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Web Slenderness Limits for Compact Beams

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A B S T R A C T

Present specifications do not distinguish between compact sections and those sections suitable for plastic design with respect to the web slenderness ratio. Because compact sections have less severe deformation requirements than plastic design sections, the result is a web slenderness ratio limit that is conservative for compact sections.

This report is the result of an investigation to determine a more suitable limit for the web depth-to-thickness ratio of compact beams. The study shows that present limits prescribed by the Canadian Standards Association Standard S16-1969 can safely be raised approximately twenty-five percent.

A C K N O W L E D G E M E N T S

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The authors wish to express their thanks also to the staff of the Department of Civil Engineering for their assistance during the testing program.

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1. INTRODUCTION

Present specifications for the design of structural steel beams[6] classify a particular member as being non-compact, compact, or suitable for plastic design. The feature that determines the classification of a member is the behavior of that member at ultimate load. Non-compact members are those members that are able to just reach the yield moment before any local buckling of the compression elements result in a loss of strength. Compact members must be able to reach their plastic moment capacity. Members suitable for plastic design must be able to reach their plastic moment and, in addition, must achieve considerable inelastic rotation before the onset of local buckling. The three classes of behavior are illustrated by means of moment-rotation curves in Fig. 1.1.

The buckling strength of an axially loaded plate is largely an inverse function of its width to thickness ratio. Because the compression elements of a beam are essentially axially loaded plates, this property can be used to determine if a member is compact or non-compact. As a member becomes "more compact", its deformation requirements become greater and the compression elements must become more stocky in order that they be resistant to local buckling. In addition, as the yield strength of the material increases, local buckling must occur at a higher stress in order that the member maintains the same type of behavior. Thus, members of higher strength material require stockier sections than those of lower strength in order to qualify as the same member type.

Current CSA S-16[6] width to thickness ratio limits for the three types of members are summarized in Fig. 1.2.

Because of their higher ultimate strength, compact beams have a correspondingly higher allowable stress than non-compact sections. The difference is about ten percent. For the economical design of built-up beams, the web will usually be so slender that the section will not qualify as compact. This imposes an immediate ten percent penalty on allowable stresses and reduces the economies to be gained from this type of construction.

Current specifications provide for only one web slenderness limit with respect to classifying a section as compact or suitable for plastic design. (No limit at all is specified for non-compact sections). Because this limit specifies sections that are suitable for plastic design, there is essentially no compact classification with respect to web slenderness ratio. That is, a section is either non-compact, or it is suitable for plastic design. As the deformation requirements of a compact section are less severe than those of a plastic design section, this situation seems unnecessarily conservative.

A limited amount of recent research has suggested that the limit for the web slenderness ratio for compact beams is indeed conservative.[4] The report presented herein is the result of an experimental investigation to study the effect of slender webs on the moment capacity of unstiffened beam sections.

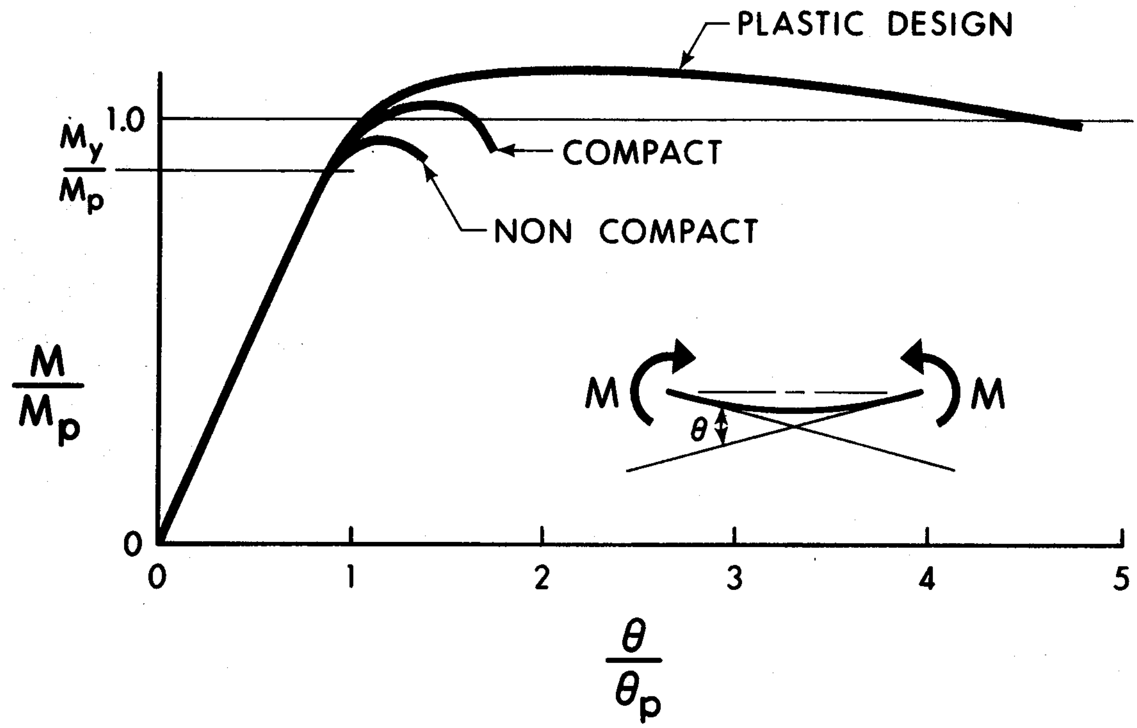
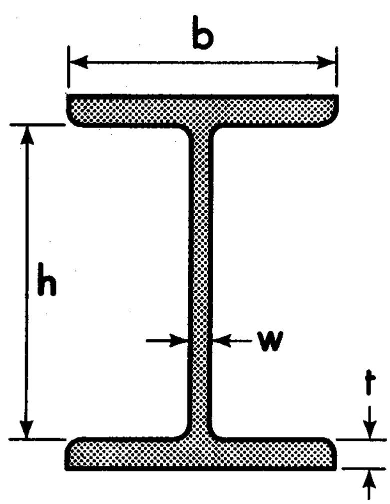


