Review of current and proposed U.S. seismic codes for steel structures

Review of Current and Proposed U.S. Seismic Codes for Steel Structures

Revue des prescriptions sismiques pour les structures en acier aux Etats-Unis

Überblick über die geltenden und vorgeschlagenen amerikanischen Richtlinien für die seismische Bemessung von Stahlkonstruktionen

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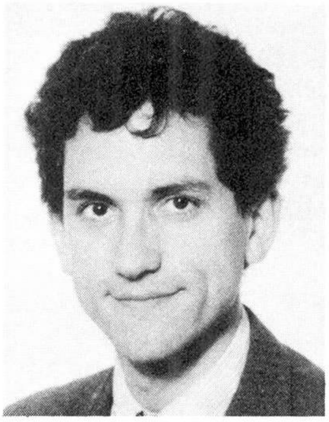

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SUMMARY

This paper is a comparative outline of several current and proposed U.S. seismic codes for steel structures. The code requirements for Ductile and Ordinary Moment Resisting Frames, Concentrically Braced Frames, K-Braced Frames and Ductile Eccentrically Braced Frames are discussed.

RÉSUMÉ

Cet article donne un résumé des prescriptions sismiques de construction en acier, courantes et en voie de developement en ce moment aux Etats-Unis. Tout les systèmes de structures en acier permis par ces prescriptions sont discutés.

ZUSAMMENFASSUNG

Dieser Vortrag vergleicht die in den Vereinigten Staaten geltenden sowie die vorgeschlagenen Richtlinien für die seismische Bemessung von Stahlkonstruktionen. Die reglementierten Anforderungen für verschiebliche und biegesteife Rahmen, konzentrisch versteifte Rahmen, K-versteifte Rahmen und verschiebliche, exzentrisch versteifte Rahmen werden erläutert.

1. INTRODUCTION

There are currently in effect almost a dozen design codes for earthquake resistant construction in the United States. These include:

a. The regional model building codes: The BOCA/Basic Building Code (BOCA) issued in Illinois; The National Building Code (NBC) issued in New York; The Standard Building Code (SBC) issued in Alabama; and The Uniform Building Code (UBC) issued in California. In addition there is Standard A58.1, American National Standard Building Code Requirements for Minimum Design Walls in Buildings and Other Structures of the American National Standards Institute (ANSI A58.1). Of these, all but ANSI A58.1 include both loading and material detailing requirements. The latest BOCA, NBC, SBC and the 1972 ANSI A58.1 derive from the "Recommended Lateral Force Requirements and Commentary" of the Seismology Committee of the Structural Engineers Association of California (SEAOC Recommendations) 2nd Edition, 1968. The current 1982 UBC is based on the 1980 edition of the SEAOC Recommendations, and the 1982 ANSI A58.1 is part 1975 SEAOC Recommendations, part ATC 3-06 (see below).
b. The SEAOC Recommendations are currently being revised. The remarks that follow are based on the 15 November 1984 Draft (SEAOC 11:84 Draft) of this document. The draft is still very much in discussion.
c. Codes developped for agencies and services of the Federal U.S. Government include: the General Service Administration which contracts most federal office buildings: the Veterans Administration which mostly builds hospitals the Departments of the Army, the Navy and the Air Force which publish the Technical Manual - Seismic Design for Buildings (1982 Tri-Services Manual).
d. Finally the Applied Technology Council, a non-profit research subsidiary established in 1971 by SEAOC, published the "Tentative Provisions for the Development of Seismic Regulations for Buildings" (ATC 3-06) in 1978. This project was funded by the U.S. National Science Foundation and National Bureau of Standards and undertaken by a group of 85 of the top earthquake engineering professionals and academicians.

The following is a comparative review of the design requirements for steel structures contained in the 1978 ATC 3-06, the 1982 UBC, Tri-Services Manual and ANSI A58.1 and the SEAOC 11:84 Draft only. These five codes best represent the range of consensus among current U.S. seismic codes. The structural systems discussed are Ductile (or Special) and Ordinary Moment Resistant Frames, Concentrically and K-Braced Frames and Ductile Eccentrically Braced Frames. The emphasis will be on two areas where a consensus is still not apparent and where additional "mission oriented" research would be helpful: Ordinary Moment Frames, or in general systems for zones of moderate seismicity, and Concentrically and especially K-Braced Frames.

