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SEISMIC DESIGN OF COMPOSITE STRUCTURES: THE AISC 2016 SEISMIC PROVISIONS

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ABSTRACT

Provisions for the design of composite structures in the USA were introduced as part of the 1994 Building Seismic Safety Council development cycle, and those recommendations served as the basis for Part II of the 1997 American Institute of Steel Construction *Seismic Provisions*. Those provisions were expanded in the 2005 edition, and were unified with the steel provisions (formerly Part I) in the 2010 edition. The 2016 *Seismic Provisions* will represent a far more unified and expanded treatment of the design of steel and composite structures. This paper describes both some of the overarching decisions on how to integrate steel and composite design that have driven the code developments for composite construction in the USA and describe some of the new provisions and composite systems that will appear in the 2016 edition of the *Seismic Provisions*. In addition, a complete design of a slender encased beam-column is provided.

Introduction

The superior behavior of composite structures under extreme loads was obvious in the aftermath of the 1906 San Francisco Earthquake and Fire (ASCE, 1906). In those events, steel structures encased in concrete or masonry performed extremely well in comparison with almost all the other structural systems being used at the time (Hamburger & Meyer, 2006). This led to the widespread use of composite columns and beams in multi-story construction in the Western part of the USA through the 1950s. In most cases the design did not take into account the beneficial strengthening and stiffening effects of the cover, as quantification of these effects did not become possible until much latter (Batho, 1936). In general, the amount of confinement steel was low and often the cover was only made of masonry rubble, so engineers preferred to neglect composite action and considered only the fire-proofing benefits. However, from their earliest specifications, both the American Concrete Institute (ACI, 1910) and the American Institute of Steel Construction (AISC, 1924) recognized the advantages of composite columns and explicitly included design provisions for these structural members. In both cases, the provisions attempted to treat composite columns as either equivalent reinforced concrete or steel ones, an approach that has handicapped their use until recently.

The use of composite floor systems was explicitly included into the AISC 1936 provisions. Design provisions for composite columns continued to be updated by ACI until the 1960s (Furlong, 2012), while AISC continuously updated both composite column and beam design provisions (Viest, Fountian, & Singleton, 1958). A particularly noteworthy development was the issuance of the 1986 Load and Resistance Factor Design Specification (AISC, 1986), the first one to address composite members from the ultimate strength standpoint. However, none of these early design provisions explicitly addressed for seismic design.

The 1971 San Fernando Earthquake pointed out notable deficiencies in seismic design in the USA and led to an effort to develop new provisions for both seismic demand on structural systems and resistance of structural members (ATC, 1978). In 1977, the U.S. Congress passed the Earthquake Hazards Reduction

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(NEHRP) Act to plan effective ways to protect the lives of building occupants during an earthquake and mitigate the impact of such disasters on the national economy. In 1979 the Building Seismic Safety Council (BSSC), an arm of the National Institute of Building Sciences (NIBS), was created to with the complex technical, regulatory, social and economic issues involved in developing and promulgating national building earthquake risk mitigation provisions.

The BSSC 1994 *NEHRP Recommended Seismic Provisions for Seismic Regulations for New Buildings and Other Structures* (BSSC, 1994), the fourth edition of such guidelines, contained the first seismic provisions for composite construction. The provisions were developed by a small committee, and were a combination of design recommendations based on experimental research in the USA, Japan and New Zealand in the 1970s-1980s, experience in the design of tall buildings in areas of high winds in the USA, and similarities to steel construction in seismic regions of the USA. The latter were particularly important in selecting the system performance factors (ductility (R), deflection amplification (C_d) and overstrength (Ω_0)) for the composite systems, as many of the systems described in those initial provisions were neither common nor had a seismic performance record that could be used to calibrate the factors.

The NEHRP/BSSC Task Group 12 - Composite Construction, author of these original provisions, recognized their importance as "enabling legislation" and attempted to keep the provisions as broad and non-prescriptive as possible. Thus in developing the recommendations for the NEHRP 1994 edition, they attempted not only to develop provisions for existing systems but also to encourage the development of new systems that promote structural efficiency (Viest, et al., 1997). The idea was that many of the newer system could be alternatives to all steel or concrete systems in low to moderate height buildings (4 to 30 stories). The BSSC documents are meant as recommendations and to be adopted into design codes after a trial period. The BSSC provisions were incorporated as Part II of the 1997 AISC *Seismic Provisions* (AISC, 1997), and were reaffirmed in essentially the same format until the 2005 edition. The composite provisions and commentary for Part II took up only 21 and 28 pages, respectively, as compared with 97 and 140 pages, respectively, for the steel structures portion (Part I).

