Proceedings of Ninth World Conference on Earthquake Engineering
August 2-9, 1988, Tokyo-Kyoto, JAPAN (Vol.V)

8-4-5

COMPARATIVE DESIGN OF 19-STORY STEEL BUILDING USING ATC 3-06, UBC 1982 AND CURRENT JAPANESE CODE

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SUMMARY

This paper presents three comparative designs of a 19-story steel building which was originally designed and constructed in Los Angeles. Redesigns using ATC 3-06, 1982 UBC and the current Japanese code are compared, and the differences are demonstrated between U.S. and Japanese aseismic codes, especially how dominant the strength and drift requirements are to a class of tall building of steel structures. Quantities of steel needed are also compared between U.S. and Japanese code design, and a new index of stiffness is proposed and discussed for the comparison of different aseismic codes.

INTRODUCTION

In the achievement of the structural design, regulations and codes have played an important role with regard to safety. Since the regulations and codes are the result among the latest theoretical considerations, past experiences and practical requirements, they have been occasionally revised. In the region of high seismicity actual experiences of earthquake damage of structures have imposed the revision of aseismic regulations and requirements.

Redesigns of a 19-story steel building in Los Angeles are presented in accordance with ATC 3-06, 1982 Uniform Building Code (UBC) and the current Japanese code. The building was originally designed and constructed using the 1964 City of Los Angeles Building Code and was examined as an example model for redesign to comply with ATC-3 and 1982 UBC requirements (Refs. 1-3).

For 1982 UBC and ATC 3-06 provisions, only the equivalent lateral force procedures are used on a three dimensional model so that the effects of accidental torsion could be applied directly. Two dimensional analyses in each direction are used for meeting the Japanese code. The Japanese analyses include a static preliminary analysis plus a check using elastic dynamic analyses at allowable stresses and higher level limit state analyses using plastic design procedure.

It will be of interest to redesign the same building originally conformed to a previous U.S. code by using another U.S codes and the current Japanese seismic code. This paper will clarify the differences between U.S. and Japanese aseismic codes, especially how dominant the strength and drift requirements are to a class of tall building of steel structures. Quantities of steel needed are also compared between U.S. and Japanese code designs.

REDESIGNS USING ATC 3-06 AND 1982 UBC

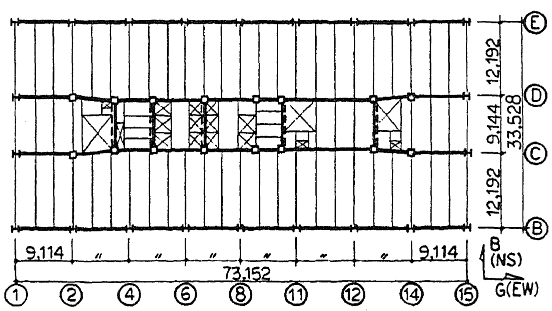
The building consists of a 19-story tower of 34m x 73m (110ft x 240ft) in plan dimension and a 4-level basement of 92m x 97m (302ft x 318ft). Total height of the structure is 82m (270ft). Figure 1 illustrate the changes of seismic framing systems employed in 3 redesigns. Frame elevations of B(NS)-direction in the Japanese code design are also sketched in Fig.2.

ATC 3-06 Design Drift requirements are even more the dominant criteria in this design (2:1 over that required by 1982 UBC). Meeting the drift requirements in the B(NS) braced frame direction also determines the sizing for the G(EW) moment system so that neither strength nor drift were limiting criteria for the moment frames. Final steel tonnage is 146% of that for the 1982 UBC design and 105% of that required for the current Japanese code design.

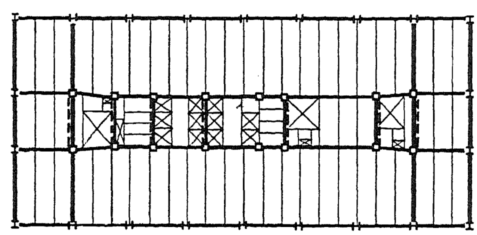
1982 UBC Design Drift requirements are predominant in both G(EW) and B(NS) directions. As the braced frame elements are slender (10: 1), the drift control is particularly severe in this direction. Final steel tonnage is 72% of that design by Japanese provisions and 69% of that required by ATC 3-06.