FIG. 1.1 TYPES OF MOMENT ROTATION BEHAVIOR



	Flange	Web
Non-Compact	$\frac{b}{2t} \leq \frac{100}{\sqrt{F_y}}$	Not specified
Compact	$\frac{b}{2t} \leq \frac{64}{\sqrt{F_y}}$	$\frac{h}{w} \leq \frac{420}{\sqrt{F_y}}$
Plastic Design	$\frac{b}{2t} \leq \frac{54}{\sqrt{F_y}}$	$\frac{h}{w} \leq \frac{420}{\sqrt{F_y}}$

FIG. 1.2 PRESENT CSA S16 WIDTH-THICKNESS LIMITS

2. PREVIOUS INVESTIGATIONS

Comparatively little research has been conducted on unstiffened beams with web slenderness ratios in the intermediate range ($80 < h/w < 120$). The first significant study on unstiffened plate girders was conducted by Lyse and Godfrey at Lehigh University in 1935.[10] Test results of ten rolled or welded steel sections with web slenderness ratios between 50 and 70 were presented. All had large deformation capacities and carried shear stresses in excess of shear yield. The conclusion reached was that, starting at a web slenderness ratio of 80, local web buckling would prevent the beam from yielding in shear. As the steel used in these tests had a specified yield stress of 33 ksi, present specifications[6] would limit the web slenderness ratio for compact sections to a value of 73.

In 1960, an extensive investigation was reported by Haaijer and Thurlimann.[8] From a theoretical study supported by test results, they specified geometric properties of sections able to undergo large plastic deformations without local buckling. They stated that, except for cases requiring very large deformations, a slenderness limit of

$$\frac{d}{w} \leq \frac{370}{\sqrt{F_y}}$$

would prevent premature local buckling of the web. (In this expression, d is the depth between flange centroids, w is the web thickness, and

F_y is the minimum specified yield stress in ksi).

In 1961 an extensive study of plate girders was conducted at Lehigh University by Basler.[1] Among the tests were two on beams having web slenderness ratios of 99 and 131 and aspect (stiffener spacing to web depth) ratios of 3. From these it was concluded that the general plate buckling formula assuming simple supports gave a conservative estimate of the shear strength of plate girder webs, and it was suggested that the limiting web slenderness ratio for unstiffened girders might be raised to 100. As the steel used had a yield of 33 ksi, this is equivalent to a ratio of $574/\sqrt{F_y}$.

In 1968, Carskaddan[3] reported tests on unstiffened hybrid beams having web slenderness ratios of from 70 to 145. He concluded that, in order for a beam with a web yield stress of 36 ksi to reach shear yield before local buckling, the web slenderness ratio must be less than 67. This is equivalent to $h/w \leq 402/\sqrt{F_y}$.

Recent research at the University of Texas[4] on unstiffened continuous plate girders suggests that plastic design can be allowed for beams with web slenderness ratios as high as $750/\sqrt{F_y}$, providing that the shear stress is lower than the critical web buckling stress calculated from elastic theory.