Up to 2005, the *Seismic Provisions* recognized seven types of composite structural systems. Since many of them have a counterpart in steel and concrete the prefix "C-" was been used to name the corresponding composite system:

- **Composite Partially Restrained Frames (C-PRF):** C-PRF consist of steel columns and composite beams joined by composite partially-restrained (semi-rigid) connections. This is a very interesting system from the seismic standpoint because the flexibility of the connections can be used to adjust the natural period of the structure and thus reduce seismic demand. In addition, the connections provide a "fuse" since they are weaker than either beams or columns, and thus a true capacity design approach can be used in their design. Because of this flexibility, however, these structures would also seem to be stability ($P-\Delta$) critical, at least under a simplified equivalent lateral load design approach.
- **Composite Ordinary Moment Frames (C-OMF):** C-OMF include a variety of configurations where steel or composite beams are combined with steel, composite or reinforced concrete columns. The term ordinary is used to indicate that little of the detailing required for critical structures (Seismic Performance Categories D and E in NEHRP) is required in this type of structure.
- **Composite Special Moment Frames (C-SMF):** C-SMFs are similar to C-OMFs except that much more stringent detailing is required in order to provide behavior similar to that of a steel SMF. In this case the columns, if composite, are required to both meet all AISC requirements for b/t and h/t ratios and have all the transverse reinforcement required for columns by Chapter 21 of ACI 318 (ACI Committee 318, 2011). As in most ductile frames, the columns and joints are required to develop the full strength of the beams so that a stable strong column-weak mechanism develops. Clearly the design of the connections is a key element in this system, and detailed provisions for a system incorporating steel beams and concrete columns are available. C-SMF are very efficient in resisting earthquake loads as shown by Japanese SRC (steel-reinforced concrete) system in which an entire steel skeleton is covered with reinforcing cages with large amounts of transverse reinforcement.
- **Composite Concentrically Braced Frames (C-CBF):** C-CBFs are similar to their steel counterparts except that some of the members (beams, columns, and braces) are composite. There is considerable debate on the applicability of braced frames in areas of high seismicity because the tendency of the braces to buckle results in poor energy-dissipation characteristics if the structure goes inelastic. To alleviate the buckling problem several researchers have proposed to utilize composite braces (either encased shapes or concrete-filled tubes, with now the variant of buckled restrained braces or BRBs) where the stiffening effect of the concrete prevents local buckling. In many cases the composite action is required only within the brace and presents a problem at the connections since ideally they should act

as truss members. In this case careful detailing is required at the ends of the braces to insure that ductile yielding in the member and not a fracture at the connection is the failure mode.

- **Composite Eccentrically Braced Frames (C-EBF):** C-EBFs, as the name implies, are analogous to the usual eccentrically braced frame except that some of the members are composite. When the EBF concept was originally developed there was some concern as to whether the floor beams, which are in effect composite beams, could accommodate the large rotational ductilities demanded by the system without causing local failures. Extensive research has been carried out in this area indicating that the floor elements are capable of withstanding the very large shear deformations required by short links.
- **RC Walls Composite with Steel Elements:** Initially, three possible variations of this system existed, and they corresponded to cases of hybrid structures. The first utilizes concrete panels as infills in steel or composite frames. The second is where large steel sections are used as boundary elements in concrete shear walls. The third one is where steel or encased composite beams are used to tie two reinforced concrete shear walls. Numerous variations of these systems have been developed since its initial incorporation into the *Seismic Provisions*.
- **Steel Plate Reinforced Shear Walls:** Since the early 1980's the concept of utilizing steel plate shear walls has been popular. The concept is very similar to the use of plate girders in bridges, except that the main element is vertical rather than horizontal. These systems have been used successfully as retrofits in critical steel structures (hospitals) where access to the structure was severely limited by the need to keep it operating during the retrofit. The system basically behaves as a CBF with the tension field action taking the lateral loads. Composite steel shear walls, in which the steel plate is covered with concrete and composite action activated by mechanical connectors, have been postulated as a system with better energy dissipation capacity. Great care is needed in connecting the plates to the boundary elements since the shear wall is such an efficient structural element that it can easily overstress the adjacent columns and beams.

The following sections discuss important updates to the composite members design rules in both the 2005 and 2010 AISC *Specification* and the AISC *Seismic Provisions* as a precursor to describing those in the 2016 edition. This is important as the *Specification* forms the basis for the *Seismic Provisions*, because since the early 2000s AISC policy has been (a) not to repeat in the *Seismic Provisions* material that is already contained in the main *Specification* and (b) to weed out any provisions that deal with the demand side of the design process. The demands are calculated based on ASCE 7 (ASCE, 2010); the provisions for seismic demand calculations and structural system factors are important, and the latter will also be discussed herein.

