FOURTH BALLOT PROPOSAL 6-4R3 (2009)

SCOPE: Part 1, *Provisions*, Sec. 14.1.4.1 and Part 2, *Commentary*, Sec. C14.1.4.1, as approved in the third ballot

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THIRD BALLOT RESULTS:

Proposal 6-4R2 (Y=19, YR= 1, N= 10, NV= 15 --67%)

A technical paper (included at the end of the 4th ballot version of this proposal) was distributed to the PUC for review prior to consideration of the ballot results and the TS6 responses.

HSEAC (No): The reason for the proposal states that AISI S110 intends that the R = 3.5 be utilized only for preliminary sizing and not for final design. However, there are no conditional statements to that effect in the proposed new entry in Table 12.2.-1. Nevertheless, it would appear that there is more uncertainty in this particular system's R factor than for other systems.

> ***TS6 Response: Nonresponsive or Editorial.*** *This comment relates to the reason statement and not the provisions being balloted. However, see Item 2 under the SEAoCC comments.*

The PUC agreed that this comment was nonpersuasive (Y = 20, N = 0, NV = 1). It was agreed, however, that the reason statement was misleading and should not have included the text cited by the commenter. A modified reason appears as part of the fourth ballot proposal.

AISI (YR): Consider making the following modifications:

1. Section 14.1.4.1.1: Instead of "grid lines" in Line 33, use the more widely accepted terminology "independent lines of resistance."

   TS6 Response: Editorial. *Change has been made.*

2. Section 14.1.4.1.4, Line 16: Based upon additional review of the system, it is recommended that the limit on width-thickness ratio be reduced from 1.58xsqrt(E/Fy) to 1.40xsqrt(E/Fy) for HSS sections. The purpose of this reduction is to take into account the fact that axial load was not applied to the column during testing. According to the 2005 AISC Specification, the values of lambda(p) and lambda(r) for stiffened elements of HSS sections subjected to uniform compression due to bending or compression are 1.22xsqrt(E/Fy) and 1.40xsqrt(E/Fy). The test results of two specimens (Specimens 7 and 9) were compared. Specimens 7 and 9 were classified as noncompact and slender sections, respectively, according to the AISC Specification. Based upon the performance of Specimen 9, it is recommended that the width-thickness ratio be reduced from 1.58xsqrt(E/Fy) to 1.40xsqrt(E/Fy) for HSS sections.

   TS6 Response: Persuasive. *Change has been made.*

3. Section 14.1.4.1.5, Line 28: Modify the sentence to read, "*Where a drift limit is required by the applicable building code*, the design story drift shall not exceed *0.03h*, unless approved by the authority having jurisdiction." The purpose of the reference to the applicable building code is to take into consideration Footnote C of ASCE 7 Table 12.12-1, which states the following for Occupancy Category I and II buildings: "cThere shall be no drift limit for single-story structures with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts. The structure separation requirement of Section 12.12.3 is not waived."

TS6 Response: Persuasive. *Change has been made.*

Rather than adopt the change recommended in Comment 1 above, the PUC preferred the following language to ensure that all the major columns of SFRS are detailed as SBMF: "The SBMF shall engage all columns supporting the roof or floor above." The PUC agreed with the change proposed in Comment 2.
Rather than adopt the change recommended in Comment 3, the PUC preferred to make the following modifications to AISI S110, Section D1.3:

- *The first sentence was deleted, since it was not necessary.*
- *The last sentence was moved into the first paragraph.*
- *"In no case shall the design story drift exceed 0.05h" was added to ensure an absolute upper bound on the drift limit.*

The PUC voted to modify the proposal as described above and to submit the proposed changes for reballoting by the BSSC member organizations (Y = 20, N = 0, NV = 1).

SEAoCC (No): Negative vote will be changed to affirmative if this entire proposal is place in Part 3 instead of Part 1.

TS6 Response: Nonpersuasive. *It was the intent of the ballot that this system and the associated requirements in AISI S110 be included in Part 1. Please note, placing the proposal in Part 3 does not preclude the system from being considered for adoption in ASCE 7. Also, it is worth noting that this type of structure has been in use since the early 60's and several thousand have been installed in California and through the western region for use as mezzanines, support platforms and as specialty support structures for the aero-space industry and even in state and city facilities. These structures, of many different sizes and configurations, have been installed in high seismic zones through California and have experienced most of the major earthquakes without any evidence of structural damage. The ductility of this type of structure has been helpful in the dissipation of the seismic energy and the purpose of the UCSD testing was to add tested evidence of the structures' performance in earthquakes and aid in the standardization of the system. It is our hope to reference AISI S110 directly in ASCE 7 and the model building codes instead continuing to use the alternate means and methods section of the building code for this type of structure.*

The PUC agreed with TS6 that this comment was nonpersuasive (Y = 19, N = 0, NV = 2).

1. This is a moment connection based upon slip and bolt bearing using snug tight HS bolts loaded in shear only. The field tightened snug tight bolts could result in highly variable frictional force with variable energy dissipation. The bolted connection occurs only in the web of the beam and column and not the flanges. The primary mechanism is bolt hole elongation due to bearing failure in the cold-formed light gage metal.

TS6 Response: Nonpersuasive. *If bolt over-tightening beyond the snug-tight condition were to occur, it is expected that the story drift will be reduced, which to some extent will offset the increase of the maximum moment developed in the bolted group. To examine the effect of over-tightening on the maximum moment, a sensitivity study through nonlinear time-history analyses of a typical CFS-SBMF was conducted; a suite of 20 earthquake ground motions was used as the input motion. The analysis results are provided in the attached document. Assuming that over-tightening resulted in a 50% increase of the bolt tension, it was concluded that, on average, the maximum story drift was reduced by 3%, while the maximum base shear (or the maximum moment in the bolt group) was increased by 11%. The analysis results also showed that the scatter (i.e., the standard deviation) of the predicted response was reduced when the bolts were over-tightened.*

It should be noted that the bolt grip length in a CFS-SBMF is very small, thereby the increase in the bolt tension due to over-tightening is also expected to be limited. Together with the observation made from the sensitivity study, it is concluded that the impact of over-tightening is insignificant.

2. The Reason for Proposal states that the R = 3.5 is for preliminary design only citing a more comprehensive design procedure in AISI S110. This is not typical of all other steel seismic force resisting systems in ASCE 7 and AISC 341 in which a direct design approach is used based upon the applicable R in the code.

   TS6 Response: Nonresponsive or Editorial. *This comment relates to the reason statement, not the provisions. The reason statement was unintentionally confusing. It is the intent of the standard that R = 3.5 be used in design, and the designer does not need to iterate on the R value. The value of R was developed based on the high ductility capacity observed from cyclic testing of 9 full-scale beam-column subassemblies. The reason statement has been corrected.*

3. The connection is supposed to be the weak link in this system and is used to protect the column and beam.

   TS6 Response: *Agree.*

   The lack of a specific tension force in the HS bolts could result in excessive frictional forces which when combined with bearing exceed the column or beam design moment capacity.