3. EXPERIMENTAL PROGRAM

3.1 Scope

A total of ten beams were tested during this investigation. Eight were designed so that the flange size was the same throughout and just met the slenderness limit for compact beams given by CSA S16 (see Fig. 1.2). A limit for the web slenderness ratio of compact sections was to be chosen from the results of these tests. The two remaining specimens were to provide data so that the web slenderness ratio limit for sections suitable for plastic design could be checked. Accordingly, the flanges of these two beams were slightly stockier, meeting the requirements for plastic design given by CSA S16. The web slenderness ratios varied among the specimens, exceeding the compact limit by amounts ranging from 18 to 114 percent.

All beams were simply supported and were subjected to a symmetric two-point loading. To preclude premature lateral buckling, the compression flange of each specimen was laterally braced, with the bracing spacing meeting the requirements for plastic design.

The loading arrangement required that stiffeners be placed on the web at the load and reaction points. In the region of high moment, these stiffeners extended less than half the depth of the beam and were placed only on the tension side of the web. It was felt that the effect of the stiffeners on local buckling of the web would be very small. Thus, all beams were considered to be unstiffened.

3.2 Specimen Description

The beams were fabricated from CSA G40.12 plate[5], which has a specified minimum yield point of 44 ksi for plates up to 1 1/2 in. thick. The flanges of eight of the specimens were 7 1/4 in. by 3/8 in., resulting in a width to thickness ratio of 9.66. This just meets the compact limit of 9.65 as specified by CSA S16. The flanges of specimens WS-7-P and WS-8-P were 6 in. by 3/8 in. The resulting width to thickness ratio is 8.00. The limit for plastic design according to CSA S16 is 8.15. The webs of all specimens were fabricated from 1/4 in. plate.

For material meeting CSA G40.12, the present web slenderness limit for compact sections is 63. The initial series of 5 tests were to be conducted on specimens with nominal web slenderness ratios of 80, 100, 120, 140 and 160. After testing the first four specimens, it became apparent that specimen WS-5, which had a nominal slenderness ratio of 160, would not reach the plastic moment. As it was only necessary to find the slenderness ratio at which a beam would just reach the plastic moment, specimen WS-5 was not tested.

From the results of the first four tests, a web slenderness ratio was chosen that would apparently allow a beam to just reach the plastic moment prior to local buckling. A sixth specimen was then fabricated with that web slenderness ratio. At the same time, two specimens were designed having flanges suitable for plastic design. These were fabricated of material from the same rolling as the other specimens to ensure that the material properties were the same throughout the test series. One of the plastic design sections had a web slenderness equal to that chosen for the confirming test on compact sections while the other had a slightly stockier web. It was felt that a limiting web slenderness ratio for plastic design sections could be chosen from the results of these two tests.

Upon completion of this portion of the testing program, the untested specimen WS-5 was modified to make specimens 9, 10 and 11 in order to increase the amount of data available. These beams also had web slenderness ratios close to what was felt would be a reasonable limit for compact design.

The beams were proportioned so that the shear stress developed when the beam reached the plastic moment would be less than the inelastic buckling shear stress. The ratio of maximum shear stress to the buckling shear stress varied significantly throughout the test series. Each specimen was simply supported and was subjected to a symmetric, two-point loading. This resulted in high shear with varying moment in the end panels and a short constant moment region in the center of the beam. The length of the constant moment region varied from specimen to specimen. The loading and stiffener arrangements are shown in inset of Table 3.1.

As noted, the compression flanges were laterally braced to meet the requirements of plastic design. A threaded rod was welded to the flange (in the plane of the web) at each bracing location and a bracing arrangement based on Watt's straightline mechanism[11] was attached. The arrangement effectively prevented any lateral movement of the compression flange but did not restrict vertical deflections in the plane of the loading. The lateral braces did not restrict torsional rotations or local buckling of flange and web. Bracing was always provided at the load points and occasionally, as required, in the region between the load points and reaction points. At each reaction point, the tension flange was laterally restrained by threaded rods attached to the stiffeners and to the testing frame. It was felt that the reaction hangers, because they were stressed in tension, would provide some measure of lateral stability of the compression flange near the ends of the beam. Similarly, because the loading jacks were in tension and because they were attached to the beam very close to the tension flange, lateral stability was inherent in the tension flange in the central portion of the beam. Thus, each specimen was also torsionally restrained to some degree.

Table 3.1 gives the essential details of each test specimen.

