Cold-Formed Steel Special Bolted Moment Frames: Capacity Design Requirements

by

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ABSTRACT

Design provisions of the Cold-Formed Steel─Special Bolted Moment Frame (CFS─SBMF) system in the proposed AISI Seismic Standard (AISI S110) are developed such that energy dissipation in the form of bolt slippage and bearing in the bolted beam-to-column moment connections would occur during a major seismic event. Beams and columns are then designed following the capacity design principles to remain elastic. Based on the instantaneous center of rotation concept, this paper presents background information for the design provisions in the AISI standard for calculating the expected maximum seismic force in the beams and columns at the design story drift. This requires that the resistance from both the bolt slippage and bearing actions in the moment connection be computed. Design tables are provided to facilitate the design. The recommended seismic design procedure is also provided.

INTRODUCTION

The American Iron and Steel Institute (AISI) is in the process of developing a seismic design Standard for cold-formed steel, *Standard for Seismic Design of Cold-Formed Steel Structural Systems─Special Bolted Moment Frames* - AISI S110 [AISI, 2007]. The first seismic force-resisting system introduced in the AISI seismic standard is termed Cold-Formed Steel─Special Bolted Moment Frames (CFS─SBMF). It is common that this type of one-story moment frames is composed of cold-formed Hollow Structural Section (HSS) columns and double-channel beams. Beams are connected to the column by using snug-tight high-strength bolts; see Figure 1 for a typical moment connection detail.

Cyclic testing of full-scale beam-column subassemblies [Uang et al., 2008] showed that the bolted moment connection can provide a high ductility capacity

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through bolt slippage and bearing (Figure 2). The test results also showed that column and beam local buckling should be avoided because it would result in a strength degradation.

This paper provides the background information for the development of capacity design provisions contained in the proposed AISI Seismic Standard for CFS–SBMF. The objective of these design provisions is to ensure that inelastic action occurs in the bolted moment connections only during a design earthquake event, and that both beams and columns should remain elastic.

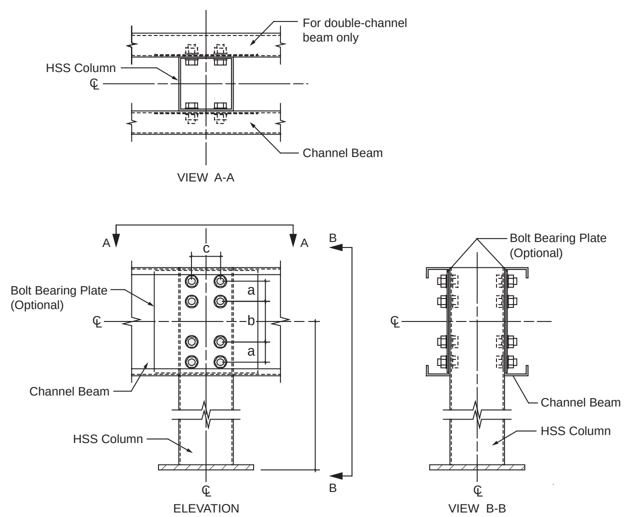
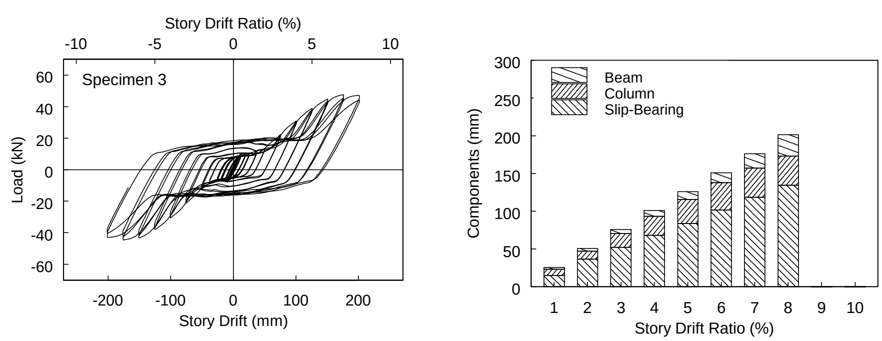


