ON THE OPTIMAL CHOICE OF THE SHAPE OF ANTISEISMIC BRACING SYSTEMS

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SUMMARY

This paper aims to point out how steel bracing systems, though usually collected in a single structural class by the current antiseismic codes, actually show widely different requirements of ductility under earthquake excitation and therefore should be subjected to different design forces according to their shape and connection mode. A mathematical model of the brace is shown which allows to numerically simulate the rather intricate behaviour of such structures. The dynamical analysis of a few sample bracing systems is performed through the aforementioned procedure and their comparison provides a first estimation of the behaviour of the most widely adopted shapes.

PROBLEM POSITION

Current antiseismic codes (Refs. 1,2,3,4), allow for important reductions of inertia forces acting on structures in order to account for their ductile behaviour when energy dissipation phenomena due to plastification occour.The adopted reducing coefficients are called "design behaviour factors" (d.b.f.) and vary according to each design code and structural class.

Up to now the small number of such classes, even if they are clearly identified, does not allow for precisely determining the d.b.f.; moreover the evaluation of such factors has been widely based on engineering judgement, by considering examples which were too simple to represent a real structure or too few to allow generalizations; hence usually the adopted values are strongly conservative to take into account the scattering coming from not considering with enough accuracy the correlations between d.b.f. and structural type, d.b.f. and design criteria. Structures belonging to the same group but designed on the basis of different criteria or differing in shape may exhibit actual d.b.f. which scatter in a wide range and differ from the conventional d.b.f. assumed by codes.

In particular the bracing systems, which provide interesting solution to make both new and existing buildings economically antiseismic, have been

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often heavily penalized by the d.b.f. assigned to them. Such handicap can be ascribed, at least in part, to the bad results shown by some real structures and to the difficulty of adequately modelling the behaviour of these systems under severe cyclic loading, while intuitive considerations about their high stiffness suggest the adoption of larger design forces.

Moreover, due to the deep difference among their usually adopted shapes (e.g.X-,K-types,etc.) and connection modes (bolted or welded), and to the variety of the possible location within the structure, they exhibit a large scattering of the actual d.b.f..

The present paper aims to give a contribution to the evaluation of the d.b.f. for bracing systems by comparing the ductility requirements under severe earthquakes of a few sample structures of the most widely adopted shapes. The comparison is performed for different heights of the structures adopting various hysteretic behaviours of the members.

The dynamic analysis of the bracing sistems is carried out through a recently implemented computer program based on a brace model exhibiting a multilinear constitutive law (Ref. 6), which allows to account for geometrical nonlinearities, elastic-plastic-hardening behaviour of the material, post-buckling behaviour of truss members and for the effects of cumulative damages affecting strenght and/or stiffness at low cycle fatigue (e.g.enlargement of clearances in bolted connections).

MODEL OF MEMBER BEHAVIOUR

A merely qualitative description is presented herein of the element model ruled by a multilinear constitutive law, already adapted to allow a sufficiently accurate schematization of the truss element behaviour. The analitical description of the complete model which can represent the behaviour of different structural elements (reinforced concrete, masonry) can be found in (Refs. 5, 6).

The aforementioned multilinear constitutive law is shown in Fig.1. It is defined through the slopes $K_1, K_2, \dots, K_7$ of its 7 linear branches corresponding to the different stiffnesses exhibited by the member during each cycle, and through the 3 limit values of axial force f_1, f_2, f_3. The branches 1 , 3, 6 may be covered in both ways (elastic behaviour), whereas the branches 2, 4, 5, 7 can be covered in the cycle direction only (plastic behaviour). As it can be easily understood by looking at Fig. 1 the main features of the model are:

a) the slopes K_3 and K_6 are at the beginning equal to the slope K_1 but during the loading history branch 3 holds unchanged the initial slope reduces when covered downward, while when covered upward its slope reduces according to the maximum value of positive deformation " max" attained from the beginning through a coefficient equal to $[1 - (\delta^+ max/\delta^+ lim)^m]$, where $\delta^+ lim$ is a suitable limit value; viceversa the slope K_6 is changed both when is covered downward and upward according to the current value of negative deformation " δ^- ", as shown in Fig. 2.