3.3 Test Set-up and Procedure

Load was applied by means of two hydraulic rams acting in tension and suspended from a testing frame. The jacks were pin-connected to brackets which were in turn bolted to partial depth stiffeners on the web of the beam being tested. The beam reactions were taken by hanger rods passing through the floor of the laboratory. Connection to the beam was by means of a pin through brackets bolted to full depth stiffeners at the beam ends. The overall test set-up is shown in Fig. 3.1.

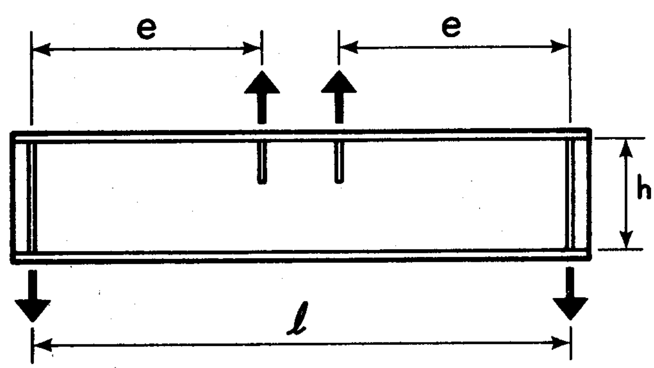
The instrumentation used is shown schematically in Fig. 3.2. Load was measured by electric resistance strain gages on load cells attached to each jack. After an initial set of readings was taken, load was applied slowly, in increments of approximately one fifteenth of the expected maximum load. At each increment, the load was held constant and all deformations were allowed to stop before a set of readings was taken.

Rotations were measured by means of mechanical rotation meters attached to the stiffeners at the reaction points and at the load points. Vertical deflections relative to the laboratory floor were measured by means of 0.001 in. dial gages mounted on pedestals on the floor. Deflections were measured at the reaction and load points. This was done so that the deflections of the load points relative to the chord joining the reaction points could be calculated.

The strain distribution due to bending over the cross-section at the centerline was measured by means of SR-4 electric resistance strain gages. Depending on the depth of the beam, 12 or 16 gages were used. Four of these were placed on the inside surfaces of the flanges, the remaining gages being placed on the web. At each location on the cross-section, one strain gage was mounted on each side of the web. Thus, there were two gages to measure strain at any particular depth. This configuration allowed a check of any local buckling of the web by comparing the measured strain from a pair of gages. Out-of-plane curvature of the web would cause the gage on the concave side of the web to have a higher compressive strain than the gage on the convex side.

Out-of-plane deflections of the web were measured by means of an apparatus consisting of three (or five, depending on the web depth) dial gages mounted on a light but rigid frame. The frame consisted of two legs, the tips of which were placed on the surface to be measured, and a mounting bracket that held the dial gages rigid with respect to the legs. The dial gages were mounted so that they could measure deflections relative to a chord joining the tips of the legs. It was necessary to first calibrate the apparatus by placing it on a surface known to be flat and comparing all succeeding readings with gage readings so obtained. During the test, the device was used to measure out-of-plane deflections of the web along vertical lines at several locations along the length of the beam. The readings were taken at approximately every second load increment. The locations measured were generally at the center of each end panel, approximately six inches each side of the load points, and about six inches each side of the centerline. The locations varied slightly from test to test as experience was gained in predicting the location of local buckling. Using this scheme, web deflection readings were taken at locations of high shear and low moment, high shear and high moment, and high moment with no shear.

TABLE 3.1 DETAILS OF TEST SPECIMENS



Specimen	Web Depth h (in.)	h/w	Span ℓ (in.)	Distance e (in.)
WS-1	20.00	76.3	192.0	72.0
WS-2	24.97	95.7	192.1	72.0
WS-3	29.97	115.3	240.0	99.0
WS-4	35.00	137.8	287.7	125.8
WS-6	23.47	91.3	192.1	72.0
WS-7-P	21.00	81.7	167.9	65.9
WS-8-P	23.50	90.4	168.0	66.0
WS-9	19.41	74.7	120.0	42.0
WS-10	20.94	81.5	119.9	43.9
WS-11	22.97	89.0	119.8	47.9