   TS6 Response: *See response to Comment 1.*

   The column and beam moment capacity is limited by local buckling of the cold-formed steel members which provide little energy dissipation.

   TS6 Response: *This has been considered and is the reason why capacity design is required and the reason for the dimensional limitations on the system's beams and columns.*

   There is a concern that the gravity load carrying portion of the connection is also the primary energy dissipation system for earthquake loads.

   TS6 Response: *Within ASCE 7, Table 12.2-1, this is not unique – consider bearing wall systems and other moment frames.*

4. The $0.03h_x$ design story drift limit set for this seismic force resisting system in Section 14.1.4.1.5 does not comply with ASCE 7 Table 12.12-1 Allowable Story Drift.

   TS6 Response: Nonpersuasive*. The original value determined from the ATC-63 90% draft study was $0.05h_x$. The more conservative $0.03h_x$ drift limit was recommended and agreed to by the PUC in response to a comment by Aschheim on an earlier version of this proposal (Proposal 6-4R): "TS 6 Response: Persuasive. According to Section 12.12.1 in ASCE 7-05, the allowable drift is 0.025 of the story height for Occupancy Category I or II building structures 4 stories or less with interior walls, partitions, ceilings and exterior wall systems that have been designed to accommodate the story drifts. For nonbuilding structures, Section 15.4.5 stipulates that drift limit in Section 12.12.1 need not apply if a rational analysis indicates that they can be exceeded without adversely affecting structural stability or attached or interconnected components. Considering that AISI S110 is intended for industrial platforms and that these frames usually do not have partitions and walls, it is suggested that the 0.05h drift limit in Section D1.3 of AISI S110 be reduced to 0.03h."*

5. As the documentation states, this connection and system was intended for industrial platforms and non-building structures and should not be applied to occupied building structures without further study.

TS6 Response: Nonpersuasive. *Research did not distinguish between occupied and unoccupied spaces. Therefore, there is no reason to limit applicability of system at this time.*

With respect to Comment 1, the PUC indicated that this system specifies snug-tight bolts for which the nut is turned by the ironworker's spud wrench. It is anticipated that there will be a small pretension in the bolt which will help prevent the nut from backing off, and 10 kips is considered a realistic estimate (and much less than the usual pretension force in a bolt). (See AISI S110, Section D1.2.3.1(1).) This was confirmed by Drs. Uang and Sato who used the trial-and-error procedure to determine the bolt tension for slip resistance. The cyclic responses of 9 full-scale beam-column subassemblies were used for calibration purposes. For bolt tension (T), they observed that T = 10 kips correlated well with the response of Specimens 1 to 7 and T= 25 kips correlated well with the two larger specimens (Specimens 8 and 9).

If bolt over-tightening beyond the snug-tight condition were to occur, it is expected that the story drift will be reduced, which to some extent will offset the increase of the maximum moment developed in the bolted group. To examine the effect of over-tightening on the maximum moment, a sensitivity study through nonlinear time-history analyses of a typical CFS-SBMF was conducted; a suite of 20 earthquake ground motions was used as the input motion. The analysis results are provided in the attached document. Assuming that over-tightening resulted in a 50% increase of the bolt tension, it was concluded that, on average, the maximum story drift was reduced by 3%, while the maximum base shear (or the maximum moment in the bolt group) was increased by 11%. The analysis results also showed that the scatter (i.e., the standard deviation) of the predicted response was reduced when the bolts were over-tightened.

It should also be noted that the bolt grip length in a CFS-SBMF is very small, consequently the increase in the bolt tension due to over-tightening is also expected to be limited. The grip length is critical, because that is the length over which the bolt is strained during the pretensioning procedure. For joints in hot-rolled construction one may have grip lengths of 2 to 4 inches and more; the specifics depend on the structure in question. For such bolt grip lengths the strain can be sufficiently accommodated to produce the requisite force. For joints in cold-formed construction, however, the grip lengths will normally be much smaller than one inch, and the pretensioning strain and the force simply cannot be developed. If one attempts to turn the nut as specified for hot-rolled construction, the strain can only be accommodated through deformations along the threads of the bolt, and the threads will be quickly sheared off (the threads are stripped). So the end result is that there will be no pretension - in fact, there won't be a bolt either.

Therefore, given the small bolt grip length in combination with the observations made from the sensitivity study, it is concluded that the impact of over-over-tightening is insignificant. Finally, it is worth noting that inspection of bolted connection is covered in AISI S110, Section E4 as follows: **"E4 Inspection of Bolted Connections.** *Connections shall be inspected to verify that the fastener components are as specified and that the joint plies have been drawn into firm contact. A representative sample of bolts shall be evaluated using an ordinary spud wrench, to assure that the bolts in the connections have been tightened to a level equivalent to that of the full effort of a worker with such wrench."*

With respect to Comment 2, the PUC agreed that the Reason for Proposal can be adjusted as proposed and considered it editorial.

The PUC agreed with the TS6 responses on Comment 3. The third comment had four parts. The PUC found the first part persuasive and the following three parts nonpersuasive.

The PUC agreed with the TS6 response that Comment 4 is nonpersuasive. However, TS 6 noted that language will be added to the commentary of AISI S110 cautioning the user that finishes and nonstructural items need to be designed to accommodate this larger anticipated drift.

The PUC agreed with the TS6 response that Comment 5 is nonpersuaisive.

The PUC vote on the comments as described above was Y =20, N= 0, NV =1.

SEAoSD (No): Same as SEAoCC.

TS6 Response: Nonpersuasive. *See the above response to SEAoCC.*

CMACN (No):
Position: Concur with SEAOCC.

TS6 Response: Nonpersuasive. *See the above response to SEAoCC.*

Negative vote will be changed to affirmative if this entire proposal is placed in Part 3 instead of Part 1

1. This is a moment connection based upon slip and bolt bearing using snug tight HS bolts loaded in shear only. The field tightened snug tight bolts could result in highly variable frictional force with variable energy dissipation. The bolted connection occurs only in the web of the beam and column and not the flanges. The primary mechanism is bolt hole elongation due to bearing failure in the cold-formed light gage metal.

2. The Reason for Proposal states that the R =3.5 is for preliminary design only citing a more comprehensive design procedure in AISI S110. This is not typical of all other steel seismic force resisting systems in ASCE 7 and AISC 341 in which a direct design approach is used based upon the applicable R in the code.

