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Local buckling and early fracture of cold-formed steel members

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ABSTRACT

Although recent investigations are well proceeding on basic properties of cold-formed members with square or rectangular tubing sections. Namely, early fracture of those members is much anticipated to take place under severe seismic loadings. This significant problem is directly related to the seismic safety of steel structures using the members with cold-formed tubing sections. This paper deals with, firsts a review of recent studies related to the problem. Then, an example of early fracture of a cold-formed bracing member is shown associated with an analysis that indicates the effect of the fracture on the overall structural behavior. Finally, possible measures and research needs against the danger are proposed based on related studies.

INTRODUCTION

Early fracture of cold-formed steel members is anticipated to occur by future major earthquakes. This is because: 1) recently cold-formed square or circular tubing sections have come to be often used as columns and bracing members; and 2) severe stress and strain in the critical sections of such members, induced by severe earthquakes, particularly associated with local buckling. Needless to say, investigations are proceeding well on basic properties of cold-formed steel sections and on the behavior of cold-formed members (Karren, 1967; Kato, 1978; Kato, 1986). However, less attention has been given to the crisis of cold-formed members with square or rectangular tubing sections.

In this paper, first of all, the past studies on this critical problem will be reviewed. Then, some remarkable findings from the US/Japan full-scale seismic test (Yamanouchi, 1989) and analysis will be briefly discussed; early fracture of one bracing member took place actually in the test. In addition, the current design requirements for such bracing members will be critically discussed. Finally, several practically possible measures against such premature fracture will be discussed on the basis of related studies. Also in this discussion, the technical problems to be solved as soon as possible will be identified considering the prevailing use of cold-formed sections as bracing members or other major structural members.

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It is well known by recent studies that square or rectangular tubular members are susceptible to local buckling. Thus, limitations of width-to-thickness ratio are stipulated in design codes of several countries for these sections. However, there have been few excellent experimental studies focusing on the local buckling and fracture of square tubular members under cyclic loading. In particular, the local buckling and following cracking is very sensitive to the so-called scale-effect. Thus, to attain experimental knowledge on these problems, it is significant to use full-scale test specimens that generally need large-scale testing facilities and high experimental costs.

Before the US/Japan full-scale test, Gugerli and Goel (Gugerli, 1982) carefully tested full-size braces with cold-formed rectangular tubing sections. According to this study, local buckling occurred in the initial stage of severe inelastic cycling. After this, complete rupture took place in a few inelastic cycles. Consequently, the study found that: 1) the width-thickness ratio was the most significant key factor to the influence and severity of local buckling; 2) severe local buckling adversely affected the fracture life of the cold-formed tubular members; and 3) as the slenderness ratios of the bracing members became smaller, the fracture of the members occurred earlier with strains much concentrated in locally buckled sections. Further, Tang and Goel (Tang, 1987) developed a criterion to predict fracture lives of rectangular tubular braces in terms of normalized cyclic displacements, on the basis of bracing member tests conducted by Liu (Liu, 1987). According to the study, the empirical fracture life is inversely proportional to the square of width-thickness ratio and is proportional to the slenderness ratio and the width-depth ratio of the section. Recently, Lee and Goel (Lee, 1987) have proposed a refined fracture criterion for square or rectangular tubular bracing members. In this formulation, the following assumptions based on the experimental results have been added to the Tang's formulation: 1) the fracture life increases as yield strength decreases; 2) the fracture life is independent of the slenderness ratio, KL/r; 3) the loading history until the first overall buckling has no effect on the fracture life; and 4) large tension forces in a member have more dominant effect on the fracture life than small tension forces. The above studies identified pivotal factors influencing the fracture life of square or rectangular tubular members.

EARLY FRACTURE OF COLD-FORMED BRACING MEMBERS IN FULL-SCALE TEST STRUCTURE

Findings from Full-Scale Test

The bracing members of the US/Japan full-scale six-story test structure (Yamanouchi, 1989) were made of the cold-formed square hollow sections having the steel of ASTM A500 Grade B. The principal dimensions of the bracing members are listed in Table 1.

By the test results from the Moderate test (2.5m/sec^2 peak input), several bracing members exhibited slight softening in their cyclic behavior as shown in Table 2. Further, these braces had overall buckling in the Final test (5.0m/sec^2 peak input). Immediately after these buckling, local buckling occurred at critical sections. At last, at 11.135sec in the input seismic record, the north brace at the third story level completely ruptured (Yamanouchi, 1989).