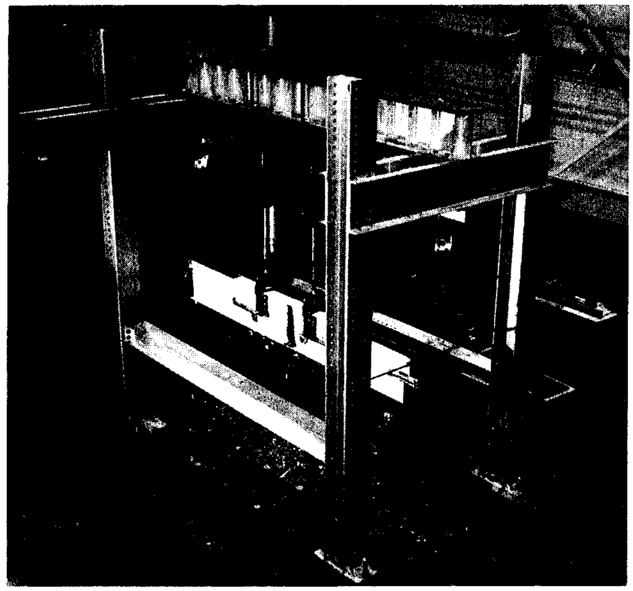


FIG. 3.1 TEST SETUP

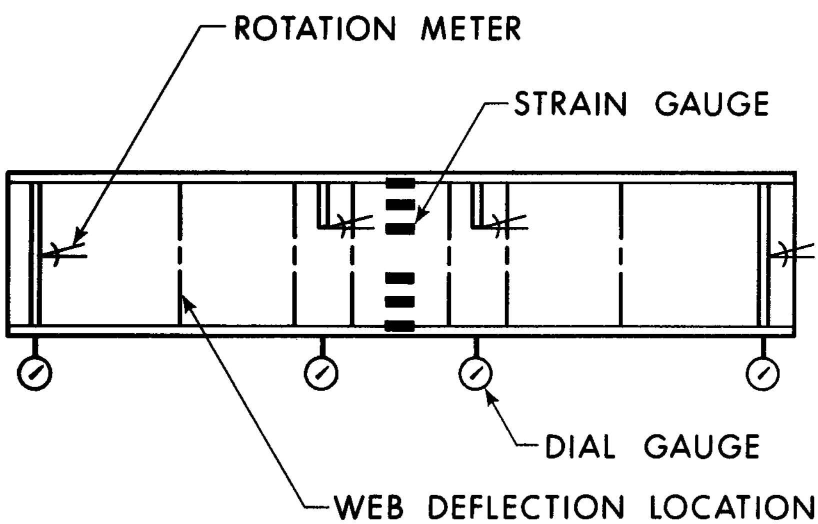


FIG. 3.2 INSTRUMENTATION

4. TEST RESULTS

4.1 General Description

All tests were similar in nature and only one will be described here in detail. Significant differences, if any, will be noted for the other tests.

Specimen WS-6 was chosen to provide confirmation of the results of the tests on specimens WS-1 to WS-4. By plotting the values of M_u/M_p versus h/w for specimens WS-1 to WS-4 as is done in Fig. 4.1, a value of $h/w \leq 590/\sqrt{F_y}$ was chosen as a possible limit on web slenderness ratio for compact sections. It was felt that a beam with this web slenderness would just reach the plastic moment before local buckling of the web occurred. Accordingly, specimen WS-6 was designed with this slenderness ratio.

As the specimen was loaded in increments of approximately one fifteenth of the expected ultimate load, careful observations were made at each increment to record any noticeable changes of the cross section. In order to control the rate of loading, the vertical deflection under one load point was plotted against each increment of load. Up to and including load increment 8 ($0.51\ M_p$), measured deflections were linear and were very close to the theoretical values. At increment 9, it was noticed that the rate of deflection was increasing and, consequently, deflections were used to control the loading rate for the remainder of the test. Up to and including increment 10, no change was observed in the shape of the cross section.

At increment 11 (0.70 M_p), a lateral movement of the compression flange was observed. This occurred in the constant moment region where the unbraced length of the compression flange was 48 inches. For this particular section, the maximum allowable unbraced length for plastic design is slightly less than this, a value of 45 inches.[6] However, as the large deformations assumed in plastic design were not present during this test, the bracing should have been adequate to prevent premature lateral buckling.

At increment 13 (0.82 M_p), a local web buckle first became noticeable. This occurred in the constant moment region, approximately six inches toward the center of the beam from one of the load points. The location was close enough to the load point that there may have been some local effects due to shear, but the major cause of the buckle was apparently due to the flexural compressive stresses. Simultaneously with observation of the web buckle, a local flange buckle was noticed forming directly adjacent. Other observations, such as the measurement of out-of-plane deflection of the web discussed in the following section, show that it is probable that the web was the first element to buckle. This would force the flange to carry additional stresses while reducing torsional restraint of the flange, leading to the formation of a local flange buckle.