2. DUCTILE MOMENT RESISTING FRAMES

Among earthquake resistant steel structural systems, Ductile Moment Resisting Frames (DMF) have received the greatest attention in recent research (Popov 1983). The detailing requirements described below have evolved mostly from the results of this research, rather than the evidence of earthquake damage. Indeed few multistory steel DMF's built in accordance to modern U.S. seismic codes have yet to be tested in severe earthquakes. The following requirements are all intended to help insure stable cyclic energy dissipating capacity for large ductility demands.

2.1 Column Strength Requirements

The SEAOC Recommendations (1975 on) suggests in its commentary that DMF's be designed for a "strong column-weak beam" mechanism. It is further suggested, based on the test results of Popov (1975) that the column axial load ration P/P_y be kept below 0.5. ATC 3-06 modified this to require that P be limited to less than 0.6 P_y. Neither the 1982 UBC nor the Tri-Services Manual address this issue. The SEAOC 11:84 Draft includes general provisions for column strength applicable to all steel systems: (a) the axial compressive stress due to gravity loads plus 3 times the modified elastic reponse spectrum given in the code (i.e. $3R_y/8$) must not exceed 1.7 times the allowable stress (F_a) unless that axial load is somehow limited by the mechanism; and (b) the K value used to calculate the effective length of the column can be taken as 1.0 if the P-delta due to the lateral loads is considered explicitly. ATC 3-06 included a similar provision for the K value. The strong column-weak beam principle is quantified in the SEAOC 11:84 Draft for the first time: the sum of column plastic moments (calculated by $Z_c(F_{yc}-F_a)$) is required to exceed the sum of girder plastic moments unless the column axial stress is less than 0.4 F_y.

2.2 Column Splice Requirement

ATC 3-06 limited the use of partial penetration welds in column splices by requiring that they be able to withstand stresses due to either: 1.25 times the full joint plastic moment at both ends; the full joint plastic moment at one end combined with half the plastic moment at the other end; or the tension imposed by half the gravity load minus the seismic axial load. The SEAOC 11:84 Draft currently includes a requirement that partial penetration welds be sized for 150 percent of the tension due to $3R_w/8$ (i.e. 3 times the design spectrum values) times the seismic axial load in conjunction with the gravity load, unless that seismic axial load is limited by the mechanism. The minimum would be to develop 50% of the column flange area.

2.3 Joint Panel Zone Shear Design

The 1982 UBC and Tri Services Manual reference the AISC Code which in its plastic design section limits the panel zone shear stress to 0.55 F_y. Though neither code specifies it, the practice (suggested in the Commentary to the SEAOC Recommendations) has been to check the panel zone for shears due to the development of girder plastic moments. This has often led to the necessity of adding costly column web doubler plates. ATC 3-06 both codified the requirement and provided possible relief by allowing an exception for panel zones capable of resisting stresses resulting from twice the design level story drifts. The SEAOC 11:84 Draft is not yet resolved on this issue. One proposal is to limit the design level (working) shear stresses to 0.4 F_y. This would lead to reductions in doubler plate requirements, but could also result in joints whose strength is governed by that of the panel zone, which can be as little as half the girder strength. Tests (Bertero et.al. 1972) have indicated that such mechanisms can lead to local kinks at the beam/column flange intersection which may cause cracking and fracture at the welds. Krawinkler (1978, 1985) has proposed instead an equation which accounts for the strengthening effect of the column flanges (up to 20-30% for the most common member sizes) in evaluating the panel zone capacity. This could be used in conjunction with either the full plastic girder moments or twice the code level seismic moments, as suggested in ATC 3-06.

2.4 Beam Column Connections

Both the 1982 UBC and Tri-Services Manual require the connection to develop the girder flexural strength or provide "adequate rotation capacity". Where the specified ultimate strength of the steel is less than 1.5 times the yield strength bolted flange connections are prohibited. ATC 3-06 includes only the former requirement. The SEAOC 11:84 Draft is also not yet resolved on this matter. There is concern that where the plastic modulus of the girder flanges alone (Z_{fl}) is less than 70% of the full plastic modulus, the typical butt welded flange and bolted web connection is insufficient (at $Z_{fl} + 0.7Z_b$ the flange stress would be $1.43F_y$). On the other hand, research work (Popov 1983) indicates that such connections perform quite well in cyclic load tests.