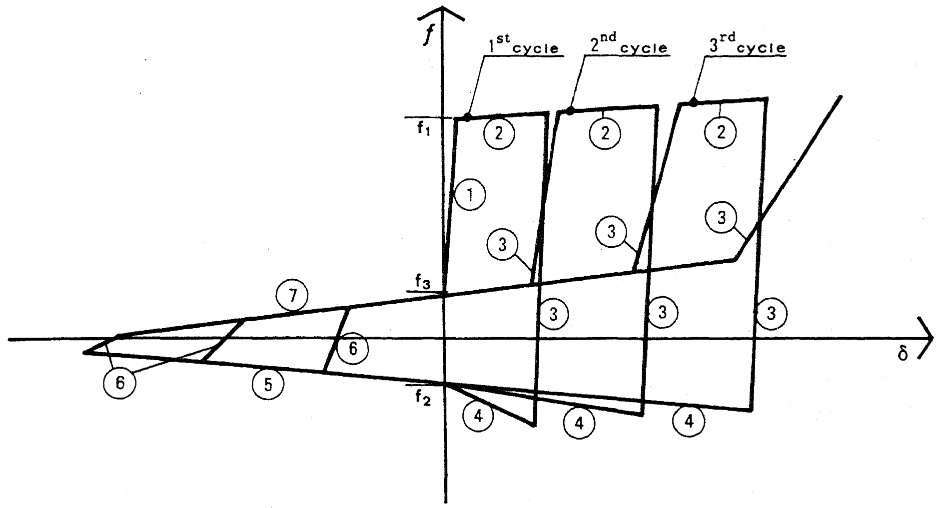


Fig. 1

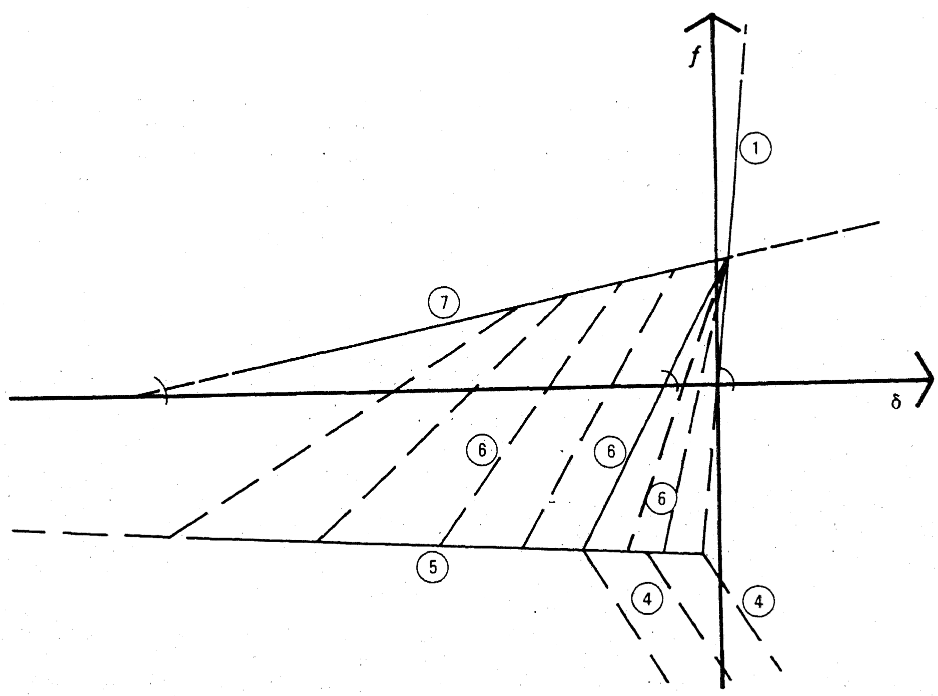


Fig. 2

b) The abscissa of the intersection point between branches 7 and 3 linearly depends on the maximum value of positive deformation $\delta^+ max$ ($\delta_i = c_L \delta^+max$)

c) the slope K_4 varies according to "δ^+max", through a coefficient equal to $\{1- [(\delta^-el+2\,\delta^+el)/(\delta^-el+\delta^+max)]^2\}$, where δ^-el and δ^+el are the limit elastic values of axial elongation in compression and in traction respectively. Furthermore, the slopes of branches 3, 4, 6, and the limit values of axial force f_1 , f_2 , f_3 may depend on the total dissipated energy through a coefficient equal to $[1-(\Omega/\Omega lim)^n]$, where $\Omega\, lim$ is a suitable limit value of the dissipated energy, allowing for a continuous transition from low to high cycle fatigue range. It is worth noticing, however, that usually the influence of the dissipated energy does not significantly affect the model behaviour under seismic excitation due to low number of severe peaks in the earthquake accelerograms.