FIGURE 1 – BOLTED MOMENT CONNECTION



(a) – Hysteresis Response (b) – Story Drift Components

FIGURE 2 – BOLTED MOMENT CONNECTION

EXPECTED SEISMIC RESPONSE

In accordance with the AISI Seismic Standard (AISI S110), a designer would first use a value of R (Response Modification Coefficient) of 3.5 for preliminary design. Figure 3 shows that the elastic seismic force corresponding to the Design Basis Earthquake (DBE, point ‘e’) is reduced by the R factor to point ‘d’ for sizing beams, columns, and bolted moment connections. Unlike other seismic force-resisting systems where point ‘d’ represents the first significant yielding event (e.g., formation of the plastic hinge in a moment frame), CFS─SBMF actually would ‘yield’ at a lower seismic force level (point ‘a’) due to slippage of the bolts in moment connections. A horizontal plateau (point ‘a’ to ‘b’) would result due to the oversize of the bolts. As the story drift is increased, the lateral resistance starts to increase from point ‘b’. Test results showed that such hardening in strength is very significant (see Figure 2), and it is not appropriate to assume an elastic-perfectly plastic (EPP) global response for either analysis or design.

Considering the effect of such significant hardening, a Deflection Amplification Factor, C_d, was also developed for CFS─SBMF in the AISI Seismic Standard (AISI S110). With the C_d value, the designer then can amplify the story drift at point ‘d’ to estimate the maximum inelastic story drift (Δ at point ‘c’) that is expected to occur in a Design Earthquake event. To ensure that beams and columns will remain elastic, the challenge then is to evaluate the maximum seismic force corresponding to point ‘c’. This seismic force level represents the required seismic strength for the beams and columns.