3. ORDINARY MOMENT FRAMES

Ordinary Moment Frames (OMF), a term coined by ATC 3-06, are non-ductile or semi-ductile moment frames intended for use in regions of moderate to low seismicity. UBC 1982 had allowed moment frames designed in accordance with the AISC Code to be used in Zones 1 and 2 (of a possible 4) with the same structural system factor (K) as Ductile Moment Frames (DMF). This is by far the least conservative of the requirements under review (see Table 1). ATC 3-06 prescribes force levels 1.78 times the force levels for DMF's for OMF's and allows their use in regions of moderate seismicity (Seismic Performance Categories A and B) with no height limit and below 160 ft. or 100 ft. for normal and essential facility in zones of high seismicity (Categories C and D). The detailing need only satisfy the normal requirements of the AISC Code, with no special provisions for ductility.

The Tri-Services Manual provides for 3 types of moment frames: Type A which corresponds to the DMF; Type B frames which are allowed up to a height limit of 160 ft. in moderate seismic zones and 80 ft. in high seismic zones and which must be sized for twice the seismic moment plus the gravity moment ($2M_e + M_d + M_l$): and Type C frames which are allowed up to 80 ft. in zones of low seismicity (Zone 1) and can be designed by the AISC Code. Girders in Type B frames would therefore have roughly 2.6 times the strength of those in OMF's designed by UBC 1982 (assuming M_e = 75% of design moment).

The 1982 edition of ANSI A58.1 proposes that OMF's be designed for 1.5 times the forces used for DMF's (i.e. K=1.0) and that the beam/column connection develop the joint capacity. The SEAOC 11:84 Draft specifies OMF force levels 2-2.4 times those for DMF's. Furthermore they are prohibited in zones of high seismicity (Zone 3 and 4) unless they can sustain loads 3 times the design response spectrum values, or 4.5 times the force levels for DMF's ($3R_w/8$). Otherwise the Draft as of yet provides no detailing requirements.

Clearly there is wide disagreement among codes for OMF's. Prescribed force levels in moderate seismic zones are either 1.0, 1.5, 1.78, 2.4 or 2.6 times those for DMF's. In zones of high seismicity, the ratio may reach as high as 4.5. The problem is that there is at present little research or earthquake reconnaissance data to permit a determination of the cyclic inelastic response of the typical bolted flange moment connections used in areas outside zones of high seismicity. Popov et al. (1969, 1970 cited in Popov 1983) in a series of tests on W8 x 20 beams found that though the butt welded flange/bolted web connection showed greater cyclic energy dissipating capacity than any other type of connection, the bolted or welded flange plate connections did withstand substantial inelastic rotations prior to crack formation at the end weld or outermost bolt line. Interestingly though, connections to the weak axis of columns failed by cracking in a manner similar to the flange plate connections. Such connections are common in DMF's. In any case the hysteresis curves for bolted flange plate connections are "pinched" or S-shaped due to bolt slippage, not unlike those for reinforced concrete DMF's (which have structural system factors no more than 1.2 (SEAOC 11:84 Draft) above those for steel DMF's). It would seem therefore that some inelastic response could be safely permitted in OMF's, suggesting that the requirements proposed in ATC 3-06 and the Tri-Services Manual are in the correct range.

Since seismic code provisions are increasingly being adopted or considered in areas of moderate seismicity (e.g. Boston, Memphis and perhaps New York) a test program to evaluate the cyclic inelastic response of currently common or "vernacular" steel details in those areas would be quite useful.

4. CONCENTRICALLY BRACED FRAMES

The UBC 1982, Tri-Services Manual, and ANSI A58.1-1982 both require that the brace member of Concentrically Braced Frames (CBF) be sized for 1.25 times the axial force resulting from the design lateral seismic load, and that the connections either develop the member or 1.88 times the design axial force (i.e. 1.25 times that force, without the usual one third stress increase). This means that the compression brace will buckle at roughly 1.6 times the design seismic force levels (1.25 x 1.7/1.33). It is essential then that CBF's be detailed to insure good energy dissipating capacity. This is difficult since the hysteresis curves for steel struts under cyclic axial load reversal are pinched and show a significant deterioration in compression capacity (Black et.al. 1980). In an effort to avoid very slender braces, which have performed poorly in past earthquakes, ATC 3-06 requires that the axial compression capacity of a brace be greater than 50% of the tensile strength. For A36 steel this corresponds to a Kl/r limit of 115. The CBF's reserve strength (in X-braces for instance) is thereby doubled as well.