REDESIGN USING CURRENT JAPANESE CODE

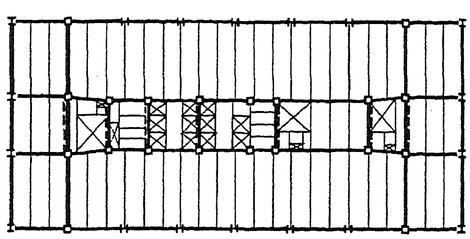
Aseismic design of building structures in Japan necessitates both the allowable stress design based on elastic analyses to moderate earthquake motions and the limit design based on plastic analyses to severe earthquake motions, where the standard shear coefficient shall be not less than 0.2 and 1.0, respectively.



(a) Original and 1982 UBC Design



(b) ATC 3-06 Design



(c) Japanese Code Design

Heavy lines indicate seismic resisting system and broken lines indicate X-brace.

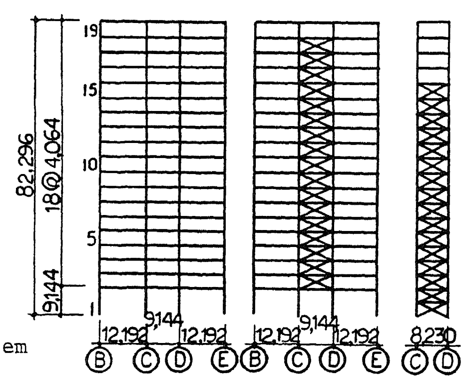


Fig.2 Frame Elevation of B(NS)-Direction in Japanese Code Design

Fig.1 Typical Floor Framing Plan

The outline of the building and Japanese code design are summarized in Table 1. Though the building has 4-floor basement and 2-story penthouse, major concern is restricted to the 19-story tower. Aseismic framing system of the structure is the same as that employed in ATC 3-06 redesign as shown in Figs.1 and 2, except that the outrigger girders in the 2nd and 14th lines are rigidly connected to the outer columns and that X type braces in a few floors at the top are removed. Dynamic response analyses are required for aseismic designs of a class of this building exceeding 60 meters (approx. 200 ft). Requirements of drift, strength and ductility stated in Table 2 for two levels of earthquake motions must be satisfied.

Lateral Loadings Lateral loading of seismic and wind loads are sketched in Fig.3, where seismic shear loads based on Building Standard Law are also illustrated for reference. Total weight of the building above the ground level for seismic design amounts about 25,300 tons (55,800 kips), thus the weight per unit floor area is 530 kg/m^2 (110 psf). Seismic lateral shear forces are determined by preliminary dynamic response analyses, where 0.080 is the lateral shear coefficient at the first story of moment resisting frames in the G direction while 0.145 of braced frames in the B direction. Lateral shear forces due to seismic loads are at least two times as large as those due to wind loads in the both directions.

Stress Analysis The structural frames are modeled as plane frames with base at first basement floor level in each direction and analyzed by the computer program SAP-V. Panel zones are considered by modification to beam and column stiffness. Lateral deformation and ratios of distributed shear forces to each frame are shown in Fig.4.

Design of Steel Members Members and connections are proportioned and selected considering adequate strength, stiffness and ductility, ease of erection and economy. To ensure the stable and ductile aseismic capacity of the structure, slenderness ratio of compressive members and axial force ratio of columns to yielding strength are restricted following AIJ guide for plastic design structure as well as width thickness ratio of steel sections. Corner columns are designed to carry additional axial forces due to seismic load in another direction.