3. The connection is supposed to be the weak link in this system and is used to protect the column and beam. The lack of a specific tension force in the HS bolts could result in excessive frictional forces which when combined with bearing exceed the column or beam design moment capacity. The column and beam moment capacity is limited by local buckling of the cold-formed steel members which provide little energy dissipation. There is a concern that the gravity load carrying portion of the connection is also the primary energy dissipation system for earthquake loads.

4. The 0.03hx design story drift limit set for this seismic force resisting system in Section 14.1.4.1.5 does not comply with ASCE 7 Table 12.12-1 Allowable Story Drift.

5. As the documentation states, this connection and system was intended for industrial platforms and non-building structures and should not be applied to occupied building structures without further study.

SEAoSC (No): Same as CMACN.

TS6 Response: Nonpersuasive. *See the above response to SEAoCC.*

SEAoNC (No): Same as CMACN.

TS6 Response: Nonpersuasive. *See the above response to SEAoCC.*

NCSEA (No): Same as CMACN

TS6 Response: Nonpersuasive. *See the above response to SEAoCC.*

SEAoC (No): Same as CMACN.

TS6 Response: Nonpersuasive. *See the above response to SEAoCC.*

The PUC agreed with TS 6 that the comments from SEAoSD, CMACN, SEAoSC, SEAoNC, and SEAoC were similar to those of SEAoCC and therefore the PUC decision regarding the SEAoCC comments applies. No PUC vote was needed since no PUC member objected to the TS6 position.

CASSC (No): The assignment of R values for this moment frame system and the performance characteristics are inconsistent with special moment frames. Therefore it should be prohibited from use in SDC D, E, and F and not be called "special."

> ***TS6 Response: Nonpersuasive.*** *The term "Special" used in ASCE 7 Table 12.2-1 implies a high ductility capacity built into the system. The high value (= 8) of R for the Special Moment Frame is partially due to the high system overstrength resulting from the redundant, multistory nature of the frame. The proposed CFS-SBMF system is for one-story application, therefore the system overstrength is low because the frame is expected to behave as a single-degree-of-freedom system. The low value of R (= 3.5) assigned to the CFS-SBMF is consistent with the low system overstrength of the frame. But the ductility capacity is comparable to that in a Special Moment frame.*

The proposed use of an R value for preliminary design while referencing a more sophisticated design method in AISI 110 for final design is not consistent with other similar seismic force resisting systems.

> ***TS6 Response: Nonpersuasive or Editorial.*** *See the above response to SEAoCC Comment 2.*

The potential consequences of over-tightening bolts beyond snug tight require clarification. The requirements should specify a means of preventing over-tightening.

> ***TS6 Response: Nonpersuasive.*** *See the above response to SEAoCC.*

The PUC agreed with the TS6 responses that the CASSC comments are nonpersuasive (Y = 19, N = 0, NV = 2) and that the second and third comments are similar to the SEAoCC comments.

AMCBO (No): The vote for this proposal shall change to "Y" if the following issues are addressed:

1. Typo errors are corrected @ line 16 to reflect proposal that "Overstrength Factor" = 3.0 and "Deflection Amplification Factor" = 3.5.

> ***TS6 Response: Nonpersuasive.*** *That is what the proposal currently states.*

BSSC Staff Note: *It appears on some paper copies that the decimal point was missing on both 3.0 and 3.5. The original proposal had yellow highlighting and it was lightened in printing which inadvertently eliminated or faded the decimal points.*

2. Field tightened "snug tight" bolt without specific tension force will result unpredictable surface contact friction and energy dissipation. If bolts are over-tensioned, it may result in excessive frictional forces and could exceed the beam or column design moment capacity controlled by local buckling of cold-formed steel members with little ductility. Specific tension force or torque in bolt tightening should be prescribed.

> ***TS6 Response: Nonpersuasive.*** *See response to SEAoCC.*

The PUC agreed with the TS6 responses that these comments were either editorial or nonpersuasive (Y = 20, N = 0, NV = 1).

Unsolicited PUC Member Comment: This proposal needs a lot of work.
P1 Ln 30: We use the term "herein amended." Do we really need to appear to write like attorneys? This should be deleted.

> ***TS6 Response: Nonpersuasive****.* *The phrase communicates the requirements succinctly.*

P2 Ln 29 We need to define "significant".

TS6 Response: ***Editorial.*** *Deleted "significant" – it is not needed.*

P2 Ln 33: The use of at all "grid lines" should be either defined or modified.

TS6 Response: ***Editorial.*** *See response to AISI Bullet #1 for modification.*

P2 Ln 36: Why does this need to be on a "level floor". Is a foundation not appropriate?

TS6 Response: ***Editorial.*** *Yes, this was the intent, so "or foundation" has been added.*

P3 Ln 23 Why are we referencing the "building code" as opposed to ASCE 7. The IBC references the ASCE 7.

TS6 Response: ***Nonpersuasive.*** *AISI standards policy is to reference the Applicable Building Code, which is the overriding document.*

P 3 Ln 26: We should reference ASCE 7 not "building code".

TS6 Response: ***Nonpersuasive.*** *AISI standards policy is to reference the Applicable Building Code, which is the overriding document.*

> *Although these comments did not necessarily need to be considered, the PUC agreed with the TS6 responses.*

FOURTH BALLOT PROPOSAL FOR CHANGE

Revise Part 1 *Provisions* Sec. 14.1.4.1 as approved in the third ballot as follows:

14.1.4.1 Modifications to AISI S110 (2007 edition). The text of AISI S110 shall be modified as indicated in Sections 14.1.4.1.1 through 14.1.2.1.5. Italics are used for text within Sections 14.1.4.1.1 through 14.1.2.1.X to indicate requirements that differ from AISI S110.

14.1.4.1.1 AISI S110, Section D1. Modify Section D1 by revising to read as follows.

D1 Cold-Formed Steel Special Bolted Moment Frames (CFS-SBMF)

Cold-formed steel–special bolted moment frames (CFS-SBMF) systems shall withstand ~~significant~~ inelastic deformations through friction and bearing at their bolted connections. Beams, columns, and connections shall satisfy the requirements in this section. CFS-SBMF systems shall be limited to one-story structures, no greater than 35 feet in height, without column splices and satisfying the requirements in this section. *The SBMF shall engage all columns supporting the roof or floor above ~~be used in all grid lines in the direction under consideration~~. The single size beam and single size column with the same bolted moment connection detail shall be used for each frame. The frame is to be supported on a level floor or foundation.*

14.1.4.1.2 AISI S110, Section D1.1.1. Modify Section D1.1.1 by revising to read as follows.

D1.1.1 Connection Limitations

Beam-to-column connections in CFS-SBMF systems shall be bolted connections with snug-tight high-strength bolts. The bolt spacing and edge distance shall be in accordance with the limits of AISI S100, Section E3. The 8-bolt configuration shown in Table D1-1 shall be used. The faying surfaces of the beam and column in the bolted moment connection region shall be free of any lubricants or debris.