Therefore the model behaviour during its loading history is described by a certain number of parameters (namely: K_1, K_2, K_4, K_5, K_6, K_7, f_1, f_2, f_3, δ^+lim, m, Ωlim, n, c_L), which have to be estimated on the basis of experimental results (e.g. Refs. 7,8,9,10,11). A few samples of calculated values for the parameters of real braces are presented in the examples shown in the following. Such estimation in eased by some qualitative considerations which come from examining the aforementioned experimental results:

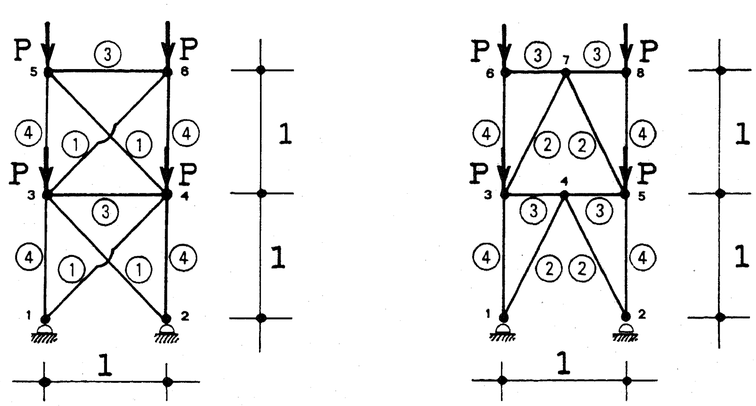
a) on the whole, slenderness ratio appears to be the most important single parameter in determining the brace behaviour; with a certain degree of approximation it is even possible to define an hysteretic behaviour normalized with respect to slenderness ratio;
b) cross-sectional shape, positioning the weak axis in plane of the structure or out, distance between stitches and their position along the member, by determining the brace's susceptibility toward lateral-torsional buckling, local buckling of outstanding legs and web buckling, can somewhat modify the hysteretic behaviour in compression of the member, but their influence is far less important than slenderness ratio's one and, mainly, they rule only the shape of the initial cycles;
c) eccentricity of load due to connection shape also leads to modifications only in the initial cycles behaviour, decreasing earlier the member capacity of carring loads in compression;
d) connection type, bolted or welded, makes little difference in the hysteretic loop shape but significantly changes the expected ductility of the member; in fact , while ductility values in compression can be good enough in both cases (≥ 20), in tension the difference between bolted and welded connections is considerable, since it appears difficult to reach values greater than $6 \div 10$ in the first case, while in the second case they can attain to the typical ductility of the material (≥ 40).

BEHAVIOUR OF SAMPLE BRACING SYSTEMS

An exhaustive comparison among most widely adopted shapes for bracing systems should be performed finding, for each shape, within each field of practical use (e.g., mono- or multi-storeyed industrial buildings, housing and office buildings, strengthening structures for existing not-antiseismic buildings etc.) and for each connection mode, the design criteria maximizing the d.b.f.; then choosing that shape which exibits the maximum d.b.f. Such procedure requires a considerable quantity of numerical analyses (even if the number of required specimens are reduced through a choice based on the experimental design criteria) which the authors are trying to perform at least within the bounds due to the small number of experimental results available for estimating the parameters governing the model of member behaviour.

Among the numerical results already obtained within such research field, a comparison is presented herein between the X and the K bracing systems shown in Fig. 3 and Fig. 4 which have been adopted in making antiseismic small buildings. Their heights are actually the average and the maximum in such kind of applications, while in the same Figs. 3 and 4 are shown dimensions, dead loads, masses of each floor and design acceleration factors pertinent to each structure.