The 2005 and 2010 AISC Specification – Chapter I (Design of Composite Members)

The 2005 AISC *Specification* (AISC, 2005) marked a milestone that positively impacted the AISC 2005 and 2010 *Seismic Provisions* for at least four main reasons:

1. In general terms, the *Provisions* introduced in Appendix I the direct design method for stability calculations. While the precise formulation of the method is unique to the AISC *Specification*, some of its features are very similar to the Eurocodes, the Australian standard, and the Canadian standard that utilize nominal loads for stability calculations. The 2005 *Specification* provided a direct path so that the inherent larger stiffness of composite columns could be used to reduce both building drift under lateral loads and second-order effects under gravity loads.
2. Insofar as composite columns, the provisions presented a completely new approach for the design of composite columns, with two objectives. The first objective was to develop a simple and seamless procedure for calculating the strength of composite members subjected to combined flexure and axial force. The procedure bridged the gap between a typical reinforced concrete and a typical steel beam-column at the cross-section strength level by using a generic strain compatibility approach that could be simplified for design into a rigid-plastic approach similar to that used by the Eurocodes (CEN, 2009). The second goal was to improve and modernize the AISC composite column provisions particularly with respect to the use of equivalent stiffness for both member stability and frame analysis (Leon, Kim, & Hajjar, 2007).
3. Insofar as composite columns, the provisions extended their use to non-compact and slender sections by providing a new set of local buckling limits. However, in this edition the *Specification* did not provide explicit equations to calculate their axial force-bending capacity.
4. Insofar as composite beams, the provisions presented an update on shear connector design and the commentary addressed issues such as long-term deflections and partial interaction that had received scant attention in previous editions (Leon, 2001).

Similarly, the 2010 AISC Specification also had major modifications that impacted composite design, including:

1. Where possible, for all detailing requirements related to concrete, the *Specification* invoked ACI 318 (ACI Committee 318, 2011). Of particular importance is the attempt to coordinate all requirements for concrete strength of steel anchors (headed shear studs) to ACI 318 Appendix D.
2. The Specification set the minimum strength of a composite column as equal to that of a bare steel column using the same steel section as the composite member. This corrects an inadvertent mistake in the previous edition which would have given strength values lower than those for the bare steel in the case of extremely slender members.
3. New material was added and revisions are made to the load transfer requirements in composite components. The expanded scope of this topic has warranted the creation of a new dedicated section for load transfer in composite members.
4. The resistance factor and safety factor for encased and filled composite beams was adjusted based upon assessment of new data.

It is important to note that in the USA, and particularly in regions of lower seismicity, it is permitted to design a broad range of steel structures for higher equivalent base shears (reduction factor $R=3$) using the *Specification* instead of having to comply with the detailing requirements of the *Seismic Provisions*. These requirements are based on the type of structural system and exposure category as stated in ASCE 7 (ASCE 7, 2010), but it had not been made clear until the introduction of a note in the upcoming 2016 *Seismic Provisions* that this exception excluded composite systems.

The 2010 AISC Seismic Provisions

The 2010 AISC *Seismic Provisions* represented an ambitious effort to extensively update and reorganize the document, particularly insofar as composite structures is concerned. Editorially, the main changes were the incorporation of the old Part II (Composite Structural Steel and Reinforced Concrete Buildings) into the main body, with composite moment resisting systems becoming Chapter G and braced and wall systems becoming Chapter H. This change has enormous symbolism, as it puts composite systems on par with conventional steel ones.

Technically, the incorporation of the old Part II into the main part of the seismic specification required that many of the existing provisions be recast in far more prescriptive fashion. This took two forms: (a) general provisions that are applicable to all composite members were moved to Chapter D (General Member and Connection Design Requirements), and (b) the development of parallel system provisions for composite systems as existing for steel ones in Chapters E (Moment Frames Systems) and F (Braced Frames and Shear Wall Systems) into Chapters G and H. The provisions in the member chapter are intended to provide the required detailing so that elements can be used in ordinary, intermediate or special systems and to address specific connection design issues. The provisions in all system chapters now follow a prescribed format that includes separate sections for basis for design, analysis, system requirements, member requirements and connections. The basis for design section is new and intended to explain the seismic response characteristic of each structural system.

The 2016 AISC Specification – Chapter I (Design of Composite Members)

The 2016 AISC Specification also contains important updates, including:

1. In Section I1-2 – General Provisions, a new analysis method for cross-sectional strength has been introduced. The method, called the effective strain method, has been added to explicitly permit the design of non-compact and slender composite beam-column sections. This method requires great care and expertise as it utilizes modified material stress-strain curves to implicitly account for the effects of steel tube local buckling, yielding, concrete cracking, compression inelasticity, and confinement.
2. In Section I1-5 – Stiffness for Calculation of Required Strength, a requirement that all stiffness values be reduced by 0.8 to account for stability effects is introduced. In addition, the flexural stiffness needs to be reduced by a further 0.8 for use in the Direct Design Method. The values of stiffness to be used in frame stability calculations had not been explicitly stated in previous versions of the Specification.
3. In Section I2-1 – Encased Members, the calculation of the effective stiffness (EI_{eff} , AISC Eq. I2-6) is given as:

$$EI_{eff} = E_s I_s + E_s I_{sr} + C_1 E_c I_c (N - mm^2)$$

where E_c is the modulus of elasticity of concrete, E_s is the modulus of elasticity of steel, I_c is the moment of inertia of the concrete section about the elastic centroid of the composite section, I_s is the moment of inertia of steel shape about the elastic centroid of the composite section, I_{sr} is the moment of inertia of reinforcing bars about the elastic centroid of the composite section. In 2016, two changes were made to Eq. (I2-6). First, the portion of the reinforcement contributing to the EI has been increased from 50% to 100% (i.e., the 2010 edition equation had a 0.5 coefficient in front of the $E_s I_{sr}$ term); this makes a difference in encased shapes with large amounts of reinforcing bars. Second, the fraction (C_1) of the concrete stiffness that could be used has been substantially increased from:

$$C_1 = 0.1 + 2 \left(\frac{A_s}{A_c + A_s} \right) \leq 0.3 \text{ (AISC 2010)} \rightarrow C_1 = 0.25 + 3 \left(\frac{A_s + A_{sr}}{A_g} \right) \leq 0.7 \text{ (AISC 2016)}$$

where A_s is the area of the steel section, A_c is the area of concrete and A_{sr} is the area of reinforcing bars. The 2010 value of C_1 was overconservative, and the new expression provides substantial stiffness gains for encased members. For a member with a reinforcement ratio of 8%, the contribution of the concrete portion to the overall stiffness increases from 26% to 49% or almost double.

A complete design example for a slender beam-column encased section is given in the Appendix.

4. Similarly, in Section I2-2 – Filled Composite Members the fraction (C_3) of the concrete stiffness that could be used in the calculation of the effective stiffness has been changed from:

$$C_3 = 0.6 + 2 \left(\frac{A_s}{A_c + A_s} \right) \leq 0.9 \text{ (AISC 2010)} \rightarrow C_3 = 0.45 + 3 \left(\frac{A_s + A_{sr}}{A_g} \right) \leq 0.9 \text{ (AISC 2016)}$$

The effect on concrete-filled members is different than for the encased sections, as for some members this will result in an increase and in others a slight decrease of the equivalent moment of inertia. For example, for a member with a reinforcement ratio of 8%, the contribution of the concrete portion to the overall stiffness decreases from 76% to 69%. The changes in the C_1 and C_3 factors stem from extensive new studies that cover a much broader set of cases than those used in calibrating the 2010 Specification (Denavit et al. 2015).

5. In Section I5 – Combined Flexure and Axial Load, alternative explicit equations for the interaction between axial forces and flexure for filled composite members with non-compact or slender sections are given as:

$$\text{When } \frac{P_r}{P_c} \geq c_p \quad \frac{P_r}{P_c} + \frac{1 - c_p}{c_m} \left(\frac{M_r}{M_c} \right) \leq 1.0 \quad (I5-1a)$$

$$\text{When } \frac{P_r}{P_c} < c_p \quad \frac{1 - c_m}{c_p} \left(\frac{P_r}{P_c} \right) + \frac{M_r}{M_c} \leq 1.0 \quad (I5-1b)$$

where P and M are the axial and bending values, with the subscripts c referring to capacity and the subscripts r to the demand, and

$$c_{sr} = \frac{A_s F_y + A_{sr} F_{yr}}{A_c f'_c}$$

For rectangular filled composite members: $c_p = 0.17 c_{sr}^{-0.4}$

When $c_{sr} \geq 0.5$: $c_m = 1.06 c_{sr}^{-0.11} \geq 1.0$

When $c_{sr} < 0.5$: $c_m = 0.90 c_{sr}^{-0.36} \leq 1.67$

For round HSS filled composite members: $c_p = 0.27 c_{sr}^{-0.4}$

When $c_{sr} \geq 0.5$: $c_m = 1.10 c_{sr}^{-0.08} \geq 1.0$

When $c_{sr} < 0.5$: $c_m = 0.95 c_{sr}^{-0.32} \leq 1.67$

These curve-fitting equations reflect the bulging characteristics of the interaction equations at low axial loads and are based on extensive advanced finite element studies (Lai et al. 2014).

The 2016 AISC Seismic Provisions

The AISC 2016 *Seismic Specification* will contain a large number of changes for composite construction:

1. Chapter A (Materials): The overstrength factors (R_y and R_t) for typical tubes have been updated to reflect that the typical ASTM A500 hollow structural sections (HSS) will now have different values for Grades B or C. The R_y and R_t for Grade B will now be 1.3 and 1.2, respectively, while Grade C stays at 1.4 and 1.3, respectively. This is an important change because the previous very large yield overstrength factor (R_y) for A500 Grades B and C represented a big problem when trying to satisfy capacity requirements.
2. Chapter A: The new ASTM A1085 grade has been introduced with R_y and R_t of 1.2 and 1.1, respectively. ASTM A1085 has a nominal yield of 345 MPa, a maximum yield of 480MPa, and a nominal ultimate strength of 450Mpa. Its most important advantages are (a) the inclusion of a minimum Charpy value of 33Nm at 4.5°C, (b) the use of real wall dimensions (as opposed to 93% of nominal in most other HSS), and (c) the availability of rectangular section up to 600mmx300mmx19mm and round sections up to 500mmx16mm.
3. Chapter B (General Design Requirements): Extensive changes have been made throughout the Seismic Provisions to align them with the provisions of the new ASCE 7-16. Along those lines, attention of the designers is called to the differences between the load combination components for “overstrength seismic loads” and “capacity-limited seismic loads” and similar terminology used in ASCE 7.
4. Chapter B: A new Section B5 discusses diaphragms, chords and collectors. This reflects concerns that designers have not paid sufficient attention to the detailing of these components, which are key in delivering the inertial forces to the vertical lateral load resisting system (Sabelli et al. 2012). Since most floor systems in steel structures are composed of filled steel decks, the design of beam and truss connections to the vertical systems requires care in both assessing the demands and providing simple and logical load paths.
5. Chapter D (General Member and Connection Design Requirements): All references to ACI 318 have been updated to reflect the complete reorganization of that document. Concrete sections now need to comply with the requirements of ACI 318 Section 18.4 for moderately ductile members and ACI 318 Section 18.6 and 18.7 for highly ductile members.
6. Chapter D: The limits for local buckling of composite members have been slightly increased to better reflect the actual dimensions of these sections.
7. Chapter E (Moment Frame Systems): For special moment frames, the performance requirements have been rewritten to say that Special Moment Frames (SMF, Section E3) “designed in accordance with these provisions are expected to provide significant inelastic deformation capacity through flexural yielding of the SMF beams and limited yielding of column panel zones, *or, where equivalent performance of the moment frame system is demonstrated by substantiating analysis and testing, through yielding of the connections of beams to columns.*” The second part of this clause is new and philosophically extremely important because for the first time the use of connection deformation is recognized as a valid deformation mechanism for special moment frames. This validates the use of PRCC-type systems, a composite system which had been treated as a separate case in Part II.
8. Chapter F (Braced Frames and Shear Wall Systems): A brand new braced frame system, labeled the multi-tiered braced frame (MT-BFs), has been introduced. Multi-tiered braced frames are braced-frames with two or more tiers of bracing, or bracing panels between horizontal diaphragm levels or locations of out-of-plane support. MT-BFs are common in tall single-story building structures when it is not practical to use single bracing member spanning from roof to foundation levels (Figure 1).
9. Chapter F: The adjustment factors for the overstrength of buckling restrained braces (BRBs) now requires the use of reasonably accurate brace hardening models so as not to have unnecessarily large connection design forces.
10. Chapters G (Composite Moment Frames) and H (Composite Braced Frames and Shear Walls) contain the composite systems that were previously in Part II of the *Seismic Provisions*. In general, the provisions do not contain any major changes, but the provisions between composite and steel systems have been made more uniform and most conflicting provisions eliminated.

System Factors for Composite Structural System

As noted earlier, the 1994 edition of the BSSC recommendations contained a number of composite systems that were not common in practice and for which the system behavior factors were either taken directly from corresponding steel systems or chosen based on the expected performance. In the USA seismic codes,

system performance factors (SPFs) include base shear reduction (R), elastic deformation amplification (C_d) and system overstrength (Ω_0) factors.

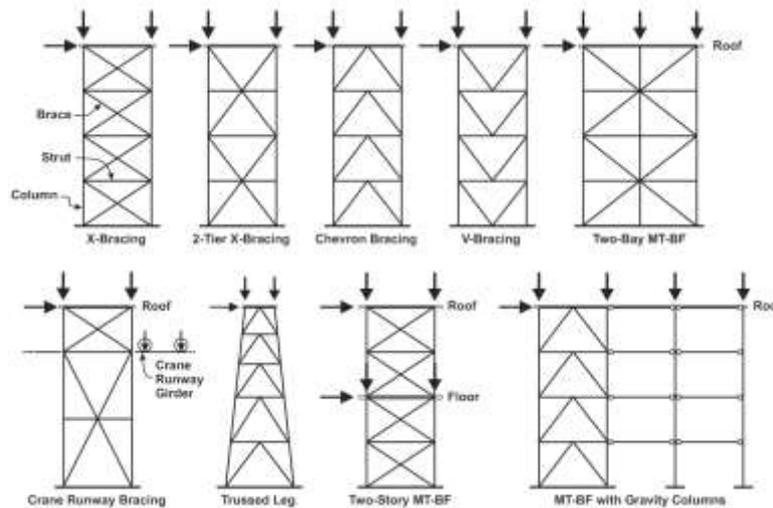


Figure 1 - Multi-tiered braced frame systems (©AISC).

In the last few years, a large effort in the USA has been dedicated to determining how to quantify systems performance factors. This effort culminated in 2009 in the issuance of a report entitled *Quantification of System Performance Factors* (FEMA P695, 2009). According to FEMA P695, life safety performance objectives can be achieved by enforcing a low probability of collapse of the seismic force resisting system when subjected to the Maximum Considered Earthquake (MCE) ground motions. The collapse can be either because of global instability of the system or partial instability. For global instability, if the system undergoes lateral dynamic instability or excessive lateral deformation, an impending sidesway collapse mechanism and failure are assumed. For partial collapse, if the local demand exceeds a component limit state criteria, a non-simulated collapse is assumed. FEMA P695 then determines a satisfactory collapse margin ratio (CMR) with respect to a surmised response modification coefficient (R factor). For a structural system that has established design provisions, a succinct collapse performance assessment process follows the flowchart shown in Fig. 2.