Table 1 Outline of 19-Story Building

Building	:	Building No.2 in the ATC-2 report
Usage	:	Office
Location	:	Osaka, Japan
Floor Area	:	total = 47,920 m^2 (515,800 ft^2) *1
		typical floor = 2,453 m^2 (26,400 ft^2)
Number of Stories	:	19 and 4-level basement and 2-story penthouse
Height	:	total = 82.296 m (270 ft)
		typical story = 4.064 m (13 ft 4 in), 1st story = 9.144 m (30 ft)
Typical Bay Size	:	G(EW) = 9.144 m (30 ft)
		B(NS) = 12.192 m (40 ft) 8.230 m (27 ft)
Plan Dimension	:	73.152 m x 33.528 m (240 ft x 110 ft)
Type of Structure	:	Framed Steel Structure
Framing	:	G(EW) = Moment Resisting Frame
		B(NS) = Moment Resisting Frame and Braced Frame
Shape of Members	:	Column □ H Beam I Brace H
Slab System	:	Light-weight Concrete and Metal Deck
Natural Period	:	G(EW) = 3.16 sec. , B(NS) = 2.37 sec.
Shear Coefficient	:	G(EW) = top 0.238 , 1st story 0.080
		B(NS) = top 0.343 , 1st story 0.145
Weight	:	total = 25,300 tons (55,800 kips), typical floor = 1,240 tons (2,740 kips)

*1 basement and penthouse are not included

Table 2 Criteria of Safety for Aseismic Design

Maximum Velocity of Ground Motion	Maximum Response of Story Angle of Drift	Maximum Response of Story Ductility *1
25 kine (0.82 ft/s)	less than 1/200	less than 1.0 (elastic)
40 kine (1.31 ft/s)	less than 1/100	less than 2.0

*1 story ductility is defined as ratio of story drift to the drift corresponding to elastic strength limit of the story

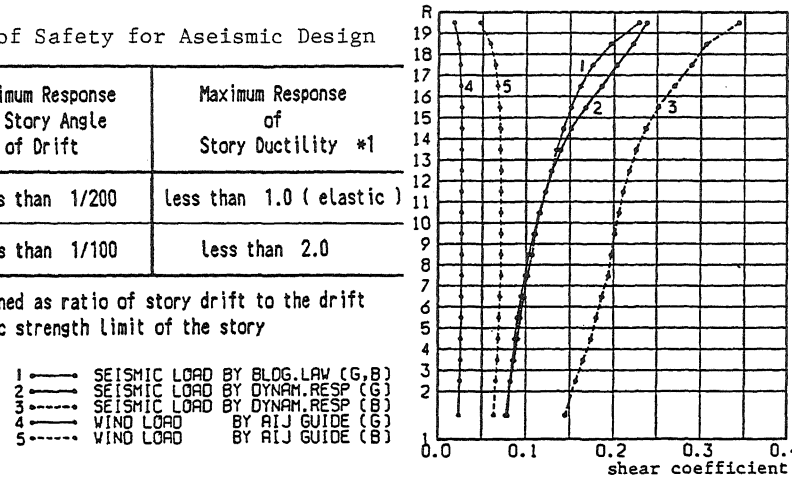
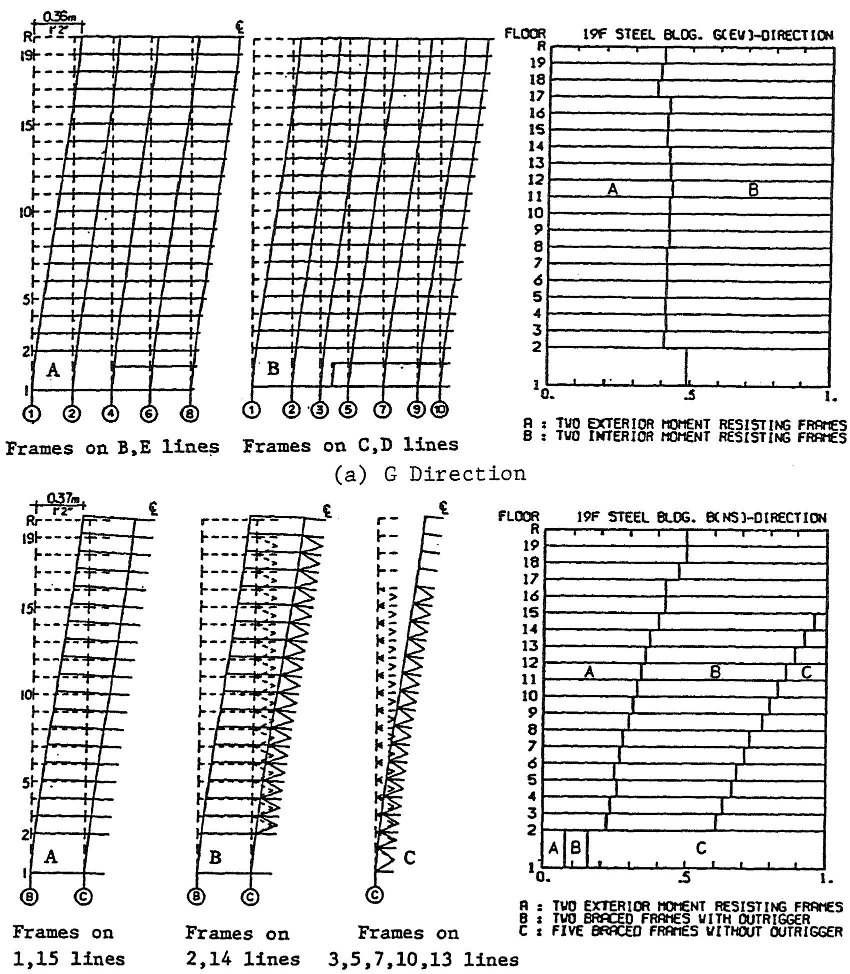