To discuss the test results on the damaged braces in more detail, the strain energy absorbed in the braces during the Final test is listed in Table 2. This amount of energy has been obtained from the total area of the axial force versus displacement hysteresis loops. The second column of this table shows the values of the energy normalized by the volume per unit length for each brace. These values indicate approximately the energy dissipated in plastic hinges in each braces, since inelastic deformations concentrate in the hinges located at the mid-span and both the ends of the brace. Looking at these normalized energy, the values on the north brace in the third story and both the braces in the second story are much more dominant than those on the other braces. On the other hand, although the north brace in the first story marks a considerably larger amount of energy (Table 2), no local buckling was observed. This is chiefly because the width-thickness ratio of the section of this brace is quite smaller than that of the other brace sections as listed in Table 1.

Another peculiarity of the brace rupture is the fast pace of crack propagation. Fig. 1 shows the time history of the axial displacement of the ruptured brace, associated with the crack growth recorded by the observation. By this figure, the brace underwent only three inelastic displacement cycles with the amplitude of 6 to 8 times the yield displacement, Δy. In addition, it was observed that the crack appeared initially at the corner of the concave side of the locally buckled section; the crack then propagated rapidly toward the compressed flange and both the webs as well. Then, the cracks came rapidly into the flange plate, leading to the complete rupture of the brace.

By this premature fracture of the brace in the full-size test that simulated seismic responses, it is authentic to anticipate similar failure in real structures by severe earthquake ground motions.

Analytical Response of Test Structure after Brace Rupture

The Final test was terminated just after the rupture of the north brace at the third story, at 11.135 sec in the seismic record. Thus, it is not only interesting but also practically significant to know or presume how the response of the test structure would have proceeded after the brace rupture if the test was continued. One way to attain this knowledge is to perform a dynamic response analysis that can simulate pertinently the response of the structure after the brace fracture. In the analysis of this paper, thus, the north brace of the third story was assumed to lose completely its axial resistibility at the same time as the experimental rupture time (11.135sec). A variation of the DRAIN-2D computer program was used for this analysis.

As a result, Fig. 2 shows the time history of interstory drift of the third story. Before the brace rupture, the drift varied chiefly in the negative region; this means that the north brace received cyclically larger compressive deformations than the south brace that was mainly under tension. By the analysis, around 15 sec, the maximum interstory drift angle of 1/52.5 would then occur in the positive side. Up to this moment, probably other braces would have ruptured one after another as noted by Tang and Goel (Tang, 1987). By their analysis, the maximum story drift angle of 1/19 would occur after 16 sec in the fifth story, associated with a large shift of the drift to the negative direction. The above maximum story drift angle at the fifth story level implies the collapse or very dangerous state of the structure. From these analyses it can be recognized that the rupture of a bracing member makes the interstory drift of the story extremely large. Of course, this peculiarity of response depends on the dominant period or spectral content of input earthquake ground motions. Certainly the used earthquake record (the 1978 Miyagi-Ken-Oki Earthquake) has the predominant period of about 1.0 sec so that the response would become large since the major periods are elongated by the brace rupture. As a conclusion, the rupture of bracing members will put a braced steel structure in jeopardy with high possibility.

CURRENT DESIGN REQUIREMENTS

Table 3 shows the international comparison about the width-thickness ratio limitations for box-section columns together with Kato's proposal for cold-formed box-section columns (Kato, 1987). In this table, the ductility required for members is classified into three grades in the Japanese codes. That is, the classes I, II, and III correspond to the ductility of η = 6, 1.5 and 0, respectively, where η is defined as $\eta = (\Delta u-\Delta y)/\Delta y$, and where Δy is the maximum flexural deformation limited by local buckling and Δu is the yield flexural deformation of members. Obviously, $\eta = 0$ means that local buckling occurs almost simultaneously with yielding. In addition the ductility in the classification adopted by the codes of ECCS (ECCS, 1984), New Zealand (NZ, 1985), AISC (AISC, 1978) and CSA (CSA, 1989) is not clear in the above definition. Therefore, the correspondence to the Japanese classes is only referential. Now, from Table 3 it can be seen that only the code of New Zealand satisfies Kato's proposal. Further, the values regulated in each code in this table are exactly the same as those on hot-formed or welded box sections (not indicated in Table 3). This implies that the peculiar properties of cold-formed members, which is discussed in the following section, are not reflected by the regulations cited in Table 3.