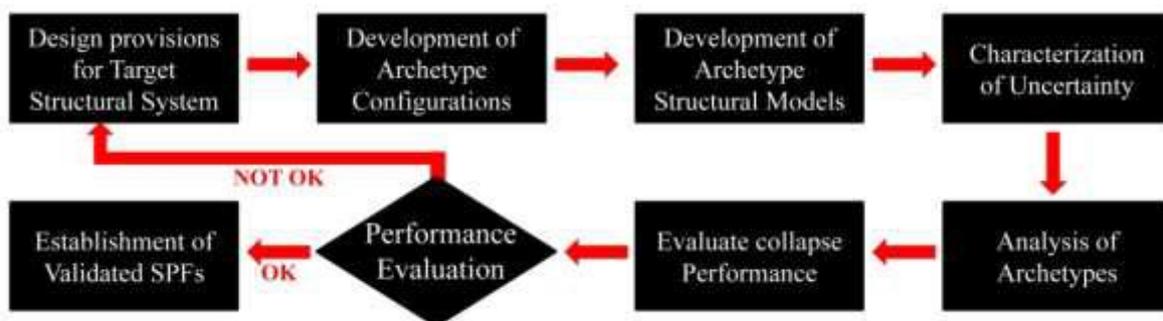


Figure 2 - Process for quantifying system performance factors (SPF)

Comprehensive studies of both PRCC (Bozorgmehr, 2012) and CSMF (Denavit et al. 2015) have been recently completed. The process starts with designing numerous different index archetypes according to current design provisions. These archetypes are intended to reflect a wide range of applicability, system attributes, and design parameters that can be anticipated to be crucial for system response. Each index archetype embodies key features related to collapse performance of a single specific frame, so that a generalized prediction of system behavior can be obtained. In the next step, analytical models are developed for each archetype in accordance with available component testing information both for key structural components (PR-CC connections in this case) and other structural elements that contribute to lateral resistance. The quality of available information and test data must be evaluated to characterize major sources of uncertainty in the analytical models. These uncertainties contribute to the variability in collapse capacity such that higher values of uncertainty will require higher collapse margin ratios in order to limit collapse probabilities to a given value (less than 10% in the case of FEMA P695). FEMA P695 then requires that a varied set of ground motions be utilized and scaled to carry out Incremental Dynamic Analyses (IDAs) of the archetype structures until collapse is recorded. For this study, nonlinear time history and pushover

analyses were performed with OpenSees 2.3.0 (PEER, 2006). It is essential that the 2D models reflect all possible collapse modes; in the case of PR-CCs only the shear failure in the connection was ignored because of the robustness of design requirements against this failure mode. In the next step, utilizing both the uncertainty level of the system and the results of nonlinear analyses, a collapse margin ratio (CMR) for each archetype is computed. Finally, the performance evaluation step is conducted in the light of two acceptable benchmarks: 1) a minimum collapse margin ratio for individual archetypes, and 2) a minimum collapse margin ratio for a Performance Group which is a selected group of archetypes with common basic structural properties and configurations. If the system performance is deemed satisfactory, then the systems performance factors assumed in the original design are considered to be adequate; if not, a new set of factors needs to be assumed and the process repeated until reasonable results are achieved. For the PRCC system, studies utilizing the current ASCE 7 system performance factors ($R=6$, $\Omega_0=3$ and $C_d=5.5$) indicated satisfactory performance.