14.1.4.1.3 AISI S110, Section D1.2.1. Modify Section D1.2.1 by revising to read as follows.

D1.2.1 Beam Limitations

In addition to the requirements of Section D1.2.3, beams in CFS-SBMF systems shall be *ASTM A653 Gr. 55 galvanized steel* cold-formed C-sections members with lips, and designed in accordance with Chapter C of AISI S100. *The beam depth shall be between 12 in (305 mm) and 20 in (508 mm).* The flat depth-to-thickness ratio of the web shall not exceed 6.18 $\sqrt{E/F_y}$.

14.1.4.1.4 AISI S110, Section D1.2.2. Modify Section D1.2.2 by revising to read as follows.

D1.2.2 Column Limitations

In addition to the requirements of D1.2.3, columns in CFS-SBMF systems shall be ASTM A500 Gr. B cold-formed hollow structural section (HSS) members painted with a standard industrial finished surface, and designed in accordance with Chapter C of AISI S100. The column depth shall be between 8 in (203 mm) and 12 in (305 mm). The flat depth-to-thickness ratio shall not exceed ~~1.58~~ 1.40 $\sqrt{E/F_y}$.

14.1.4.1.5 AISI S110, Section D1.3. Modify Section D1.3 by revising to read as follows.

D1.3 Design Story Drift

~~*The design story drift, Δ, when subjected to the forces resulting from the motion of the design earthquake, shall be computed in accordance with the applicable building code.*~~ Where the applicable building code does not contain design coefficients for CSF-SBMF systems, the provisions of Appendix 1 shall apply. The design story drift shall not exceed *0.03h*, unless approved by authority having jurisdiction. In no case shall the design story drift exceed 0.05h.

For structures having a period less than T_S, as defined in the applicable building code, alternate methods of computing Δ shall be permitted, provided such alternate methods are acceptable to the authority having jurisdiction.

~~The design story drift shall not exceed 0.03 h unless approved by the authority having jurisdiction.~~

Revise Part 2 *Commentary* Sec. C14.1.4.1 as approved in the third ballot as follows:

~~**C14.1 STEEL**~~

~~This general discussion invokes both the language of ASCE7 and the referenced standards for the design of steel structures for seismic resistance.~~

~~**C14.1.1 Reference Documents.**~~

~~This section lists a series of structural standards published by AISC, AISI, ASCE and SJI that are to be applied in the seismic design of steel members and connections in conjunction with the requirements of ASCE 7-05.~~

~~**C14.1.2 Seismic Design Categories B and C.**~~
~~In the lower Seismic Design Categories B and C, the Engineer is allowed a choice in the design of a steel lateral force resisting system. The first option is to design the structure to meet the design and detailing provisions required for structures in higher Seismic Design Categories, with the commensurate seismic design parameters (R, Cd and Omega-zero). The second option is to use a lower R factor of 3 (and higher resulting base shear) but not follow the seismic design and detailing provisions. The concept of this option is that the higher base shear force will compensate for the reduced ductility of the members and connections resulting in similar levels of performance in these lower Seismic Design Categories.~~

~~**C14.1.3 Seismic Design Categories D through F.**~~
~~In the higher Seismic Design Categories, the Engineer is not given a choice, but must follow the seismic design provisions of either AISC or AISI using the seismic design parameters specified for the chosen structural system. It is not considered appropriate to design structures without specific design for seismic response in these high seismic design categories.~~

C14.1.4 Cold-Formed Steel. This section adopts three standards by direct reference --

AISI NAS, North American Specification for the Design of Cold-Formed Steel Structural Members,

AISI S110, Standard For Seismic Design Of Cold-Formed Steel Structural Systems – Special Bolted Moment Frames, and

ASCE 8, *Specification for the Design of Cold Formed Stainless Steel Structural Members.*

Each document has specific limits of applicability. AISI NAS applies to the design of structural members that are cold-formed to shape from carbon or low-alloy steel sheet, strip, plate or bar not more than one-inch in thickness. **[AISI NAS:** A1.1] Building on the requirements of AISI NAS, AISI S110 has additional special seismic design provisions for a newly designated seismic force resisting system entitled "cold-formed steel – special bolted moment frame (CFS-SBMF)". Finally, ASCE 8 governs the design of structural members that are cold-formed to shape from annealed and cold-rolled sheet, strip, plate, or flat bar stainless steels. [**ASCE 8:** 1.1.1] All three documents focus on load-carrying members in buildings; however, allowances are made for applications in nonbuilding structures, if dynamic effects are appropriately considered.

Within AISI NAS and ASCE 8, there are requirements on the general provisions for the applicable types of steel; design of elements, members, structural assemblies, connections and joints; and mandatory testing. In addition, AISI NAS contains a chapter on the design of cold-formed steel structural members and connections undergoing cyclic loading. Both standards contain extensive commentaries for the benefit of the user.

<u>Design requirements for cold-formed steel – special bolted moment frames (CFS-SBMF) are based upon research (Uang and Sato, 2007) at UCSD. This system is intended to withstand inelastic deformations through friction and bearing at the frame's bolted connections, with beams, columns, and beam-to-column connections sized accordingly. Currently, CFS-SBMF systems are limited to one-story structures, no greater than 35 feet in height. Although the 2007 edition of AISI S110 addresses only CFS-SBMF systems, it is anticipated that the scope of later editions will be expanded through research on additional types of cold-formed steel seismic force-resisting</u>

systems. Like the other two documents listed in this section, AISI S110 contains an extensive commentary to aid the user in the design and construction of CFS-SBMF systems.

~~The first edition of AISI S110 (2007) focuses on design requirements for cold-formed steel – special bolted moment frames (CFS-SBMF) systems only. Based upon research (Uang and Sato, 2007) at UCSD, this system is intended to withstand inelastic deformations through friction and bearing at the frame's bolted connections, with beams, columns, and beam-to-column connections sized accordingly. Currently, CFS-SBMF systems are limited to one-story structures, no greater than 35 feet in height. It is anticipated that later editions of this standard will be expanded in scope through research on additional types of cold-formed steel seismic force resisting systems. Like the other two documents listed in this section, this document contains an extensive commentary to aid the user in the design and construction of CFS-SBMF systems.~~

C14.1.4.1 Modifications to AISI S110 (2007 edition)

C14.1.4.1.1 ~~CFS-SBMF of the same-size beams and same-size columns are expected to be used along each grid line in the direction under consideration.~~ CFS-SBMF need to use the same-size beams and same-size columns throughout. In addition, the system needs to engage all primary columns, which support the roof or floor above, and those columns need to be supported on a level floor or foundation.

C14.1.4.1.2 These modifications were made for consistency with the test database.