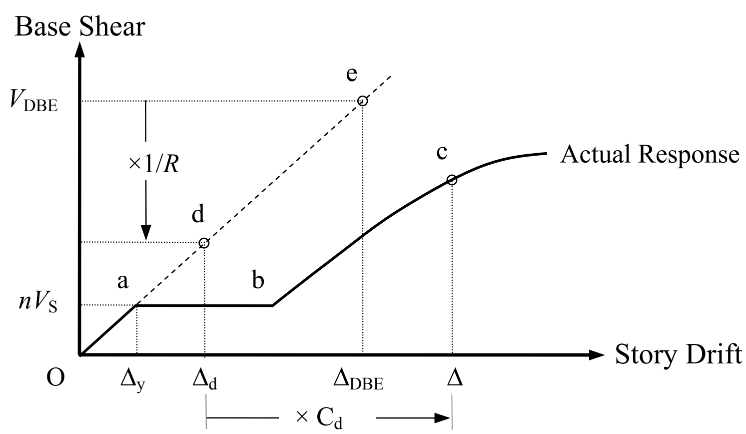


FIGURE 3 – GENERAL STRUCTURAL RESPONSE OF CFS─SBMF

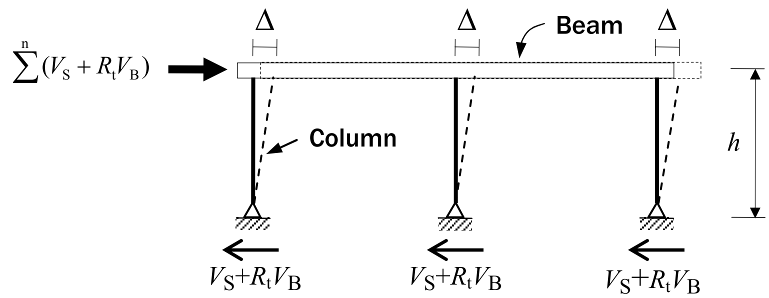


FIGURE 4 – YIELD MECHANISM AND COLUMN SHEAR DISTRIBUTION

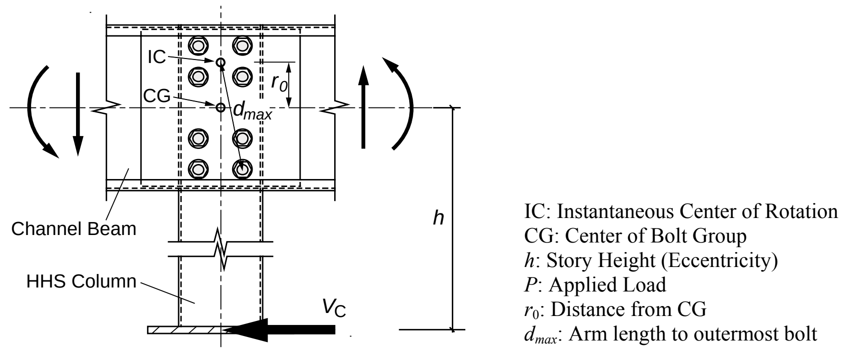


FIGURE 5 – FREEBODY OF ONE COLUMN

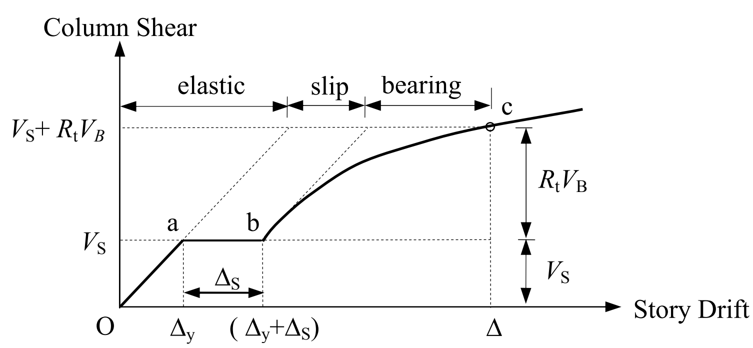


FIGURE 6 – LATERAL RESISTANCE OF ONE COLUMN

It is common that same-size beams and same-size columns are connected by high-strength bolts with the same configuration. Referring to a sample frame shown in Figure 4, interior column(s) will resist more shear than exterior columns in the elastic range. Once the frame responds in the inelastic range to point ‘c’ in Figure 3, however, it is reasonable to assume that column shears will equalize as shown in Figure 4. Capacity design of the beams and columns can be performed if the maximum shear force developed in the columns can be evaluated. Specifically, the required moment for both beam and column at the connection location is

$$M_e = h\left(V_{\mathrm{S}} + R_{\mathrm{t}} V_{\mathrm{B}}\right) \qquad (1)$$

where h = story height, and R_t = the ratio of expected tensile strength to specified tensile strength. V_S and V_B represent resistance due to bolt slippage and bearing.

SLIP COMPONENT OF COLUMN SHEAR AND SLIP DRIFT

The freebody of one column is shown in Figure 5. With the shear at the base of the column, the bolt group is in eccentric shear. To show the components of lateral resistance of the yield mechanism in Figure 4, Figure 3 is replotted for one column only and shown as Figure 6. To calculate the maximum force developed at point ‘c’, it is necessary to first compute the column shear (V_S) that causes the bolts to slip and the amount of slip, expressed in the form of story drift (Δ_S).

Since the bolt group is in eccentric shear, the instantaneous center of rotation concept [Crawford and Fisher, 1971; Salmon and Johnson, 1996] can be used to compute V_S. Given the bolt oversize, the slip drift (Δ_S) can also be computed in the analysis. These two quantities for some commonly used bolt configuration are provided in Table 1. To facilitate design, a regression analysis of the values contained in Table 1 was also conducted, which resulted in the following two expressions:

$$V_{\mathrm{S}} = C_{\mathrm{S}} kNT / h \qquad (2)$$

$$\Delta_{\mathrm{S}} = C_{\mathrm{DS}} h_{\mathrm{OS}} h \qquad (3)$$

where C_S, C_{DS} = regressed values from Table 2, k = slip coefficient, N = number of channels in a beam, T = snug-tight bolt tension, h_{OS} = hole oversize (= 1/16 in. for standard holes), and h = story height. A value of k equal to 0.33 and value of T equal to 10 kips were used [Uang et al., 2008].

