis for seeking a variance to IBC requirements. This discrepancy between IBC and NBCC should be studied for future editions of these two codes to develop a consistent rationale for one-story buildings.
5. If the building qualifies for use of "Simplified Analysis Procedure" and this analysis method is used to derive seismic forces, height limits are increased to 240 feet for buildings in Seismic Design Category D or E (refer to IBC 2003, Section 1617.6.2.4.1). For the types of buildings discussed in this paper, this would require that the building lateral load-resisting system use braced frames (OCBFs allowed) in both directions. This may be difficult, depending on the building aspect ratio and whether bracing at interior frame lines would be allowed by the building owner.
6. ASCE 7-05 provides a new seismic design methodology for nonbuilding structures similar to buildings. This methodology provides three different R values for a given structural system. The first, highest R value is the same as that provided for building structures and has the same height limits and detailing requirements required by ANSI/AISC 341 (Seismic Provisions) for building structures. The second, somewhat smaller R value (resulting in higher design forces) has expanded height limits but maintains the ANSI/AISC 341 detailing requirements. The third, lowest R value removes height limits altogether and requires design per the AISC specification (ANSI/AISC 360) but does not require the special detailing requirements from ANSI/AISC 341. The authors believe that this provides a very rational and inclusive approach for all forms of industrial buildings. Again, this is not currently an adopted code option, but a building official may be willing to accept a variance based on this published design philosophy.
7. For one-story buildings with long-span roof trusses incorporated into moment frames, the authors suggest categorizing the frame as an OMF with a strong beam/weak column system. Therefore, when this frame is subject to design-level earthquake forces, flexural hinging should occur in the column. IBC and ANSI/AISC 341 do not prohibit this behavior for OMF systems in one-story buildings. The truss should be designed for both gravity load and lateral load end moments obtained from an appropriate frame analysis. The roof truss should be designed for end moments (due to earthquake forces) consistent with developing the maximum expected flexural capacity of the supporting column (1.1 R_yM_p). The top and bottom chord of the truss must be connected to the column to transfer the corresponding forces. Research at Georgia Institute of Technology (Georgia Tech) that was sponsored by the Steel Joist Institute

(SJI) supports this design approach. This design approach will be included in a new release of *SJI Technical Digest 11,* "Design of Joist-Girder Frames."

Characteristic 3 — Framing Systems — Common framing systems used in industrial buildings were previously discussed. Strategies for designing these framing systems within the constraints of existing published design criteria are as follows:

1. The Metal Building Manufacturer's Association produced the publication, *Seismic Design Guide for Metal Building Systems Based on the 2000 IBC.* This publication addresses design of pre-engineered metal building structures per provisions consistent with IBC 2000. The majority of these provisions are consistent with IBC 2003. Height limit constraints were discussed in the previous section of this paper.
2. Crane-supporting building framing systems using stepped or nonprismatic column arrangements are technically irregular with regard to seismic performance and should be analyzed using a dynamic analysis procedure. This analysis will provide a more accurate estimate of potential dynamic magnification (of ground motion) in the structure and offer insight with regard to the distribution of seismic shears over the height of the structure. With this information, the designer can better understand where inelastic demand can be expected in the structure, and appropriate detailing and construction can be specified for these areas. A two-dimensional model should be sufficient to obtain the desired information. Inclusion of an axial tie in the model representing the crane bridge may affect results if there is a large disparity in the lateral stiffness of different column lines. The authors suggest the following approaches for seismic design of various crane building systems:

   a. For buildings in Seismic Design Categories A, B or C, no height limits exist for steel framing systems if the designer chooses to use an R value of 3, categorizing the system as a "steel system not specifically detailed for seismic resistance." As noted before, it is recommended that this approach be followed when applicable.
   b. For crane-supporting buildings with separate columns supporting the weight of the crane system and rigid building frames providing lateral stability (see Figure 3), design the rigid frame using the provisions of the building code and ANSI/AISC 341. Consider the crane level to be one level in the building structure and the roof level to be the other.
   c. For rigid frames with stepped crane columns (see Figure 4) using standard wide flange shapes (or comparable plate girder sections), fixed connections between the columns and roof beams, and either fixed or pinned column base, consider the frame to be an OMF. Design the connections recognizing where inelastic demand may be required. This would include the roof-beam-to-column connection, which should be designed in accordance with ANSI/AISC 341. In addition, the connection detail between the lower and upper column shafts (at the step in profile) should be detailed to provide ductile performance. This should include using weld metal that meets the notch-toughness requirements provided in ANSI/AISC 341, providing CJP welds and developing maximum expected material strengths (1.1 R_yF_y) of each column profile at the splice location, and using weld access holes that conform to ANSI/AISC 341 requirements. Furthermore, when fixed-base columns are used, the column bases should be designed to provide ductile behavior limited either by the maximum expected flexural strength of the column profile (1.1 R_yM_p) or the flexural capacity associated with the maximum expected yield strength of the anchor rods (1.1 $R_yF_yA_e$), whichever is less.
   d. For heavy crane–supporting buildings using cantilevered, laced columns (see Figure 5), the effective flexural stiffness of the laced columns is typically significantly higher than the single column shaft extended above the laced column to the roof. The low R values prescribed for an inverted pendulum system in the building code were intended to apply to fixed-based column or cantilevered systems that are relatively flexible. Laced column crane-supporting building systems are usually designed in this fashion to limit movement due to crane and wind loads; therefore, this system is usually very stiff. Based on this and the previous discussion relative to overstrength of these systems, the authors suggest categorizing this system as an OMF, subject to the same seismic design factors (R, Ω, and C_d) presented in the

building code. Buildings framed in this fashion are often very tall and may exceed height limits for an OMF. Rationale explained in the discussion on building geometry may be used to extend these height limits. Recognizing the large disparity in strength and stiffness between the lower, laced column shaft and the single upper column shaft, the connection of the single upper column shaft to the laced column shaft would be an obvious potential location for inelastic demand under seismic loading. Therefore, this connection should be designed to develop the maximum expected flexural capacity of the single upper column shaft ($1.1\ R_yM_p$). The overall building stability is dependent on the integrity of the column base details; therefore, the anchor rods and column base details should be designed to provide ductile behavior at this location. The authors suggest designing the anchor development in the supporting foundation and the column base detail for the maximum expected yield strength of the anchor rods ($1.1\ R_yF_yA_e$). In addition, the authors recommend using amplified seismic loads (Ω level forces) for the design of the axial lacing members between column shafts to attempt to prevent buckling or yielding of these members and their connections. Since these buildings use nonprismatic columns, a dynamic analysis may be necessary to more accurately predict the magnitude and distribution of seismic shears. However, realizing that the lateral stiffness of the lower, laced column is significantly higher than the framing above the lower laced column, an approximate and conservative approach would be as follows:

i. Analyze the structure as two independent elements. Analysis A would be for the building structure (and supported mass) above the laced column. This structure would be modeled as a one story, fixed-base frame, assuming that its support (the laced column) is rigid and, therefore, motions at the base of the single shaft column are approximately the same as the ground motions. Analysis B would consider only the laced columns, supporting the mass of the building structure and the supported crane.
ii. A building period and seismic shear would be developed for the structure included in Analysis A. This shear would be used to design the portion of the structure above the laced columns.
iii. A period for the laced columns would be calculated for Analysis B. This period would be used to determine the seismic shear associated with the mass of the crane. This shear would be applied at the top of the laced columns. The laced columns would be designed for the shear and overturning moment from the base of the columns included in Analysis A and the shear determined in Analysis B.

A simple approach to the design of these building structures (especially those that are irregular) would be to use the approach for nonbuilding structures similar to buildings presented in ASCE 7-05. By choosing to use a very low R value (resulting in higher seismic design forces), height limits and ANSI/AISC 341 detailing requirements could be avoided. This is not currently an adopted code option, but a building official may be willing to accept a variance based on this published design philosophy.

Characteristic 4 — Bracing Arrangements — Discontinuities or offsets in bracing over the height of a building structure can result in both vertical and plan irregularities in the building structure. These irregularities are discussed in both IBC and ASCE 7. The designer should be aware of design requirements and restrictions associated with these irregularities.