C14.1.4.1.3 To be consistent with the test database (Uang and Sato, 2007), the limitations on both beam depth, steel grade, and surface treatment are added in Section D1.2.1 of AISI S110.

C14.1.4.1.4 To be consistent with the test database (Uang and Sato, 2007), the limitations on column depth, steel grade, and surface treatment are added in Section D1.2.2 of AISI S110. The width-thickness ratio was reduced based upon further review of the test specimens.

C14.1.4.1.5 AISI S110 is intended primarily for industrial platforms; however, the standard is not limited to these non-building structures and does not prohibit architectural attachments (such as partition walls). Therefore, the 0.05h drift limit in Section D1.3 of AISI S110 has been reduced to 0.03h to more closely align with the 0.025h drift limit of ASCE 7. The sentence, "In no case shall the design story drift exceed 0.05h." was added to ensure an absolute upper bound on the drift limit.

REASON FOR FOURTH BALLOT PROPOSAL

Proposal 6-4R3 presents changes that respond to comments from the third member organization ballot including the Reason for Proposal.

This proposal introduces the first edition of AISI S110, *Standard For Seismic Design Of Cold-Formed Steel Structural Systems – Special Bolted Moment Frames*, which is based upon research conducted by Drs. Uang and Sato at UCSD (2007). Specifically, the document focuses on providing design provisions for a newly defined seismic force resisting system entitled "Cold-formed Steel – Special Bolted Moment Frame" or CFS-SBMFs. This type of system is expected to experience substantial inelastic deformation during significant seismic events. It is intended that most of the inelastic deformation will take place at the bolted connections, due to slip and bearing. In order to develop the designated mechanism, requirements based on the capacity design principles are provided for the design of the beams, columns and associated connections. Additionally, the document has specific requirements for the determination of story drift and the application of quality assurance and quality control procedures.

In Appendix 1, AISI S110 makes initial recommendations for the seismic design coefficients of the CFS-SBMF system. The Response Modification Coefficient, R, is set at 3.5. Cyclic testing has shown that CFS-SBMFs have very large ductility capacity and significant hardening. This justifies the use of a value of 3.5 for the R-factor. The derivation of the deflection amplification factor, C_d, can be found in the AISI S110 Commentary, Section D1.3.~~This contribution due to system reduction together with engineering judgment and system testing appears to justify an R-value of 3.5 in a qualitative sense. However, it is intended that R = 3.5 be used for preliminary design. Once preliminary member sizes are determined~~, Furthermore, a capacity design procedure has been provided in Section E of AISI S110 such that the designer can explicitly calculate the seismic load effect with overstrength, E_m,. at the design story drift level. Alternatively, a conservative system overstrength factor, Ω_o, is also provided to be compatible with the conventional approach to compute E_m in ASCE 7. Finally the height limitation of 35 feet for all SDCs is based on practical use only and not from any limits on the CFS-SBMF system strength.~~provides a procedure that allows the designer to calculate the maximum seismic forces and story drift in the system. In other words, the most significant feature of AISI S110, Section E is that it is based on a calibrated, reliable physical model and R = 3.5 is only used for preliminary sizing.~~

~~For this particular system, no system overstrength factor has been established rather, it is intended that the seismic load effect with overstrength, E_m, be based upon the expected strength determined in accordance with AISI S110, Section D.1.2.3. The derivation of the deflection amplification factor, C_d, can be found in the AISI S110 Commentary, Section D1.3. Finally, the height limitation of 35 feet for all SDCs is based on practical use only and not from any limits on the CFS-SBMF system strength.~~

Effect of Bolt Tension Variation on the Seismic Response of CFS-SBMF

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1. Introduction

According to the AISI S110 Standard, high-strength bolts used in a Cold-Formed Steel−Special Bolted Moment Frame (CFS-SBMF) are to be in a snug-tight condition. Based on the cyclic test results of 9 full-scale beam-column subassemblies, it was shown that using a 10-kip bolt tension for the 1-in. diameter high-strength bolts would provide a satisfactory correlation of the test results in the slip range. Therefore, this bolt tension force was adopted in the AISI S110 Standard to compute the slip resistance of a bolt group.

Since bolts are to be tensioned to a snug-tight condition, some raised a concern on whether over-tightening or under-tightening will affect the maximum moment that can be developed in the bolted connection. Since the inelastic action in a CFS-SBMF is to be developed in the bolted moment connection during a seismic event, a high moment in the bolt connection may over-load the beams and columns and cause the frame to failure in a nonductile manner.

2. Objective

A sensitivity study was conducted to address the above concern, By varying the bolt tension force, a series of nonlinear time-history analyses of a sample CFS-SBMF by using a suite of 20 earthquake ground motions was conducted to evaluate statistically the effect of the variation of bolt tension on the seismic response including the story drift and the maximum moment in the bolt connection.

3. Structural Model

A two-bay interior CFS-SBMF (see Figure 1) was first designed based on the design loads shown in Table 1. The frame was assumed to be located in a high seismic region with $S_{DS} = 1.1$ and $S_{D1} = 0.64$.

Table 2 shows the member sizes and the moment connection details. ASTM A653 steel and ASTM A500 Grade B steel were specified for the beams and columns, respectively, and 1-in. diameter A325 high-strength bolts were used for the moment connections.

The analysis software OpenSees (Open System for Earthquake Engineering Simulation) was used for the analyses. Both the beams and columns were modeled by nonlinear fiber elements with five integration points along the member length. Each bolted moment connection was modeled by a zero-length rotational spring with the hysteresis rule similar to that shown in Figure 2.

Three frame models were considered in this study. The "Standard" model assumes that the bolts were properly tightened to the snug-tight condition such that the bolt tension is the same as that assumed (T = 10 kips) in the S110 Standard. The "Over-tightened" model assumes that all the bolts were over-tightened such that the bolt tension is 50% higher (i.e., T = 15 kips). It should be noted that the bolt grip length for CFS-SBMF application is very small. Therefore, it is conservative to assume that bolt over-tightening would result in a 50% increase in tension. The "Under-tightened" model assumes that the bolt tension is 70% of that assumed in the S model (i.e., T = 7 kips).

4. Static Pushover Analysis

A comparison of the static pushover analysis results can be made from Figure 3. The variation in the bolt tension mainly affects the response in the slip range, but not too much in the bearing range.

5. Nonlinear Time-History Analysis

The fundamental period of the frame was 0.78 seconds. It was assumed that the frame had 5% damping of critical. A suite of twenty large-magnitude, small-distance earthquake ground motions were used for the nonlinear time-history analysis. These records are listed in Table 3. Each ground motion was scaled such that the spectral acceleration at the fundamental period of the frame matches the 5% damped IBC Design Basis Earthquake Response Spectrum.