After increment 15 (0.88 M_p), local buckling of web and flange became much more severe, reducing the moment capacity and leading to a rapid decrease in load for small increases in deflection. After increment 20, the load had dropped to about 85% of its maximum value and the test was stopped.

4.2 Results and Discussion

As all specimens were fabricated of material from the same rolling, the yield strength of the material was constant throughout the test series. From a series of standard coupon tests, it was determined that the yield strength of the flanges of the specimens was 46.8 ksi and that of the webs was 41.7 ksi.

The results of the tests on the ten specimens are given in Table 4.1. Table 4.2 shows the out-of-plane deflections present in the web before testing.

The test results are also plotted in the form of maximum strength as a function of web slenderness ratio in Fig. 4.1 and also in the form of a moment-shear interaction diagram in Fig. 4.2.

Figure 4.3 shows the moment versus rotation behavior for the ten beams tested. The rotations plotted are the relative rotations between the load points as shown in the inset in Fig. 4.3(a). The moments and rotations have been non-dimensionalized in order that comparisons between specimens may be made.

Fig. 4.4 is a plot of the maximum out-of-plane deflections of the web versus the vertical deflection of the load point. Vertical deflection was chosen as the ordinate rather than load because, as yielding progresses, load increases at a slower rate. This would appear as an increasing rate of web deflection, even if the rate did remain relatively constant. Using vertical deflection as the ordinate allows the point of buckling of the web to be determined with greater accuracy. The ordinate was non-dimensionalized by dividing by the plastic deflection in order that comparisons may be made between specimens. Also shown in the inset in each plot are the locations at which the web deflections were measured.

The web deflection for specimen WS-6 is plotted in Fig. 4.4(e). From the curve for location 3, it can be seen that the rate of web deflection began to increase at increment 12. As mentioned previously, the first visual indication of a web buckle occurred at load increment 13. The measured deflections for curve 3 in Fig. 4.4(e) were taken at a location approximately 3 inches from the point where the maximum buckle formed. The buckle extended close to where the measurements were taken. The plots of web deflections in Fig. 4.4 are a good indication of the nature of failure, that is, the curves (in conjunction with visual observations) indicate whether the failure was induced by a local flange buckle or a local web buckle. If the curves show no significant changes, such as those for specimen WS-1 (Fig 4.4 (a), then local flange buckling was the mode of failure. Conversely, if a curve shows a steadily increasing rate of deflection, as in Fig. 4.4(a), failure was induced by a local web buckle.

From specimen WS-1 to WS-4, the web slenderness ratio increased significantly and the following observations can be made. Specimen WS-1, $(h/w)\sqrt{F_y} = 493$, was observed to fail due to a local flange buckle. Fig. 4.4(a) shows that very little web deflection occurred. Specimen WS-2, $h/w\sqrt{F_y} = 618$, also failed by a local flange buckle. The corresponding figure shows slightly higher web deflections. However, no significant increase in the rate of deflection occurred until the maximum moment had been reached. Specimen WS-3, $h/w\sqrt{F_y} = 744$, failed when a local web buckle formed at the centerline of the beam. Unfortunately, because of the strain gages mounted on the centerline, no web deflection measurements were taken at exactly this location. Fig. 4.4(c) shows little deflection at the nearby locations, however. In contrast to the preceding two cases, the local buckle formed in a region of pure moment. As shown by the rotation curve in Fig. 4.3(c), Specimen WS-3 definitely behaved as a non-compact section, reaching only 89 percent of M_p before failure. The behavior of specimen WS-4, $h/w\sqrt{F_y} = 890$, was similar to but more pronounced than that of specimen WS-3. Fig. 4.4(d) shows that large web deflections occurred.

Specimens WS-7-P and WS-8-P had flanges that were sufficiently stocky to qualify for plastic design and webs that exceeded the present compact limit. From the results of tests WS-1 to WS-4, it was felt that these two specimens would behave as compact sections. It was hoped that the specimen with the stockier web (WS-7-P) would also have sufficient rotation capacity to qualify for use in plastic design, thus providing data on a web slenderness ratio for plastic design sections.

Specimen WS-7-P, $\frac{h}{w}\sqrt{F_y} = 528$, failed when a local flange buckle formed at a moment corresponding to 0.99 M_p. Fig. 4.4(f) shows that, although total deflections were relatively small, an initially flat section began to deflect at load increment 16 (0.92 M_p). Although the web buckle was not observed visually, it could have been large enough to force the flange buckle.