Characteristic 5 — Loading Considerations — The design engineer must use judgment in evaluating what portion of process loading is to be included in the seismic dead load and the subsequent calculation of seismic shear. These process loads should be what would be expected under normal operating conditions. Cranes supported by a building structure represent a special condition, in that the mass of the crane is typically significant and can move over the extent of the crane runway. AIST's *Technical Report 13* suggests that crane weights include only the dead load of the crane (not lifted load), positioned to generate the maximum seismic effect. For buildings with multiple cranes and/or multiple crane aisles, this document states that all cranes should be positioned to generate the maximum seismic effect on the building frame. The authors believe that this is overly conservative. Unless there is a logical reason for all cranes to be positioned to produce this effect (e.g., crane access platforms located in a particular building bay or a bay devoted to crane maintenance), the authors suggest that the design engineer consider one crane in one building aisle, positioned to generate the maximum seismic effect for any given element in a frame.

Characteristic 6 — Lack of Rigid Diaphragm — Previous discussion adequately describes this issue. IBC and ASCE 7 identify the pertinent design requirements.

Characteristic 7 — Lack of Public Exposure — Previous discussion adequately describes this issue. Further refinement of importance factors for this condition could be a point of discussion for code officials in the future.

Characteristic 8 — Presence of Hazardous Material — Previous discussion adequately describes this issue. IBC and ASCE 7 identify the pertinent design requirements.

Characteristic 9 — Type of Facade — As previously noted, industrial buildings often will have facade systems that do not pose a safety risk if the building experiences large drifts during a seismic event. As noted previously, IBC 2003 has no seismic drift limits for these types of buildings if the exterior wall system has been designed to accommodate these drifts. However, the vertical load carrying capacity of all components of the building structure must not be compromised when the building structure experiences these drifts. Furthermore, mechanical and electrical components of the building structure and any supported equipment are to be designed to meet code anchorage and performance requirements when the building structure experiences these drifts.

Current and Needed Research Related to Seismic Design of Industrial Buildings

As previously mentioned, research was recently completed at Georgia Tech on seismic behavior of steel joist girder structures. The results of this research indicate that one-story rigid frame structures using joist girders (or conventional trusses) can be adequately designed for seismic performance consistent with an ordinary moment frame structure as defined by IBC and ASCE 7. SJI intends to publish a technical digest outlining the design requirements for such a system. As previously discussed, this will assist design engineers with the design of one-story industrial building structures with larger clear spans where truss framing is more economical.

Research is currently being performed on seismic design for industrial buildings by Professor Robert Tremblay at Ecole Polytechnique, Montreal, Canada. The focus of this research is on simplifying and developing industrial building seismic design criteria, taking into account a number of the distinguishing characteristics of these buildings as discussed previously in this paper.

A number of ideas or thoughts that require further research or study have been proposed in the previous discussions included in this paper. These are summarized below:

- Appropriate R values for industrial buildings.
- Appropriate height limit requirements for one-story building structures.
- Possibility of extrapolating ASCE 7-05 design methodology for nonbuilding structures for use in design of industrial building structures.
- Dynamic seismic response of industrial crane building structures that use stepped crane columns.
- Further refinement of importance factors, specifically for industrial buildings with limited public exposure.

AISC is in the process of forming an ad hoc committee on design of industrial building structures and nonbuilding structures. These concerns or topics may be addressed by this committee.

Part Two: Seismic Design for Nonbuilding Structures Often Encountered in Industrial Facilities

As previously noted, this discussion will be limited to silos and bins, elevated tanks and ground-supported conveyors. Here, silos and bins are defined as ground-supported or elevated storage vessels that are used to store solid (or granular) material. A tank will be defined as a vessel used to store liquid material.

Current Code Requirements

IBC 2003 references Section 9.14 of ASCE 7-02 for seismic design requirements for nonbuilding structures. ASCE 7-02 allows the use of other design methodologies that are documented in approved national standards, with the following provisions: (1) the seismic ground accelerations, including modifications for site factors (soil conditions), match those derived from ASCE 7-02, and (2) the seismic base shear and overturning moment are at least 80 percent of those derived from ASCE 7-02. A number of these standards are referenced in ASCE 7-02. For the nonbuilding structure types noted above, some useful recognized standards include:

Silos and Bins — ACI (American Concrete Institute) 313, *Design and Construction of Concrete Silos and Stacking Tubes.* It should be noted that, even if the silo or bin is constructed from steel rather than concrete, this document can assist in the determination of seismic forces associated with the stored material.