Fig.3 Distribution of Lateral Shear Loads



(a) G Direction

(b) B Direction

Fig.4 Ratio of Seismic Lateral Shear Distribution to Each Frame

Dynamic Analysis and Seismic Safety of Structure Dynamic eigenvalue analyses are performed using the same frame model as static frame analyses to reduce an equivalent shear-type lumped mass model for dynamic responses. Fundamental periods of the structure are 3.16 sec. in the G direction and 2.37 sec. in the B direction. Expected maxima of velocity at earthquake bedrock are listed in Table 3. Peak value of input motion to the structure is decided considering the amplification of the surface soil. Three accelerogram data, El Centro 1940 NS, Taft 1952 EW

Table 3 Expected Velocity of Earthquake Motion

City	Return Period (Year)	Expected Velocity at Bedrock (cm/s)	
		Mean	Mean + Standard Dev.
Osaka	50	3.3	5.4
	100	4.4	6.1
	150	5.2	6.3
Tokyo	100	4.5	9.1

Amplification from bedrock to ground surface is estimated by Kanai's spectrum G(T). When the dominant period of bedrock motion T (sec.) coincides with that of surface soil Tg (sec.), amplification is given by $5\sqrt{Tg}$. Since Tg of the Umeda soil in Osaka city is between 0.3 and 0.5 sec. by microtremor observation, the amplification of surface soil becomes 3 or 4. Then we get two levels of peak velocity expected at the ground surface, i.e., 6 cm/s x 4 and 1.5 x 6 cm/s x 4, which leads to 25 cm/s and 40 cm/s for aseismic design.

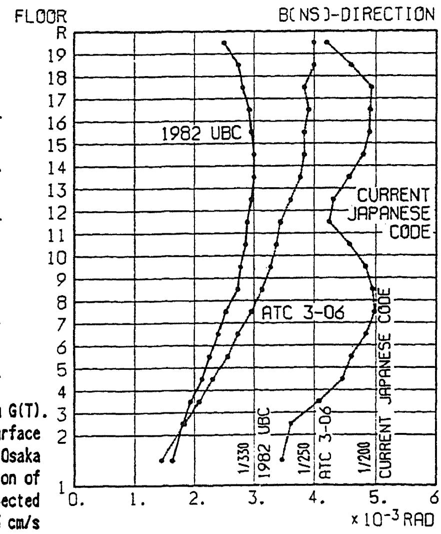


Fig.5 Comparison of Resulted Drift

Table 4 Maxima of Earthquake Response and Safety of Structures

Maximum Velocity of Input Motion	Direction of BLDG.	Story Drift Angle Requirement	Strength and Ductility Requirement
25 cm/s	G (EW)	14F : 1/211 < 1/200	1F : 0.95 < 1.0
	B (NS)	7F : 1/201 < 1/200	6F : 0.95 < 1.0
40 cm/s	G (EW)	14F : 1/132 < 1/100	1F : 1.51 < 2.0
	B (NS)	8F : 1/126 < 1/100	6F : 1.52 < 2.0

Damping ratio of the steel structure is assumed 2 % of critical.