All the structures have been designed to attain yielding or buckling of a member under inertia forces linearly distributed along the heigh with resultant equal to the product of the sum of the floor masses by the



a) b)

l = 300. cm ; P = 58.8 KN (vertical dead load)

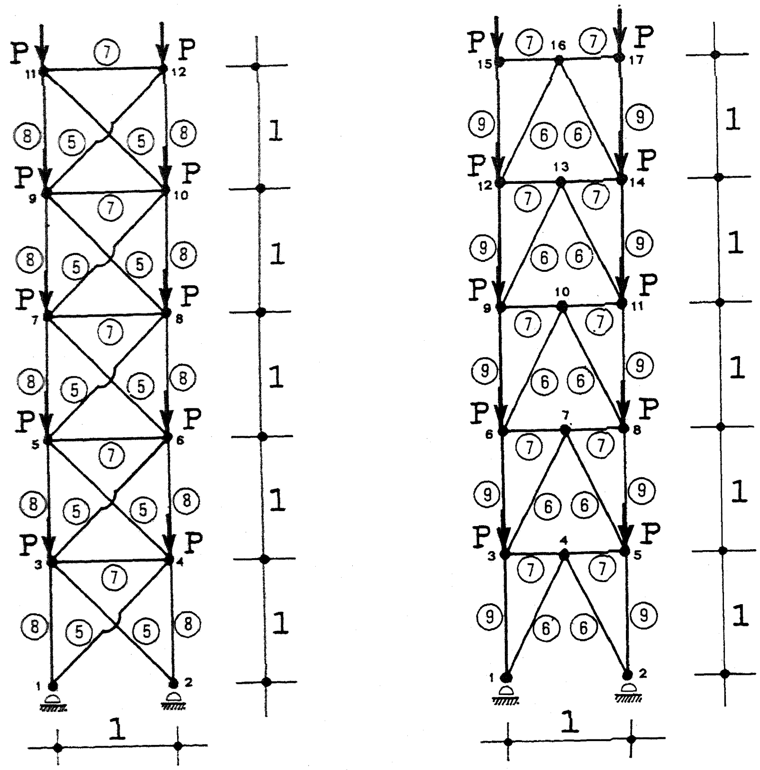
M = 600. N sec²/ cm (mass of each floor).

A_d = 0.1 g (design acceleration factor).

1 = 2 L 65x7 2 = 2 L 65x9

3 = IPE 100 4 = HEB 120

Fig. 3



a) b)

l = 300. cm , P = 58.8 KN (vertical dead load)

M = 600. N sec^2/ cm (mass of esch floor)

A_d= 0.2 g (design acceleration factor)

(5) = 2 L 100x12 (6) = 2 L 100x14

(7) = IPE 140 (8) = HEB 220 (9) = HEB 200

Fig. 4

aforementioned design acceleration factors. Such criterion leading to X- and K- structures about of the same weight allows a more significant comparison. Furthermore welded connections have been assumed for all the structures (no enlargement of clearances permitted).

Moreover, in order to evaluate the importance of a proper schematization of the member behaviour, the results associated with three different models of such behaviour will be shown in the following:

a) indefinitely elastic behaviour;
b) elastic-plastic behaviour, with bounds in tension corresponding to the yielding stress and in compression equal to the buckling value obtained through the method;

c) the complete model of member behaviour presented in the previous paragraph.

Tab. 1 shows the values, deduced from experimental results, assigned to the model ruling parameters, previously mentioned , to obtain respectively the c) and b) behaviours (about the c) behaviour, the parameters not mentioned in Tab. 1 $\Omega \lim = \infty$, $m = 1/3$, $n = 1$, $c_L = 1/1{,}07$ have the same

Member	Complete model						Elastic-plastic Model			
	f_1 (KN)	$f_2 = f_3$ (KN)	$K_1=K_3==2K_6$ (KN/cm)	K_4 (KN/cm)	$K_7=2K_2=\frac{3}{2}k_5$ (KN/cm)	$\delta^+ \lim$ (cm)	$f_1=f_3$ (KN)	f_2 (KN)	$K_1=K_3=K_6$ (KN/cm)	$K_2=K_4=K_5=K_7$ (KN/cm)
1	388.8	62.2	844.6	- 14.1	1.5	20.5	388.8	71.9	844.6	.75
2	491.6	93.1	1351.5	- 126.7	"	16.5	491.6	118.5	1351.5	"
3	230.1	185.2	706.6	- 149.5	"	14.5	230.1	185.2	706.6	"
4	759.7	284.2	2332.4	-1177.1	"	14.5	759.7	421.4	2332.4	"
5	1014.4	254.8	2203.6	- 438.7	"	20.5	1014.4	400.8	2203.6	"
6	1170.8	392.0	3219.1	-1494.3	"	16.5	1170.8	655.6	3219.1	"
7	366.4	332.7	1125.0	- 99.9	"	14.5	366.4	332.7	1125.0	"
8	2033.3	1766.0	6242.6	- 658.6	"	14.5	2033.3	1766.0	6242.6	"
9	1745.2	1522.9	5357.7	- 478.5	"	14.5	1745.2	1522.9	5357.7	"