A comparison of the sample global responses for three models is shown in Figure 4. Figure 5 summarizes the maximum story drift produced by all 20 ground motions. It is observed that a larger bolt tension would reduce not only the story drift but also the scatter of the story drift. Figure 6 shows a summary of the base shear. (The maximum moment in a bolted connection is proportional to the base shear.) The average value and the standard deviation of both the story drift and base shear are listed in Table 4.

6. Observations and Conclusions

The bolt grip length for CFS-SBMF application is small, which implies that the increase in bolt tension due to over-tightening is not expected to be high. Even with an assumption that the bolt tension would be increased by 50% due to over-tightening, it is observed from Table 4 that both the story drift and base shear (or moment in the bolted connection) are not very sensitive to the variation of bolt tension. In this case, over-tightening reduces the story drift by 3%, although the maximum moment in the bolted connection is increased by 11%. Therefore, it is concluded from this sensitivity study that over-tightening is not a concern.

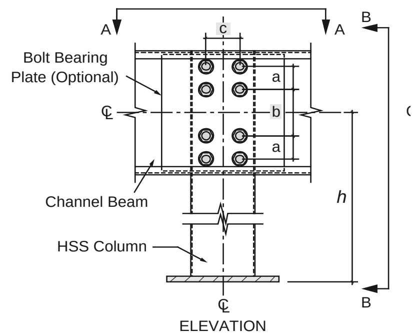
Table 1 Design Loads

Dead Load	Weight (psf)
Floor System	2
Steel Deck	2
Steel Joint	2
Steel Beams/Columns	3
Miscellaneous	3
Total	12

Live Load = 125 psf

Table 2 Member Sizes and Bolted Connection Configuration

Component	Dimensions (in)		
Beam	Double 12CS3½×105		
Column	HSS8×8×¼		
Bolted Connection	*a*	*b*	*c*
	3	6	4¼



A
A
B
c
Bolt Bearing
Plate (Optional)
a
b
a
Channel Beam
h
HSS Column
B
ELEVATION

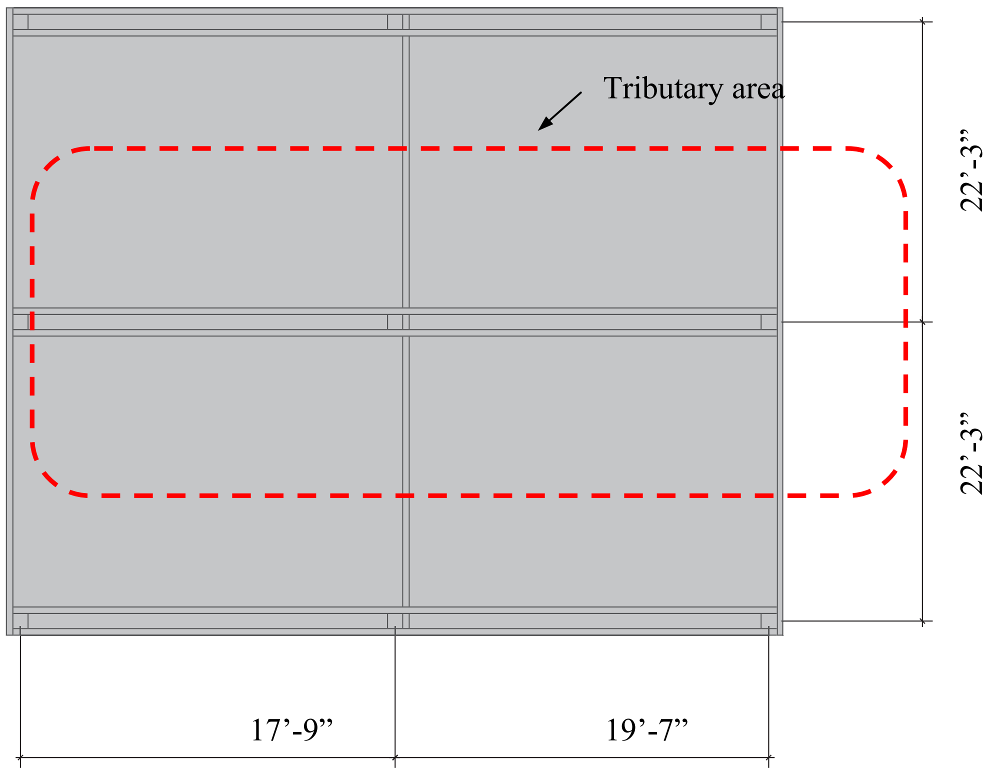
Table 3 Ground Motion Records

Name	Event	Year	Station	*R* (km)	*PGA* (g)	Duration (sec.)	Scaling Factor
P01	Loma Prieta	1989	Agnews State Hospital	28.2	0.172	40.0	2.92
P02	Loma Prieta	1989	Capitola	14.5	0.443	40.0	1.05
P03	Loma Prieta	1989	Gilroy Array #3	14.4	0.367	39.9	1.35
P04	Loma Prieta	1989	Gilroy Array #4	16.1	0.212	40.0	1.82
P05	Loma Prieta	1989	Gilroy Array #7	24.2	0.226	40.0	2.52
P06	Loma Prieta	1989	Hollister City Hall	28.2	0.247	39.1	0.91
P07	Loma Prieta	1989	Hollister Differential Array	25.8	0.279	39.6	1.06
P08	Loma Prieta	1989	Sunnyvale-Colton Ave.	28.8	0.207	39.3	1.83
P09	Northridge	1994	Canoga Park-Topanga Can.	15.8	0.420	25.0	1.18
P10	Northridge	1994	LA-N Faring Rd.	23.9	0.237	30.0	1.73
P11	Northridge	1994	LA-Fletcher Dr.	29.5	0.240	30.0	1.35
P12	Northridge	1994	Glendale-Las Palmas	25.4	0.206	30.0	6.63
P13	Northridge	1994	LA-Hollywood Store FF	25.5	0.231	40.0	1.55
P14	Northridge	1994	La Crescenta-New York	22.3	0.159	30.0	3.94
P15	Northridge	1994	Northridge-17645 Saticoy St	13.3	0.368	30.0	1.70
P16	San Fernando	1971	LA-Hollywood Store Lot	21.2	0.174	28.0	5.94
P17	Superstition Hills	1987	Brawley	18.2	0.156	22.1	4.39
P18	Superstition Hills	1987	El Centro Imp. Co. Cent	13.9	0.358	20.0	1.07
P19	Superstition Hills	1987	Plaster City	21.0	0.186	22.2	2.62
P20	Superstition Hills	1987	Westmoreland Fire Station	13.3	0.172	40.0	1.90