Consequently, the present code conditions concerning the cold-formed square or rectangular tubular members are inconsistent with the crucial results by recent investigations.

POSSIBLE MEASURES AND FUTURE RESEARCH NEEDS

In view of the circumstances mentioned above, structural designers should examine the following measures immediately:

1) Avoid using cold-formed square or rectangular tubing sections as bracing members, if other sections such as H-shaped sections are usable; or otherwise hot-formed or built up square (or rectangular) tubular sections may be adopted;
2) Use cold-formed sections with small width-thickness ratios (compact section);
3) Fill cold-formed tubing members with concrete to delay local buckling or to lessen its severity;
4) Anneal cold-formed members, before installing, to change somewhat mechanical properties of the members.

The item 1) is the easiest way, if it is allowed. By the past studies (6), H-shaped sections have even longer fracture lives than cold-formed square hollow sections.

Regarding the item 2) Tang and Goel (Tang, 1987) proposed a 50 percent reduction of the width-thickness ratio specified by the AISC Specifications (Part 2: Plastic Design) for compression members in order to minimize the effect of local buckling and delay the fracture. That is, the proposed value is (B-2t)/t=14. Also, Uang and Bertero (Uang. 1986) suggested a reduction in the B/t ratio to the value of 18, through their scale model test. Further, Kato commented in his paper (Kato, 1987) that the B/t ratio of cold-formed square tubular bracing members should be smaller than his proposal, B/t=20 for columns. Consequently, considering the above proposals and the past experimental results (Gugerli, 1982; Liu, 1987; Lee, 1987), the width-thickness ratio for cold-formed square or rectangular tubular members can be understood to be less than 16 for braces and 20 for columns in terms of B/t to preclude early fracture under severe inelastic cyclic displacements.

Relating to B/t ratios, recently in Japan, cold-formed square sections having thick wall-thickness up to 40mm by pressing have widely been used for columns in medium- and high-rise buildings. Although sufficient experimental data have not yet been attained, local buckling was considerably delayed in the case of $B/t \leq 20$ in a few tests which simulated realistic columns (Nakamura, 1990).

The effectiveness of the item 3) has been experimentally confirmed by Liu (Liu, 1987) and Lee (Lee, 1987). By their studies, the presence of concrete was not able to delay the occurrence of local buckling itself. However, it changed the local buckling mode and reduced the severity of local buckling. Then, the fracture life was much elongated owing to the delay of the crack initiation.

In a cold-formed square or rectangular tubing section, yield strength generally has a higher value than that of the pre-formed virgin steel sheet (Kato, 1988), as shown in Fig. 3. This is due to severe plastic cold-work in forming the section. In particular, the yield strength at the corner of the section is extremely heightened, whereas the tensile strength at the corner is less increased (Fig. 3). Thus, the yield ratio, defined as the ratio of yield strength to tensile strength, is very large at the corner resulting in poor strain capacity that leads to early cracking.

Now, under the above discussion, the measure of the item 4) can be conceived as an effective way to delay cracking. Theoretically, by annealing cold-formed members the high yield strength, high yield ratio and lack of deformability at corner portions may be lessen, since residual stresses or strains confined in sections can be released by annealing (Aoki, 1985). However, the effect of annealing on delaying the local buckling initiation cannot be expected (Aoki, 1985; Kato, 1980).