The SEAOC 11:84 Draft requirements for CBF's represent a shift in emphasis to increasing ductility rather than strength. Briefly the provisions include:

a. A limit of $720/(F_y)^{1/2}$ for the brace slenderness ratio L/r (the CBF reserve strength is therefore 2.14 times the design axial load for A36 Steel).
b. A cyclic reduction factor of $1/[(1 + Kl/r)/C_c]$ to be applied to the allowable compressive stress. This provision is still under discussion.
c. A requirement that the brace end connection either develop its tensile strength or 3 times the design response spectrum values ($3R_w/8$).
d. A requirement that in any plane of braced frames an equal amount of compression and tension braces be provided for either loading direction.

Altogether these and other provisions are intended for CBF systems (e.g. X-bracing) which permit the tensile yielding of the brace. Though pinched, the resulting experimental hysteresis curves show stable cyclic inelastic response with no strength deterioration and some energy dissipation. Furthermore, since the brace is sized for compression, the maximum resistance obtained in each half cycle is at least double the design axial force. The important point is that the mechanism assumed (cyclic inelastic buckling and tensile yielding of the brace) is more explicitly manifest in the SEAOC 11:84 code provisions leading a designer to consider how the other elements (beams, columns, connections) must be sized to

Comparison of Lateral Force and Drift Requirements of Several Seismic Codes — Table 1

	UBC 1982	ATC 3-06 [2]	Tri-Services *(ANSI A58.1-82 similar.)	SEAOC 11:84 Draft
Modified Response Spectrum Equation (S = soils factor, max. 1.5; Z, A_a and A_v are zone factors)	$ZS/15T^{1/2}$ (0.14/Z max.)	$1.2A_vS/T^{2/3}$ ($2.5A_a$ or $2A_a$ if S=1.5, max.)	$ZS/15T^{1/2}$ (0.14/Z max.)	$0.5ZS/T^{2/3}$ (1.1/Z or 0.9/Z if S=1.5, max.)
Base Shear Equation	V = ZICSKW	$V = 1.2A_vSW/RT^{2/3}$	V = ZICSKW	$V = ZICW/R_w$
Steel Moment Resisting Frames				
Approx. Period Estimate	0.10 x No. of stories	$0.035h_n^{3/4}$	0.10N	$0.035h_n^{3/4}$
Structural Sys. Factor	K	R	K	R_w
Ductile Mom. Frms.	0.67	8	0.67*	12
Ordinary Mom. Frms.				
High Seismicity Zone.	not incl.	not incl.	1.0 (1.5)[1]* (Type B, h_n less than 48m)	5 or 6 (2-2.4) (h_n less than 48m)
Moderate Seismicity Zone.	0.67 (Zone 1&2)	not incl.	1.0 (1.5)* (Type C to 24m in Zone 1, B to 48m in Zone 1&2)	5 or 6 (2-2.4) (no height limit)
Elastic Drift Limit	0.005K	$0.015/C_d$x1.4	0.005K	$0.04/R_w$
Ductile Mom. Frms.	0.0033	0.0019	0.0033	0.0033
Ordinary Mom. Frms.	0.0033 (1)[1] (Zone 1&2)	0.0027 (1.4)	0.005 (1.5)	0.0067 - 0.008 (2-2.4)
Steel Braced Frames				
Approx Period Estimate	$0.05h_n/D^{1/2}$	$0.05h_n/L^{1/2}$	$0.05h_n/D^{1/2}$	$0.05h_n/D^{1/2}$
Structural System Factor	K	R	K	R_w
Concentrically Br. Fr.				
Brg. Wall Sys.	1.33 (2)[1]	4 (2)	1.33 (2)	4 (3)
Bldg. Fr. Sys.	1.0 (1.5)	5 (1.6)	1.0 (1.5)	6 (2)
Dual Sys.				
w/ DMF	0.8 (1.2)	6 (1.3)	0.8 (1.2)	8 (1.5)
w/ OMF	0.8 (Zone 1&2)	not incl.	0.8 (Type B, Zone 1&2)	not incl.
Ductile Eccentrically Br. Fr.				
Bldg. Fr. Sys.	not incl.	not incl.	not incl.	10 (1.2)
Dual Sys. w/ DMF	"	"	"	12 (1)
Elastic Drift Limit	0.005K	$0.015/C_d$x1.4	0.005K	$0.04/R_w$
Concentrically Br. Fr.				
Brg. Wall Sys.	0.0067 (2)	0.0031 (1.6)	0.0067 (2)	0.01 (3)
Bldg. Fr. Sys.	0.005 (1.5)	0.0024 (1.3)	0.005 (1.5)	0.0067 (2)
Dual Sys.				
w/ DMF	0.004 (1.2)	0.0021 (1.1)	0.004 (1.2)	0.005 (1.5)
w/ OMF	0.004 (1.2) (Zone 1&2)	not incl.	0.004 (1.2) (see above)	not incl.
Ductile Eccentrically Br. Fr.				
Bldg. Fr. Sys.	not incl.	not incl.	not incl.	0.004 (1.2)
Dual Sys. w/ DMF	"	"	"	0.0033 (1)