BEARING COMPONENT OF COLUMN SHEAR AND BEARING DRIFT

Referring to point ‘c’ in Figure 6, the design story drift (Δ) is composed of three components: (i) the recoverable elastic component which is related to the

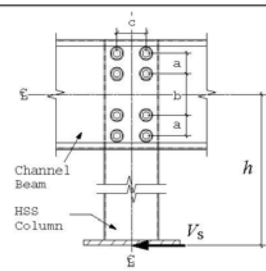
TABLE 1 – VALUES OF G_S, AND G_{DS} FOR ECCENTRICALLY LOADED BOLT GROUP

$$V_S = N \times G_S \times R_s$$

$$\Delta_S = G_{DS} \times h_{os}$$

N = 1 for single-channel beams
= 2 for double-channel beams

where
V_S = column shear causing slip
R_S = slip strength per bolt
(= $k \times T$)
k = slip coefficient
T = snug-tight bolt tension
h = story height, ft
a, b, *and* c = bolt spacing, in.
Δ_S = slip drift due to slip
G_S, G_{DS} = coefficient tabulated below
h_{os} = hole oversize

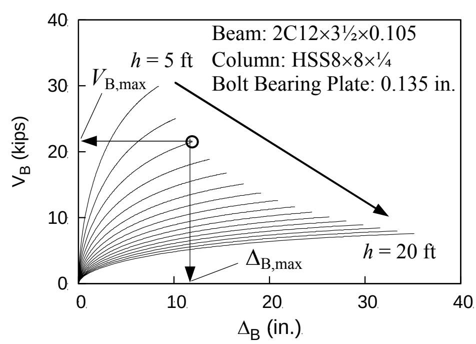


c, in.	h, ft	Bolt spacing a and b, in.					
		a = 2-1/2, b = 3		a = 3, b = 6		a = 3, b = 10	
		G_S	G_{DS}	G_S	G_{DS}	G_S	G_{DS}
4-1/4	8	0.296	40.5	0.416	26.6	0.562	17.6
	9	0.264	45.8	0.370	30.3	0.501	20.1
	10	0.237	51.0	0.333	34.0	0.452	22.7
	11	0.216	56.3	0.303	37.7	0.411	25.3
	13	0.183	66.9	0.257	45.1	0.349	30.6
	15	0.158	77.5	0.223	52.6	0.303	35.9
	17	0.139	88.1	0.197	60.1	0.268	41.4
	19	0.125	98.7	0.176	67.6	0.240	46.9
	21	0.113	109	0.159	75.1	0.217	52.5
	23	0.103	120	0.145	82.6	0.198	58.1
	25	0.0946	130	0.134	90.2	0.182	63.7
	27	0.0879	141	0.124	97.7	0.169	69.3
	29	0.0818	152	0.115	105	0.157	75.0
	31	0.0763	162	0.108	113	0.147	80.7
	33	0.0714	173	0.101	120	0.138	86.4
	35	0.0678	183	0.0955	128	0.130	92.1
6-1/4	8	0.355	36.2	0.460	25.8	0.597	18.2
	9	0.315	40.9	0.410	29.3	0.531	20.9
	10	0.284	45.6	0.369	32.9	0.479	23.5
	11	0.259	50.4	0.335	36.4	0.436	26.2
	13	0.218	59.8	0.284	43.5	0.370	31.6
	15	0.189	69.3	0.246	50.5	0.321	37.0
	17	0.167	78.7	0.217	57.6	0.283	42.5
	19	0.150	88.2	0.194	64.7	0.253	48.0
	21	0.135	97.6	0.176	71.8	0.229	53.5
	23	0.124	107	0.161	78.9	0.210	59.0
	25	0.114	117	0.148	85.9	0.193	64.6
	27	0.105	126	0.137	93.0	0.179	70.1
	29	0.0977	135	0.127	100	0.166	75.7
	31	0.0915	145	0.119	107	0.156	81.2
	33	0.0859	154	0.112	114	0.146	86.8
	35	0.0810	164	0.105	121	0.138	92.4

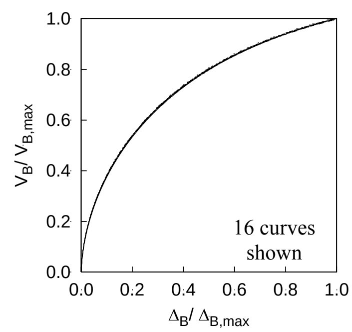
TABLE 2 – VALUES OF COEFFICIENTS C_S, C_{DS}, C_B, AND $C_{B,0}$