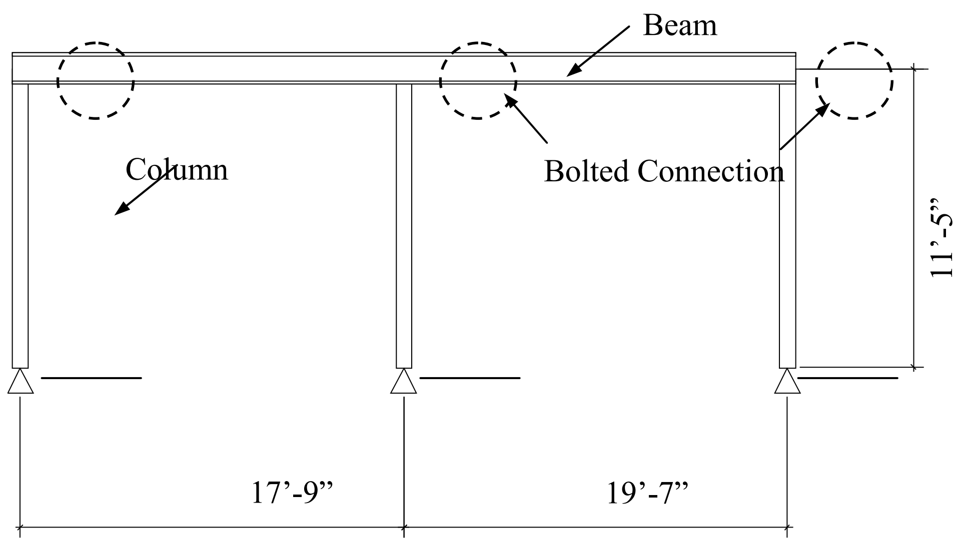
Table 4 Summary of Nonlinear Time History Analysis Results

Model	Base Shear (kips)	Story Drift (%)
Under-tightened (T = 7 kips)	9.7 [4.8]	3.0 [1.1]
Standard (T = 10 kips)	9.8 [3.5]	2.9 [0.93]
Over-tightened (T = 15 kips)	10.9 [2.2]	2.8 [0.72]

[]: standard deviation



(a) Plan



(b) Elevation

Figure 1 Analysis Model

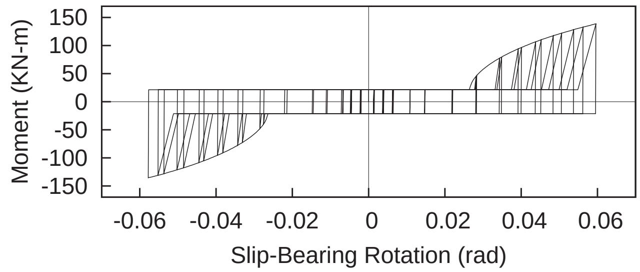


Figure 2 Hysteretic Model for the Bolted Moment Connection

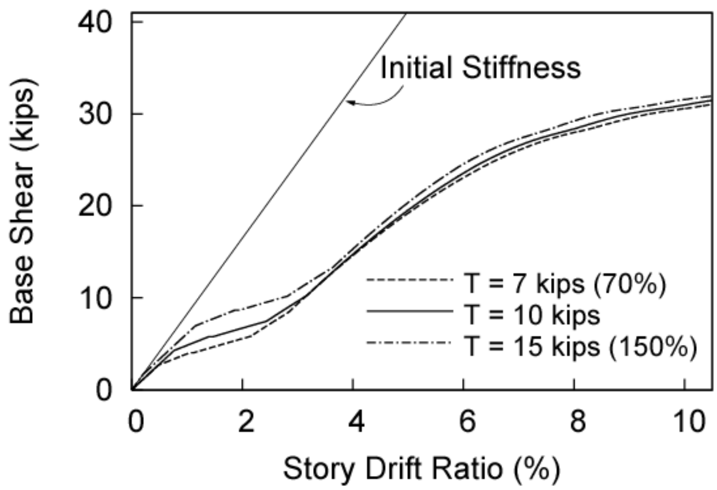
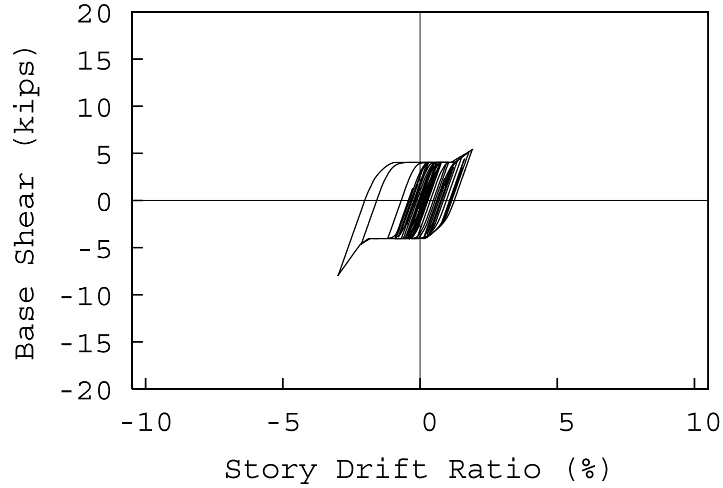
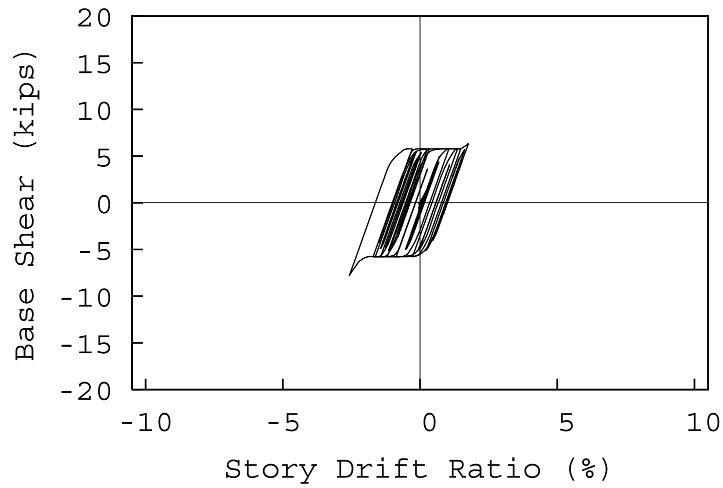