1 Numbers in parenthesis indicate the ratio of the particular value to that for Ductile Moment Resisting Frames.

2 ATC 3-06 is an ultimate design (or Strength design) code. All the others are allowable stress design codes.

realize the mechanism.

Finally, one may note from Table 1 that the SEAOC 11:84 Draft Structural Systems Factors assigned to CBF's are substantially higher than those in the other codes. This reflects the draft state of this document. The factors have yet to adjust to the improvements in the material sections of the draft code.

5. K-BRACED FRAMES

K-Braced Frames (KBF) differ from X-braced or other CBF's in two respects.

a. Once the compression brace buckles, the strength of the frame is dependent on the beam strength only. The brace will not therefore develop its tensile strength.
b. With repeated load reversals, the buckling capacity of a slender (Kl/r over 80) brace can deteriorate to less than 50% of its original value (Jain et.al. 1979).

Together these observations indicate that the strength of KBF's, after several cycles of loading may be only 25% of what a CBF designed per ATC 3-06 or the SEAOC 11:84 Draft would exhibit. Since there is virtually no data on the performance of KBF's in past earthquakes and very little experimental results it is not clear how severe the problem is.

In the mean time, it has been suggested (Nordenson 1984) that one might either limit the brace Kl/r to around 40, lessening the reduction in buckling capacity, or perhaps view KBF's as two phased systems: a CBF up to buckling and an Eccentrically Braced Frame with flexural links thereafter.

6. DUCTILE ECCENTRICALLY BRACED FRAMES

Ductile Eccentrically Braced Frames (DEBF) are gaining favor as ductile alternates to CBF's. These frames are both quite stiff and capable of large ductility and energy dissipating capacity. The inelastic action is localised in a shear or flexural yielding link and the balance of the frame is designed to develop that mechanism (Kasai et.al. 1984). The SEAOC 11:84 Draft is the first seismic code to include DEBF's. The provisions include the following requirements:

a. the compression strength of the brace should exceed 1.5 times the axial load corresponding to the link yield mechanism.
b. the columns should remain elastic for the given mechanism calculated for 125% of the material yield strength.
c. the maximum link end rotation should not exceed 0.06 rad. at $3R_w/8$ times the design drift (= 3.75 to 4.5 depending on the system).
d. connections and elements must satisfy the DMF requirements.
e. brace to beam connections must develop the brace capacity.
f. links should be laterally braced at their ends and have sufficient web stiffeners ty transfer the brace force and insure adequate ductility for cyclic inelastic load reversals.

7. CONCLUSION

A fair degree of consensus exists regarding the design of DMF's and DEBF's, though there is still debate on certain provisions in the SEAOC 11:84 Draft. The requirements for CBF's are still in development and it is not yet clear what the recommendations will be. Still it seems that the direction is toward an increase in dependable ductility through detailing which could result in a slight reduction in required elastic strength.

OMF's and especially KBF's have yet to be seriously considered partly because there is little data or test results available to substantiate code provsions and partly because, in the case of OMF's there is little interest since the codes are generally geared for zones of high seismicity. The problems associated with KBF's, if experiments confirm the weaknesses hypothesized, could be serious, and since these structures are often used, should receive more attention.

8. ACKNOWLEDGEMENT

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