Bolt spacing*, in.			C_S (ft)	C_{DS} (1/ft)	C_B (ft)	$C_{B,0}$ (in./ft)
a	b	c				
2½	3	4¼	2.37	5.22	4.20	0.887
3	6		3.34	3.61	5.88	0.625
3	10		4.53	2.55	7.80	0.475
2½	3	6¼	2.84	4.66	5.10	0.792
3	6		3.69	3.44	6.56	0.587
3	10		4.80	2.58	8.50	0.455

* See Figure 1



(a) Typical Bearing Response Curves



(b) Normalized Response Curves

FIGURE 7 – SAMPLE RESULT OF BEAING RESPONSE

lateral stiffness, K, of the frame, (ii) the slip component, Δ_S, which can be computed from Eq. (3), and (iii) the bearing component computed from following equation:

$$\Delta_B = \Delta - \Delta_S - \frac{nM_e}{hK} \tag{4}$$

where n = number of column in a frame line (i.e, number of bays plus 1), M_e = expected moment at a bolt group computed from Eq. (1).

Applying the instantaneous center of rotation concept to an eccentrically loaded bolt group [Uang et al., 2008], the relationship between the bearing component of the story drift, Δ_B, and the bearing component of the column shear, V_B, can be established. Figure 7(a) shows a sample result. For a given frame height, the last point of each curve represents the ultimate limit state when the bearing deformation of the outermost bolt reaches 0.34 in. (8.6 mm) [AISC, 2005]. Ultimate bearing shear of the column, $V_{B,max}$, and corresponding bearing drift deformation, $\Delta_{B,max}$, for some commonly used bolt configuration and story heights

TABLE 3 – VALUES OF G_S, AND G_{DS} FOR ECCENTRICALLY LOADED BOLT GROUP

$$V_{B,max} = N \times G_B \times R_0$$

$$\Delta_{B,max} = C_{DB} \times \Delta_{B,0}$$

N = 1 for single-channel beams
= 2 for double-channel beams

where

$V_{B,max}$ = column shear causing bolt maximum bearing

R_0 = minimum values of dtF_u of connected beam and column webs

F_u = tensile strength

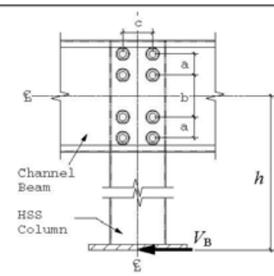
t = bearing thickness

d = bolt diameter

G_B = coefficient tabulated below

$\Delta_{B,0}$ = maximum bearing drift deformation

C_{DB} = bearing deformation adjustment factor [Eq. (6)]



c, in.	h, ft	Bolt spacing a and b, in.					
		a = 2-1/2, b = 3		a = 3, b = 6		a = 3, b = 10	
		G_B	$\Delta_{B,0}$, in.	G_B	$\Delta_{B,0}$, in.	G_B	$\Delta_{B,0}$, in.
4-1/4	8	0.524	6.92	0.728	4.77	0.983	3.50
	9	0.466	7.81	0.649	5.40	0.878	4.00
	10	0.420	8.71	0.586	6.04	0.794	4.49
	11	0.381	9.61	0.533	6.68	0.724	4.98
	13	0.323	11.4	0.453	7.95	0.616	5.97
	15	0.281	13.2	0.393	9.23	0.536	6.96
	17	0.247	15.0	0.347	10.5	0.474	7.95
	19	0.222	16.8	0.311	11.8	0.425	8.94
	21	0.200	18.6	0.281	13.1	0.385	9.92
	23	0.183	20.4	0.257	14.3	0.352	10.9
	25	0.169	22.2	0.237	15.6	0.325	11.9
	27	0.156	24.0	0.220	16.9	0.301	12.9
	29	0.145	25.8	0.204	18.2	0.281	13.9
	31	0.136	27.6	0.191	19.5	0.262	14.9
	33	0.127	29.4	0.180	20.7	0.247	15.8
	35	0.120	31.2	0.169	22.0	0.233	16.8
6-1/4	8	0.637	6.17	0.814	4.48	1.05	3.36
	9	0.566	6.97	0.725	5.08	0.935	3.82
	10	0.510	7.77	0.654	5.68	0.845	4.29
	11	0.464	8.57	0.595	6.28	0.771	4.76
	13	0.393	10.2	0.504	7.48	0.655	5.70
	15	0.341	11.8	0.438	8.68	0.570	6.65
	17	0.302	13.4	0.387	9.88	0.504	7.59
	19	0.269	15.0	0.347	11.1	0.452	8.54
	21	0.244	16.6	0.314	12.3	0.410	9.48
	23	0.222	18.2	0.287	13.5	0.374	10.4
	25	0.205	19.8	0.264	14.7	0.345	11.4
	27	0.189	21.4	0.244	15.9	0.319	12.3
	29	0.176	23.0	0.228	17.1	0.298	13.3
	31	0.165	24.6	0.213	18.3	0.279	14.2
	33	0.154	26.2	0.201	19.5	0.262	15.2
	35	0.146	27.8	0.189	20.7	0.247	16.1