A similar study was conducted in the last several years for composite frame systems that include both composite special moment frames (C-SMF) and composite special concentrically braced frames (C-SCBF) (Denavit M. , Hajjar, Leon, & Perea, 2015) (Denavit & Hajjar, 2014; Denavit M. , Hajjar, Perea, & Leon, 2013). A set of 60 archetype frames, selected to be representative of the range of composite frames seen in practice, were designed according to the AISC *Seismic Provisions* (AISC 2010). Using a suite of new finite element formulations for composite systems (Denavit and Hajjar 2014), nonlinear static pushover and transient dynamic analyses were performed on the frames to evaluate the seismic performance of the frames and assess the seismic performance factors that have been in place for many years in ASCE 7. The formulations included fiber-based beam finite elements and appropriate nonlinear models for the panel zones in C-SMFs and the gusset plate connection regions in C-SCBFs. The seismic performance factors for C-SMF systems are $R=8$, $\Omega_0=3$, and $C_d=5.5$, and for C-SCBF they include $R=5$, $\Omega_0=2$, and $C_d=4.5$. The archetype structures used in this study had building floor plans that were 3 bays by 5 bays, with a bay width of either 20 feet or 30 feet. The buildings were either 3 or 9 stories tall, with a story height of 13 feet. Each structure had two moment frames (three bays each) or two braced frames (one bay each) in each direction, with the remaining framing supported by leaning columns. The columns in the C-SMF systems were either rectangular concrete-filled steel tubes (RCFTs) or steel reinforced concrete (SRCs). The columns in the braced frames were assumed to be circular concrete-filled steel tubes (CCFTs). Steel wide-flange shapes were used for the girders and either steel HSS or steel wide-flange members were used for the braces. Two different concrete material strengths were used in the composite columns, including 4 ksi and 12 ksi concrete, while appropriate normal strength steel was used for all steel components. Fully-restrained connections and strong-column/weak-beam design were used in C-SMFs. Two levels of gravity loading were used to represent office live loading and warehouse live loading. The equivalent lateral force method (ASCE 2010) was used for the seismic design. Two different levels of seismic loading were used, corresponding to the levels design earthquake associated with the maximum (D_{max}) and minimum (D_{min}) of seismic design category D (ASCE 2010). The parametric study investigated both the use of $C_d = R$ and the current values of C_d . The beam element used in these studies for the composite and steel members is a distributed plasticity formulation developed specifically for steel-concrete composite frames. The element uses a mixed basis (i.e., using both displacements and forces as primary variables) to allow for accurate modeling of both material and geometric nonlinearities. The concrete and steel material models account for the salient features of each material, as well as the interaction between the two, including concrete confinement and local buckling. The results of the static and dynamic analyses showed that both composite frame systems exhibited excellent seismic behavior and current seismic performance factors were found to be acceptable. While the C-SCBF frames were generally governed by strength, the C-SMF frames were often governed by stiffness, and as such often had significant overstrength. Frames designed with the current deflection amplification factor, C_d , were found to be generally acceptable in this regard. However, a potential change to set increase C_d by setting $C_d = R$ within ASCE 7 should be considered further and possibly accompanied by a corresponding increase in the acceptable drift limits such that future seismic drift requirements are comparable to the current seismic drift requirements so as not to induce excessive overstrength in composite special moment frames.

Conclusions

This paper briefly reviews the development of composite steel-concrete seismic provisions in American codes. This development has been marked by remarkable freedom in generating up-to-date prescriptive design recommendations while maintaining designer flexibility. The 2016 edition of the AISC *Seismic Provisions* contains significant technical improvements as well as the symbolic recognition that composite systems are equivalent in performance to the conventional steel ones.

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Appendix – Example Design for a Composite Column

Calculate the axial strength of the shown encased composite SRC column with an effective length of $KL = 12$ m. Assume the column is continuously braced about the minor axis.

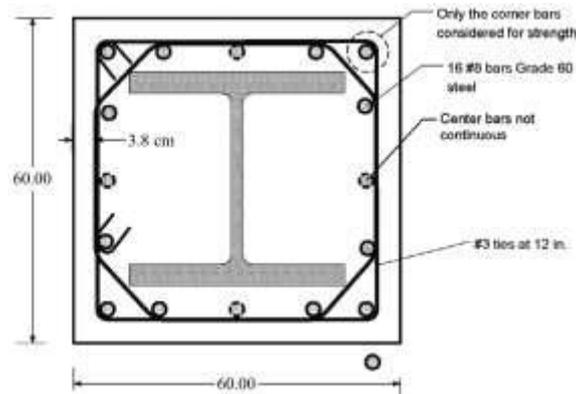


Figure. SRC section used in example (not scaled)

The concrete cross-section is 60 cm. x 60 cm. ($f'_c = 35$ MPa, $E_c = 28$ GPa), which is encasing an ASTM A992 ($F_y = 345$ MPa, $E_s = 200$ GPa) steel section W360x314 [W14x211] ($A_s = 400$ cm.², $I_s = I_x = 111,000$ cm.⁴, $Z_x = 6,390$ cm.³), and 16-25M [16-#8] ASTM A706 ($F_{yr} = 415$ MPa, $E_s = 200$ GPa) bars distributed along the perimeter of the section providing a rebar reinforcement ratio of approximately 2.2% (see Figure). Many of these reinforcing bars will be needed to maintain confinement and rebar spacing requirements, and will not necessarily be continuous through the joint due to the presence of framing beams. In this example, only the four corner bars, located at a distance of $e_1 = 24$ cm. from the column centerline, will be assumed as continuous and used in the strength calculations [$A_{sr} = 4(5.1$ cm.²) = 20.4 cm.²].