Tab. 1 Model ruling parameters

Structure shown in	Behaviour of members	in tension		in compression	
		member	ductility	member	ductility
Fig. 3a	elastic	1-4	5.57	2-3	5.72
	elastic-plastic	2-3	10.65	2-3	16.98
	complete model	2-3	11.93	1-4	18.17
Fig. 3b	elastic	1-4	5.69	4-2	5.86
	elastic-plastic	4-2	12.73	4-2	17.22
	complete model	4-2	13.53	1-4	20.35
Fig. 4a	elastic	1-4	5.92	2-3	5.87
	elastic-plastic	2-3	11.03	1-4	15.16
	complete model	1-4	14.62	1-4	17.46
Fig. 4b	elastic	1-4	6.05	4-2	6.21
	elastic-plastic	1-4	13.09	4-2	17.74
	complete model	4-2	19.07	4-2	21.13

Tab. 2 Ductility requirements

value for all members, while, obviously, the b) behaviour is completely described by the values of the two limit forces and two branch slopes shown in the last four columns of Tab. 1).

Finally, Tab. 2 presents the maximum ductility requirements both in tension and in compression of each sample structure, for each adopted model of member behaviour, under seismic excitations corresponding to the accelerograms of Tolmezzo E-W May/6/1976 and El Centro N-S May/18/1940.

REFERENCES

1. Italy: Min.LL.PP., D.M. March, 3 - 1975, "Norme Tecniche per le costruzioni in zone sismiche", Suppl. Ord. G.U. n.93, April 8, 1975.
2. U.S.A.: A.T.C. "Tentative Provisions for the the Development of Seismic Regulations for Buildings", A.T.C. Pub. No. 3-06, U.S.Dep.of Commerce, June 1978.
3. New Zeland: "Code of Practice for the Design of Concrete Structures" and "Commentary on....", Standard Association of New Zeland, Wellington, D2 3101, Part 2, 1978.
4. Japan: "Standards of aseismic civil engineering constructions in Japan" Building Standard Law Enforcement Order, Ministry of Construction Notification No. 1074.
5. U. Andreaus, G. Ceradini, P. D'Asdia: "Modelling of structures exhibiting internal constraints not perfectly bilateral" (in italian), Proc. 6th Nat. Congr. of Theor. and Appl. Mech., Genoa, October 1982, Vol. 5, pp.302 - 311.
6. U. Andreaus, G. Ceradini, P. D'Asdia: "A Simple model for the dynamic analysis of deteriorating structures", Proc. 7th Conference on Structural Mechanics in Reactor Technology, Chicago, August 1983, Vol. L, pp. 591 - 598.
7. M. Shibata et al.: "Elastic-Plastic Behaviour of Steel Braces under Repeated Axial Loading", Proc. 5-th W.C.E.E., Rome, June 1974, Vol. 1, pp. 845 - 848.
8. M. Wakabayashi et. al.: "Cyclic Behaviour of Restrained Steel Brace under Axial Loading", Proc. 6th W.C.E.E., New Delhi, January 1977, Vol.3, pp. 3181 - 3186.
9. M. Wakabayashi et al.: "Hysteretic Behaviour of Steel Braces subjected to Horizontal Load due to Earthquake", Proc. 6th W.C.E.E., New Delhi, January 1977, Vol. 3, pp. 3188 - 3193.
10. E. P. Popov, R.G. Black: "Steel Struts under Severe Cycling Loadings", J. of the Struct. Eng. Div., ASCE, Vol. 107, No. ST9, September 1981, pp. 1857 - 1881.
11. G. Ballio, G. Campanini: "Comportamento di aste di controvento soggette a cicli alterni di carico", Giornate Italiane della Costruzione in Acciaio, C.T.A., Palermo, October 1981, Vol. I, pp. 57 - 70.