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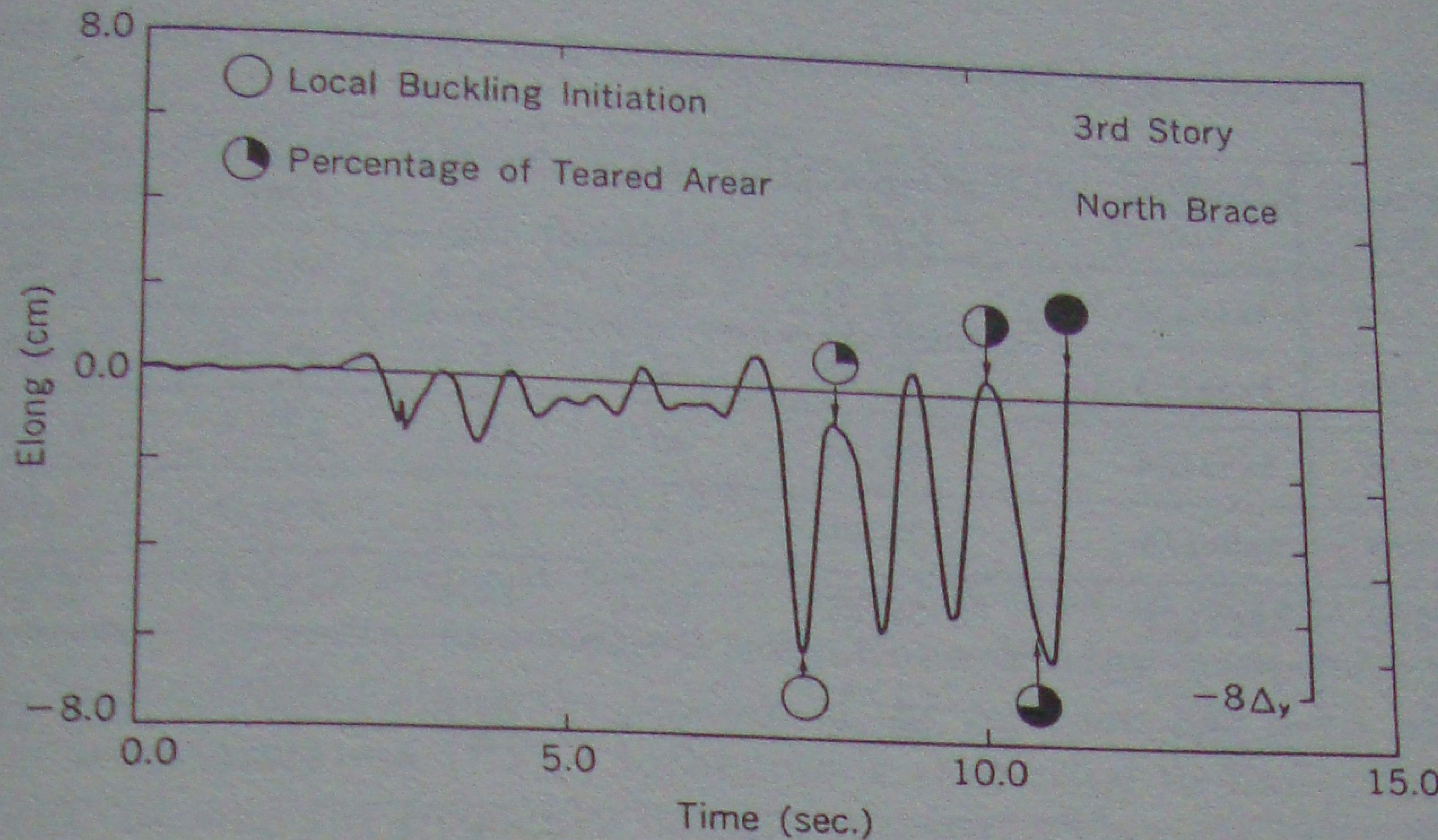