1. Determine cross-section properties:

$$A_c = A_g - A_s - A_{sr} = 60^2 - 400 - 20.4 = 3,179.6 \text{ cm.}^2$$

$$I_{sr} \approx A_{sr} e_1^2 = 20.4(24)^2 = 11,750.4 \text{ cm.}^4$$

$$I_c = I_g - I_s - I_{sr} = 60^4/12 - 111000 - 11750 = 957,250 \text{ cm}^4$$

2. Determine the coefficient C_1 :

$$C_1 = 0.1 + 2 \left(\frac{A_s}{A_g} \right) = 0.1 + 2 \left(\frac{400}{3600} \right) = 0.322 > 0.3 \rightarrow C_1 = 0.3$$

3. Compute the effective stiffness:

$$EI_{eff} = E_s I_s + 0.5 E_s I_{sr} + C_1 E_c I_c$$

$$EI_{eff} = 200(111000) + 0.5(200)(11750) + 0.3(28)(957250) = 32,011,559 \text{ GPa-cm}^4$$

$$EI_{eff} = 320,116 \text{ kN-m}^2$$

4. Compute the Euler load:

$$P_e = \frac{\rho^2 EI_{eff}}{(KL)^2} = \frac{\rho^2 (320,116 \text{ kN-m}^2)}{(12 \text{ m})^2} = 21,940.4 \text{ kN}$$

5. Calculate the squash load:

$$P_{no} = A_s F_y + A_{sr} F_{yr} + 0.85 A_c f'_c$$

$$P_{no} = 400(345) + 20.4(415) + 0.85(3600)(35)$$

$$P_{no} = 241,059 \text{ MPa} \times \text{cm}^2 = 24,105.9 \text{ kN}$$

6. Calculate the slenderness parameter:

$$\frac{P_{no}}{P_e} = \frac{24,105.9 \text{ kN}}{21,940.4 \text{ kN}} = 1.099$$

$$l = \sqrt{\frac{P_{no}}{P_e}} = \sqrt{1.099} = 1.048$$

7. Calculate the strength reduction factor, χ , due to slenderness effects:

Since $P_{no}/P_e < 2.25$ (or $\lambda < 1.5$), the nominal strength is controlled by the inelastic buckling equation.

$$C = 0.658^{P_{no}/P_e} = 0.658^{1.12} = 0.631$$

8. Calculate the nominal compressive strength:

$$P_n = C P_{no} = 0.631(24,105.9 \text{ kN}) = 15,219.8 \text{ kN}$$

9. Calculate the axial strength for points C and D:

$$P_C = 0.85 A_c f'_c = 0.85(3600)(35) =$$

$$= 94,593.1 \text{ MPa} \times \text{cm}^2 = 9459.31 \text{ kN}$$

$$P_D = \frac{0.85 A_c f'_c}{2} = \frac{9459.31 \text{ kN}}{2} = 4729.65 \text{ kN}$$

10. Calculate nominal strength for points C and D:

$$P_{nC} = C P_C = 0.631(9459.31 \text{ kN}) = 5972.33 \text{ kN}$$

$$P_{nD} = C P_D = 0.631(4729.65 \text{ kN}) = 2986.17 \text{ kN}$$

10. Calculate the nominal flexural strength for point D:

$$Z_r = A_{sr} e_1 = 20.4(24) = 489.6 \text{ cm}^3$$

$$Z_c = \frac{bh^2}{4} - Z_{sx} - Z_r = \frac{60^3}{4} - 6390 - 489.6$$

$$= 47,120.4 \text{ cm}^3$$

$$M_{nD} = Z_{sx} F_y + Z_r F_{yr} + \frac{1}{2} Z_c (0.85 f'_c)$$

$$= 6390(345) + 489.6(415) + \frac{1}{2}(47120.4)(35)$$

$$= 3,108,650 \text{ MPa} \times \text{cm}^3 = 3,108.6 \text{ kN} \times \text{m}$$

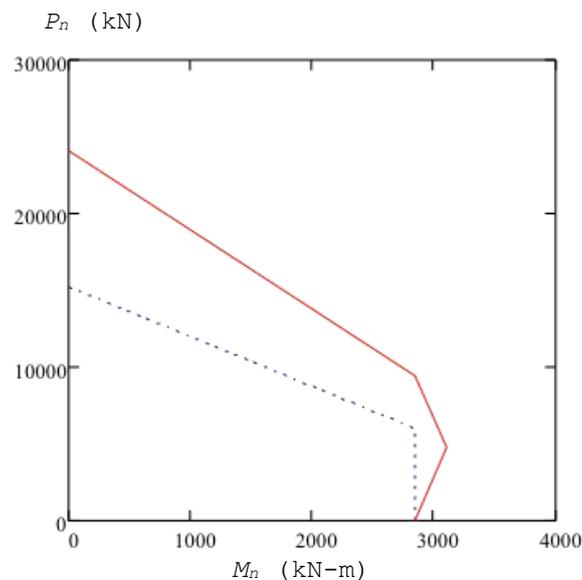
10. Calculate the nominal flexural strength for point C:

$$h_n = 13.79 \text{ cm}$$

$$Z_{sn} = t_w h_n^2 = 473.72 \text{ cm}^3$$

$$Z_{cn} = b h_n^2 - Z_{sn} = 11,111.83 \text{ cm}^3$$

$$M_{nB} = M_{nD} - Z_{sn} F_y - \frac{1}{2} Z_{cn} (0.85 f'_c) = 2,383 \text{ kN} \cdot \text{m}$$



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