Table 5 Comparison of Code Requirements and Quantity of Steel

	Natural Period T1(sec)		Shear Coefficient at 1st Story Cs		Story Drift Angle R		Stiffness Index Cs/R		Steel Quantity
Direction	G(EW)	B(NS)	G(EW)	B(NS)	G(EW)	B(NS)	G(EW)	B(NS)	(tons)
ATC 3-06 *1	3.16	2.36	0.036	0.072	1/370	1/330	13.3	23.8	7271
1982 UBC *2	3.20	3.27	0.025	0.030	1/300	1/250	7.5	7.5	4989
Japanese Code	3.16	2.37	0.080	0.145	1/200	1/200	16.3	29.0	6943

*1 R is calculated by 3/(200xCd);Cd=5.5(G),5.0(B) *2 R is calculated by k/200;k=0.67(G),0.80(B) and Osaka 205 1963 EW are used for input motion with peak acceleration values corresponding to peak velocity of 25 and 40 cm/s. In the G direction both drift and strength or ductility responses are uniform throughout the height and both requirements govern the design, while in the B direction the drift requirement is more critical. As summarized in Table 4, maximum responses of drift and strength or ductility meet the criteria stated in Table 2. Thus, aseismic performances of the structure will be satisfactory.

COMPARISON AND DISCUSSION

The resulted drifts by three designs are demonstrated in the B(NS) direction in Fig.5. As summarized in Table 5 drift limits by ATC 3-06 and 1982 UBC are 1/333 and 1/250, respectively. They are 1.66 and 1.25 times as severe as the drift limit 1/200 in the Japanese design. On the other hand, the required base shear coefficients 0.072 by ATC and 0.030 by UBC are about 1/2 and 1/5 of that by the Japanese design. But the steel quantity by Japanese code design almost equals to that by ATC 3-06 design. In the comparison of different codes, therefore, the required strength represented by the base shear C_s and the deflection constraint R should not be discussed separately. In order to take account of both requirements, the stiffness index I_s as defined by the following expression may be a better rule.

$$I_s = C_s / R \tag{1}$$

Table 5 also explains that as far as the stiffness indices make no wide difference between two design codes just as Japanese and ATC 3-06 codes, resulted buildings may have the comparable stiffness and quantity of materials.

CONCLUDING REMARKS

Comparative designs of the steel structure by U.S. and Japanese codes have been discussed in this paper through the case study of a 19-story building. The building had been originally designed and constructed in accordance with 1964 City of Los Angelese Building Code and here have been redesigned to comply with ATC 3-06, 1982 UBC and the current Japanese code, respectively. Quantities of the steel needed have been also estimated and compared between U.S. and Japanese code design.

It should be emphasized that the current Japanese code imposes the design shear force of 2 or more times as large as U.S. codes, while the drift requirements by U.S. codes are severer than those by the Japanese code. The resulted steel quantities of the U.S. and Japanese designs, therefore, do not make so remarkable differences as might be expected only through the design shears. It may be reasonably concluded that the stiffness index newly defined in this paper would be a better rule rather than the design force or deflection alone for the code comparison of the steel structures where the drift requirements sometimes govern the structural design.

REFERENCES

1. ATC Report, Redesign of Three Multistory Buildings : A Comparison Using ATC 3-06 and 1982 Uniform Building Code, 7-34, ATC-2-04.
2. Pinkham, C.W., Comparison of Design Procedure and Results of 19-Story Steel Frame Building Using Provisions from UBC82, ATC-3-06 and Japan Building Standard Law, ATC 15-1, Proc. 2nd U.S.-Japan Workshop on Improvement of Seismic Design and Construction Practices, 373-374, (1986).
3. Hisatoku, T. and Nagase, T., Redesign of A 19-Story Steel Building in Los Angeles Using Current Japanese Code, ATC 15-1, Proc. 2nd U.S.-Japan Workshop on Improvement of Seismic Design and Construction Practices, 353-372, (1986).