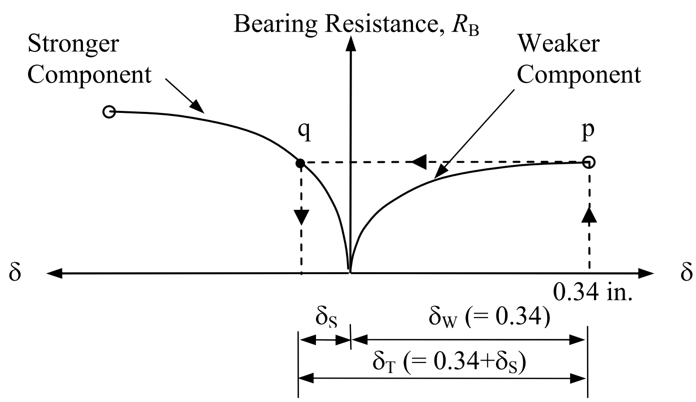


FIGURE 8 – BOLT BEARING DEFROMATION IN STRONGER AND WEAKER COMPONENTS

are computed and are tabulated in Table 3. The variable R_0 refers to the governing value (or minimum value) of dtF_u of the connected components (beam and column webs).

Each bolt in the moment connection bears against not only the column web but also the beam web. The bearing force exerted by the bolt to both components is identical. But the bearing deformation can be different between these two components, depending on the relative bearing strength, tF_u, where t = thickness of the component, F_u = tensile strength. The $\Delta_{B,0}$ values in Table 3 correspond to the maximum drift when the bearing deformation is contributed by the weaker component (either beam or column) only. That is, it is assumed that the stronger component is rigid. The Bearing Deformation Adjustment Factor, C_{DB}, in Table 3 accounts for the additional contribution to bearing deformation from the stronger component. Refer to point 'p' in Figure 8, where the ultimate bearing deformation [= 0.34 in. (8.6 mm)] of the weaker component is reached. Since the bearing force of the bolt on both weaker and stronger components is identical, it can be shown that the corresponding bearing deformation (unit in inch) of the stronger component (i.e., point 'q') is

$$\delta_S = -\frac{1}{5}\ln\left[1 - 0.817\left(\frac{(tF_u)_W}{(tF_u)_S}\right)^{1.82}\right] \tag{5}$$

The C_{DB} factor represents the ratio between the total bearing deformation and 0.34 inch.

TABLE 4 – BEARING DEFORMATION ADJUSTMENT FACTOR C_{DB}

Relative Bearing Strength	0.0	0.4	0.5	0.6	0.7	0.8	0.9	1.0
C_{DB}	1.00	1.10	1.16	1.23	1.33	1.46	1.66	2.00

where
relative bearing strength (RBS) = $(tF_u)_{(weaker)}/(tF_u)_{(stronger)}$
t = Thickness of beam or column component
F_u = Tensile strength of beam or column

$$C_{DB} = \frac{0.34 + \delta_S}{0.34} = 1.0 - 0.588 \ln\left[1 - 0.817\left(\frac{(tF_u)_W}{(tF_u)_S}\right)^{1.82}\right] \tag{6}$$

A regression analysis of Table 3 was conducted to derive the following design formulae, and Table 4 is provided for the bearing deformation adjustment factor, C_{DB}, to facilitate design.

$$V_{B,\max} = C_B N R_0 / h \tag{7}$$

$$\Delta_{B,\max} = C_{B,0} C_{DB} h \tag{8}$$

where C_B, $C_{B,0}$ = regressed values from Table 2.