Fig. 1 Rapid Propagation of Crack after Local Buckling

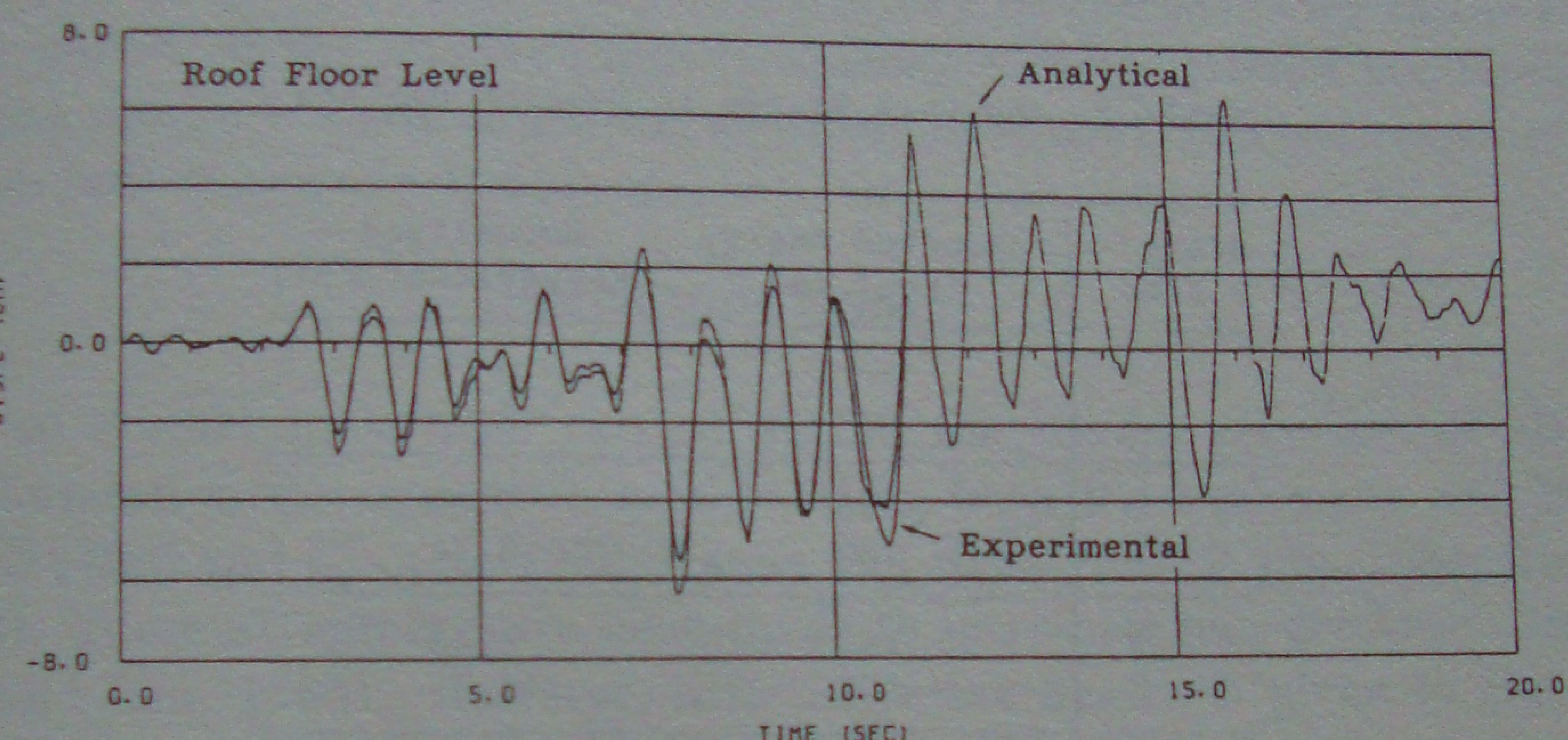

Fig. 2 Analytical Time History of Interstory Drift after Brace Rupture

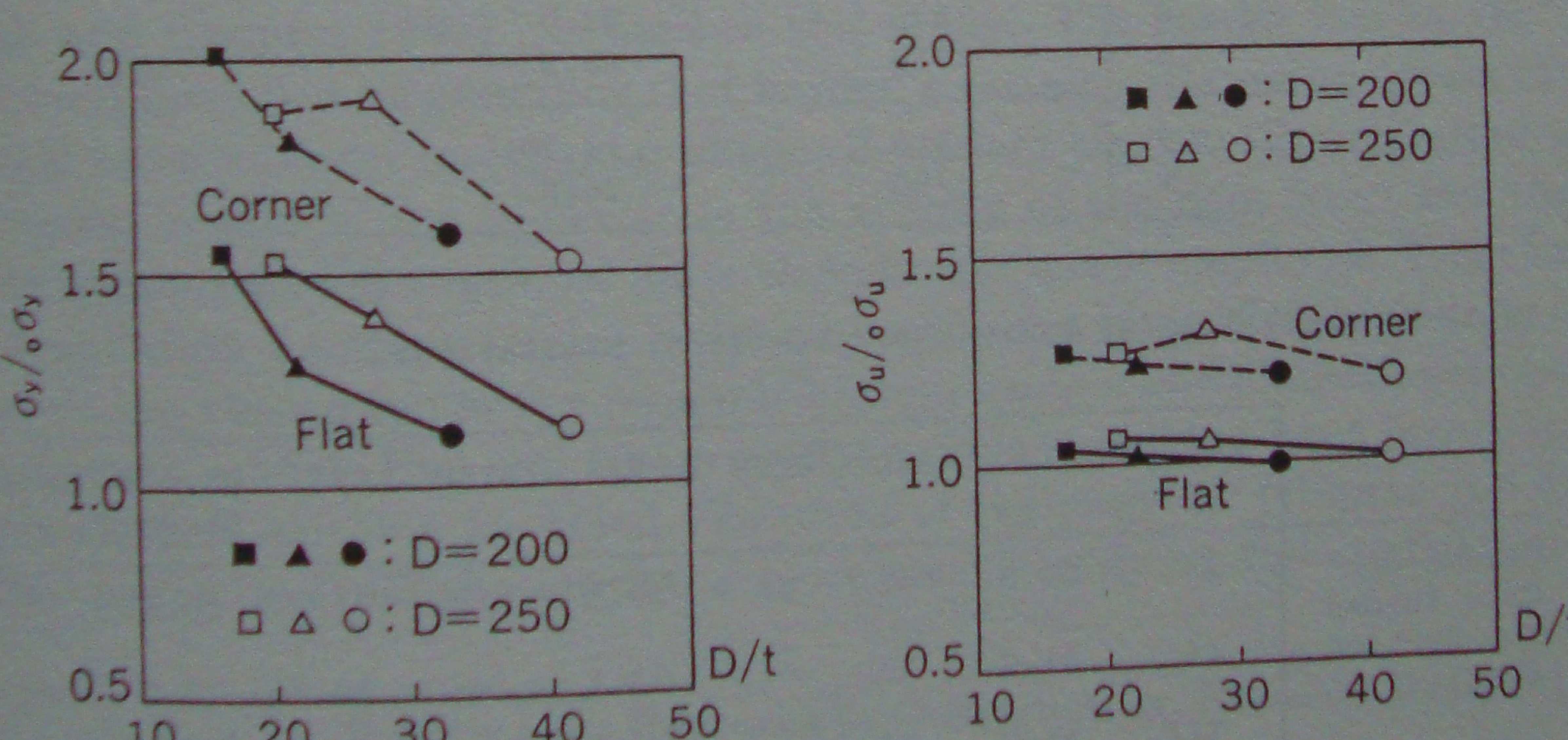

Fig. 3 Increase in Yield Stress and Maximum Stress of Cold-Formed Square Tube (Ratio to Virgin Steel Sheet)

Table 1 Geometry of Square Tubular Bracing Members Used for Phase I Test

Story & Side	Sizes (inch)	Sectional Area (cm^2)	B/t	(B/2t)/t	Length (cm)	Radius of Gyration(cm)
6 S/N	4x4x1/5.56	17.21	22.2	20.2	442.3	3.94
5 S/N	5x5x1/5.56	21.85	27.8	25.8	441.0	4.98
4 S/N	5x5x1/4	29.61	20.0	18.0	435.7	4.88
3 S/N	6x6x1/4	36.06	24.0	22.0	434.0	5.92
2 S/N	6x6x1/4	36.06	24.0	22.0	432.9	5.92
1 S	6x6x1/2	66.84	12.0	10.0	513.4	5.61
1 N					512.4	

Note: (1) B=width of section, and t=wall thickness
(2) Brace length designates face-to-face length, not based on center-line dimension

Table 2 Absorbed Energy in Bracing Members

Story & Side	Absorbed Energy (tonf cm)	Normalized Energy(tonf/cm^2)
6 S	—	—
N	—	—
5 S	70	3.20
N	180	8.24
5 S	143	4.83
N	500	16.9
3 S	393	10.9
N	1321	36.6
2 S	1875	52.0
N	1750	48.5
1 S	214	3.20
N	1790	26.8

Table 3 Comparison of Width-Thickness (B/t) Limitations

	Class	AIJ	BSL	ECCS	NZ	AISC	CSA	Kato's Study
Column of Cold Formed Box Section (SHS)	I	27.0	27	28	19.5	30.0	27.4	20.0
	II	31.0	32	—	23.4	—	34.2	29.6
	III	41.0	41	—	31.1	37.1	43.7	36.0