Specimen WS-8-P, $\frac{h}{w}\sqrt{F_y} = 584$, failed at a load of 0.95 M_p by the formation of a web buckle at a location of high shear and moment. Fig. 4.4(g) shows that the rate of web deflection increased very rapidly at increment 12 (0.86 M_p).

Because specimen WS-7-P had an ultimate moment that was less than the plastic moment by only a very small amount, its behavior can be considered compact. Specimen WS-8-P did not behave as a compact section, however.

Specimens WS-9, WS-10 and WS-11 all had webs that were non-compact by present standards, yet all behaved as compact sections, with strengths significantly exceeding M_p. Figs. 4.4(h) to 4.4(j) show that fairly large web deflections occurred. However, the largest deflections always occurred in a region of high shear and low moment. In addition, the deflections occurred at a fairly constant rate. Shear stresses were relatively much higher in these three specimens than in the preceding seven, causing shear induced deflections to predominate over web deflections due to compressive bending stresses. Each of these three specimens was observed to fail by local flange buckling. This observation is supported by the data in Figs. 4.4(h) to 4.4(j).

4.3 Effect of Initial Web Deflections

The webs of all beams had varying amounts of initial out-of-flatness, probably the result of residual stresses induced by welding the relatively light web plate. The initial deflections can be seen in Fig. 4.4 as the web deflection at a vertical deflection of zero, and they are also tabulated in Table 4.2. In the table, maximum initial measured deflection is divided by the depth of the web over which out-of-plane deformations were measured.

Using Table 4.2 in conjunction with the gross test results presented in Table 4.1, it can be seen that, with one exception, all specimens which failed to reach the plastic moment had higher initial deflections than those that did reach M_p. The exception is specimen WS-7-P, and its maximum moment was only 1 percent below M_p, which is within the limits of experimental error. Additionally, it can be noted that all specimens that reached M_p failed due to a local flange buckle while those that did not failed due to a local web buckle.

It should also be noted that only one specimen (WS-4) exceeded the out-of-flatness limit of $\delta_{max}/d \leq 0.00667$ given by CSA Standard W59.1.[7]

From Fig. 4.4, it can be seen that, as expected, webs initially not flat began deflecting almost immediately upon application of load. This, of course, leads to earlier buckling and loss of strength.

4.4 Effect of Shear Stress

The specimens tested had a wide range of shear stresses at maximum loads. This is shown in the interaction diagram of Fig. 4.2.

The critical web buckling shear, assuming elastic behavior, can be calculated from the general plate buckling formula;

$$v_{cr} = \frac{k\pi^2 E}{12(1-\mu^2)(h/w)^2} \tag{4.1}$$

and

$$V_{cr} = v_{cr}\, h\, w \tag{4.2}$$

where

v_{cr} - shear stress causing buckling of web

V_{cr} - shear force causing buckling

h - web depth

w - web thickness

k - constant depending on edge support conditions

= 5.34 (assuming pinned boundary)

E - Young's modulus

μ - Poisson's ratio

Basler[1] gives the following correction for inelastic buckling to be used when Eqn. 4.1 gives a value of $v_{cr} \leq 0.8\ v_y$;

$$v'_{cr} = v_y \left(1 - \frac{0.16 v_y}{v_{cr}}\right) \tag{4.3}$$

where v_y = shear yield stress

$$= \frac{F_y}{\sqrt{3}}$$

Fig. 4.2 shows that, for this series of tests, the magnitude of the maximum shear stress had little or no effect upon the moment capacity of the beams. Specimens 9, 10 and 11 had very high shear stresses, yet were able to significantly exceed the plastic moment.

Specimens 3, 4, 6 and 8-P had higher initial web deflections ($\delta/d \geq 0.00449$) and Fig. 4.2 shows that the behavior of these beams was not good. Specimens 1, 2, 7-P, 9, 10 and 11 had lower initial deflections ($\delta/d \leq 0.00319$) and their behavior was satisfactory.

This comparison is very broad and a more meaningful one would compare members with similar web slenderness ratios. For example, specimens 1 and 9 had comparable slenderness ratios (76.3 vs. 74.7) and comparable web deflections (δ/d = 0.0030 vs. 0.0027). The relative shear was over 60 percent greater in Specimen 9 than in Specimen 1 yet Specimen 9 achieved a 5 percent higher moment than did Specimen 1.

Specimens 6, 8-P and 11 had similar web slenderness ratios (91.3, 90.4, 89.0). Specimens 6 and 8-P in addition had comparable shears and web deflections. Specimen 8-P reached a higher moment than did 6, probably the result of the stockier flange of Specimen 8-P. Specimen 11 had almost double the shear of 6 and 8-P yet carried a substantially higher moment. It should also be noted that the web deflection of Specimen 11 was approximately half that of Specimens 6 and 8-P.