For a given beam size, column size, and a bolt configuration, Figure 7(a) shows that the response curve is dependent on the story height. Eqs. (7) and (8) define the ultimate bearing strength point of each curve in the bearing response curve [see Figure 7(a)]. Normalizing each curve by its ultimate bearing strength point, however, Figure 7(b) shows that the normalized curves can be approximated very well by the following expression:

$$\left(\frac{V_B}{V_{B,\max}}\right)^2 + \left(1 - \frac{\Delta_B}{\Delta_{B,\max}}\right)^{1.43} = 1 \tag{9}$$

Given a value of Δ_B from Eq. (4), Eq. (9) can be used to compute the bearing component of the column shear, V_B, and, hence, M_e in Eq. (1). But since Eq. (4) also contains M_e, iteration is required to compute the expected moment, M_e. A flowchart is provided in Figure 9. The following value is suggested as the initial value for Δ_B:

$$\Delta_B = \frac{\left[\Delta - (\Delta_S + \Delta_y)\right]K}{nV_{B,\max}/\Delta_{B,\max} + K} \tag{10}$$

where Δ_y is the story drift at point 'a' in Figure 6.

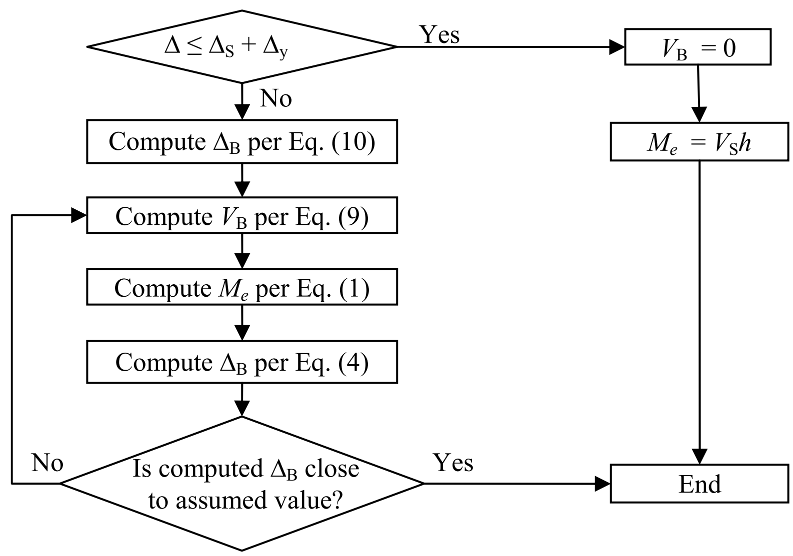


FIGURE 9 – FLOWCHART FOR COMPUTING EXPECTED MOMNET

DESIGN PROCEDURE FOR CFS–SBMF

The recommended seismic design procedure follows.

Step 1 – Preliminary design

Perform a preliminary design of the beams, columns, and bolted connections by considering all basic load combinations in the applicable building code. Use a value of R equal to 3.5. In determining the earthquake load, use a rational method to determine the structural period.

Step 2 – Compute both the base shear (nV_S) that causes the bolt groups to slip and the slip range (Δ_S) in terms of story drift.

For a given configuration of the bolt group, Eqs. (2) and (3) can be used to compute both V_S and Δ_S. n represents the number of columns in a frame line.

Step 3 – Compute the design story drift, Δ

Follow the applicable building code to compute the design story drift, where the Deflection Amplification Factor is given in the AISI Seismic Standard (AISI S110).

Step 4 – Perform capacity design of beams and columns

Beams and columns should be designed based on special seismic load combinations of the applicable building code; the seismic load effect with

overstrength, E_m, is to be replaced by the required strength in Eq. (1). The flowchart in Figure 9 can be used for this purpose.
Step 5 – Check P-Δ effects following the applicable building code.

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