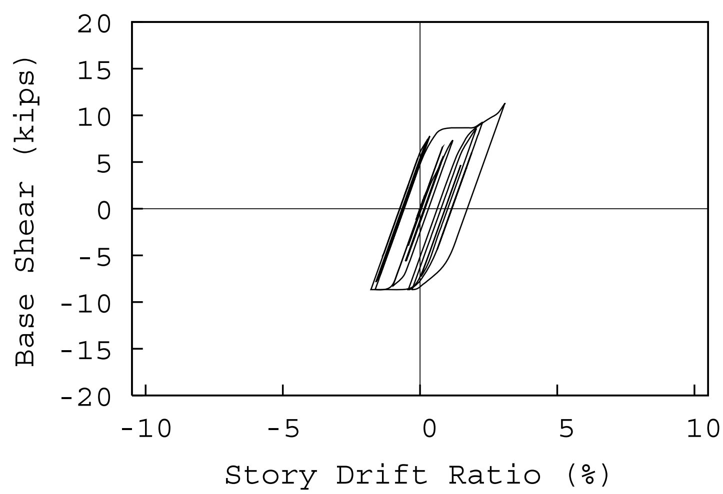
Figure 3 Pushover Analysis Results



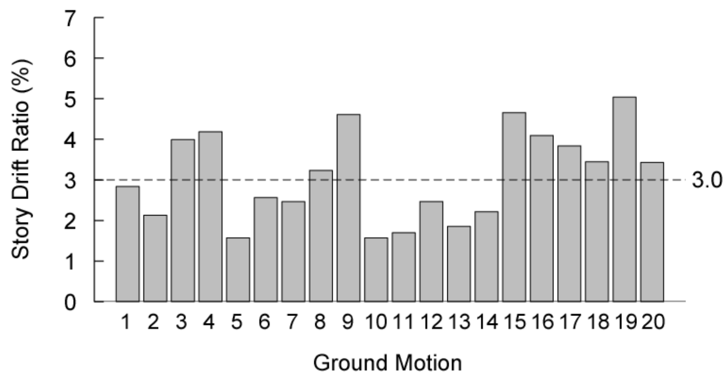
(a) Under-tightened Model (T = 7 kips)



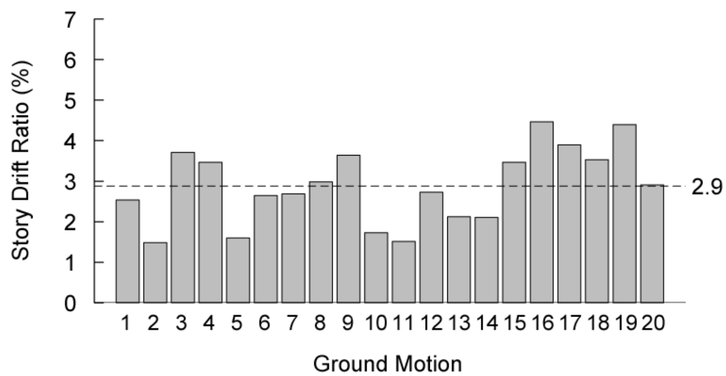
(b) Standard Model (T = 10 kips)



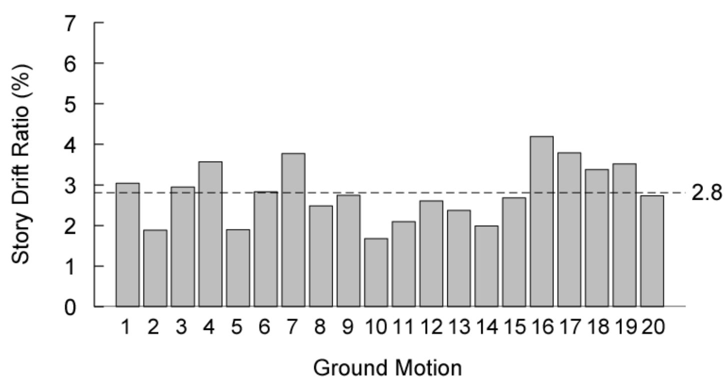
(c) Over-tightened Model (T = 15 kips)
Figure 4 Sample Global Responses (P01 Ground Motion Record)



(a) Under-tightened Model (T = 7 kips)

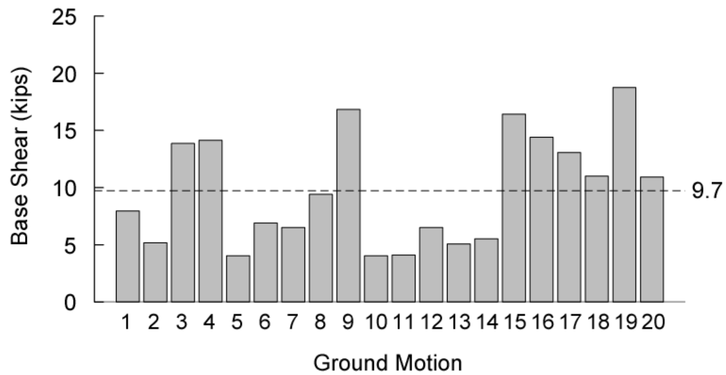


(b) Standard Model (T = 10 kips)

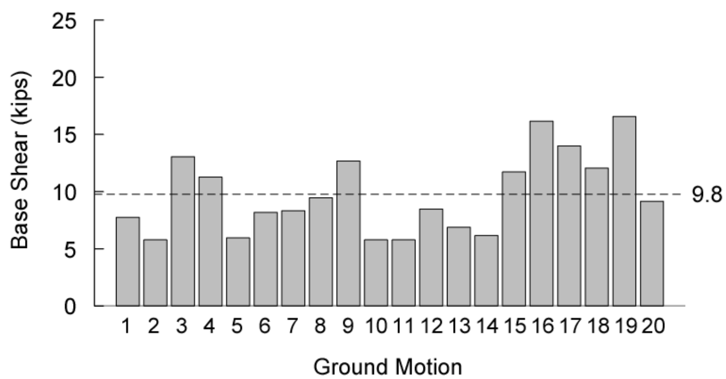


(c) Over-tightened Model (T = 15 kips)

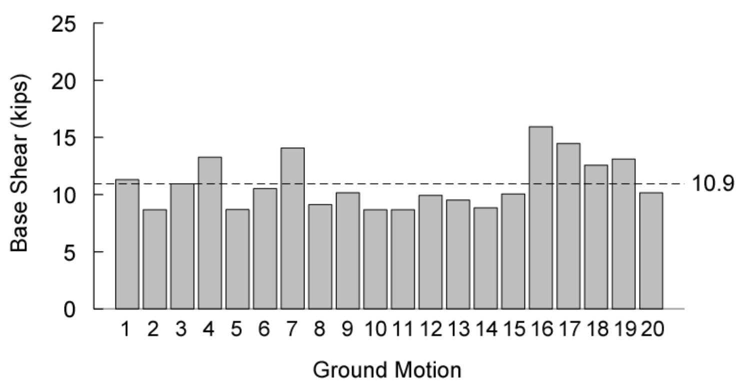
Figure 5 Maximum Story Drift Ratio



(a) Under-tightened Model (T = 7 kips)



(b) Standard Model (T = 10 kips)



(c) Over-tightened Model (T = 15 kips)

Figure 6 Maximum Base Shear