Tanks

- ANSI/AWWA (American Water Works Association) D100, 1997, *Welded Steel Tanks for Water Storage.*
- ACI 350R-89, *Environmental Engineering Structures.*
- API (American Petroleum Institute) 620, 1998, *Design and Construction of Large, Welded, Low Pressure Storage Tanks.*
- API 650, *Welded Steel Tanks for Oil Storage.*

ASCE 7-02 includes provisions for nonbuilding structures that are supported directly on grade and nonbuilding structures supported by other structures (for this discussion, the building structure). If the weight of the supported nonbuilding structure is less than 25 percent of the weight of the building structure and nonbuilding structure, the supported nonbuilding structure is considered to be a component with design forces and requirements determined from the building component equations. If the weight of the nonbuilding structure is more than 25 percent of the weight of the building structure and nonbuilding structure, the nonbuilding structure is to be included as part of a combined system composed of the supporting structure and nonbuilding structure. The designer must then choose the appropriate R value, considering both elements of the system. However, if the nonbuilding structure is not rigid (natural period of nonbuilding structure less than or equal to 0.06 seconds), the maximum R value allowed is 3.0.

The design forces on nonbuilding structures are determined using the equivalent lateral force procedure documented for building design with an adjusted minimum limit for the seismic response coefficient, C_s. A dynamic analysis procedure is required only for nonbuilding structures located at a site where $S_{DS} \geq 0.50$ g and that are judged to be irregular and cannot be accurately modeled as a single mass system. Seismic coefficients (R, Ω and C_d) and height limits (as a function of seismic design category) are provided for various types of nonbuilding structures. Two categories of nonbuilding structures are "nonbuilding structures similar to buildings" and "nonbuilding structures not similar to buildings." Nonbuilding structures similar to buildings are intended to be detailed consistent with the comparable detailing requirements for building structures. No specific detailing criteria are provided for nonbuilding structures not similar to buildings. However, the design engineer should attempt to recognize what element(s) in the structure are intended to be the ductile fuse for the structure or what elements should be specifically protected (often, connections and/or anchorage details for the structure). These elements should be designed to allow for the development of the intended fuse. Surrounding and protected elements should be designed strong enough to force the fuse to occur in the intended elements. If uncertainty exists relative to detailing requirements for connections or the surrounding elements in the structure (nonfuse elements), the authors suggest using Ω-level (i.e., amplified) seismic forces for the design of these connections or elements.

As previously noted, ASCE 7-05 provides a new seismic design methodology for nonbuilding structures similar to buildings. This methodology provides three different R values for a given structural system. The first, highest R value is the same as that provided for building structures and has the same height limits and detailing requirements required by ANSI/AISC 341 (Seismic Provisions) for building structures. The second, somewhat smaller R value (resulting in higher design forces) has expanded height limits but maintains the ANSI/AISC 341 detailing requirements. The third, lowest R value removes height limits altogether and requires design per the AISC specification (ANSI/AISC 360), but does not require the special detailing requirements from ANSI/AISC 341. The authors have already stated their appreciation of this design philosophy. This is especially true in nonbuilding structures where the expense of design fees and sophisticated fabrication costs associated with the design of high-ductile seismic systems may not be warranted (i.e., these costs exceed the increased costs associated with using a less-sophisticated design and higher design forces).

As mentioned earlier, the purpose of this section is to make the reader aware of nonbuilding structure seismic design criteria, design requirements for these structures provided in the building code, and other standards that may be applicable. The seismic design for each of these nonbuilding structure types is discussed at length in the documents noted above. Also as noted above, an ad hoc committee is currently being formed by AISC on the design of industrial buildings and nonbuilding structures. It is assumed that seismic design of steel-framed, nonbuilding structures would be an included area of study and review for this committee.

Conclusion

Current building codes provide seismic design criteria for buildings that are based on characteristics of typical commercial and institutional buildings. Industrial buildings have many characteristics that distinguish them from commercial and institutional buildings. Therefore, the design of these facilities for seismic loads can be problematic, with the design engineer and building officials uncertain of how to apply the provisions provided in the building code to these structures. These characteristics have been defined in this paper, and the authors have provided strategies and thoughts on how these structures may be designed, accounting for these characteristics. Definitive answers are not provided to all design issues discussed, but the intent of this paper is to define these issues with the hope of generating discussion and future study on these issues. As noted, IBC 2006 and ASCE 7-05 provide refinements that assist with some of these issues.

Many industrial facilities include nonbuilding structures that also require consideration for seismic loads. The design of these structures is typically based on a combination of building code requirements and recognized industry standards. This paper provided a cursory review of these design requirements, with references to applicable recognized standards.

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