While it is impossible to completely separate the different effects on the moment capacity, it is valid to state that shear had little effect. Initial web deflections had a much larger effect, and it was large enough to partially obscure the effect of web slenderness on the moment capacity.

TABLE 4.1 TEST RESULTS

Specimen	$h/w\sqrt{F_y}$	M_u (in-k)	$\frac{M_u}{M_p}$	V_u (kips)	$\frac{V_u}{V_{cr}}$	Buckled Element
WS-1	493	3840	1.023	53.4	0.499	Flange
WS-2	618	4910	0.977	68.2	0.661	Flange
WS-3	744	5740	0.893	58.0	0.681	Web
WS-4	890	6820	0.863	54.2	0.796	Web
WS-6	590	3940	0.879	54.7	0.521	Web
WS-7-P	528	3380	0.993	51.3	0.480	Flange
WS-8-P	584	3770	0.954	57.1	0.529	Web
WS-9	482	3910	1.077	93.1	0.904	Flange
WS-10	526	4580	1.147	104	0.976	Flange
WS-11	575	4780	1.066	99.7	0.968	Flange

TABLE 4.2 INITIAL WEB DEFLECTIONS

Specimen	Depth, d inch	Deflection δ inch	$\frac{\delta}{d}$
WS-1	19.0	.057	.00300
WS-2	23.5	.075	.00319
WS-3	28.5	.128	.00449
WS-4	33.5	.496	.01485*
WS-6	22.0	.103	.00469
WS-7-P	20.0	.025	.00125
WS-8-P	22.0	.108	.00491
WS-9	19.0	.052	.00274
WS-10	19.5	.050	.00256
WS-11	22.0	.057	.00259

*Exceeds CSA W59.1 limit of $\frac{\delta}{d} \leq .00667$

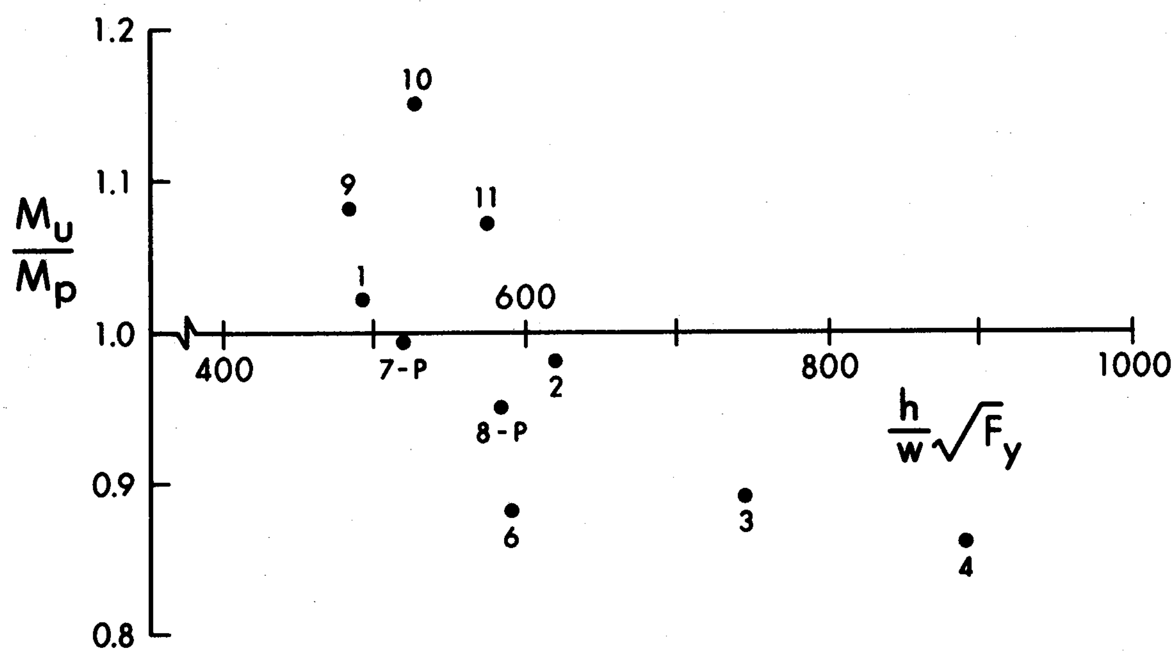


FIG. 4.1 EFFECT OF WEB SLENDERNESS ON MOMENT CAPACITY

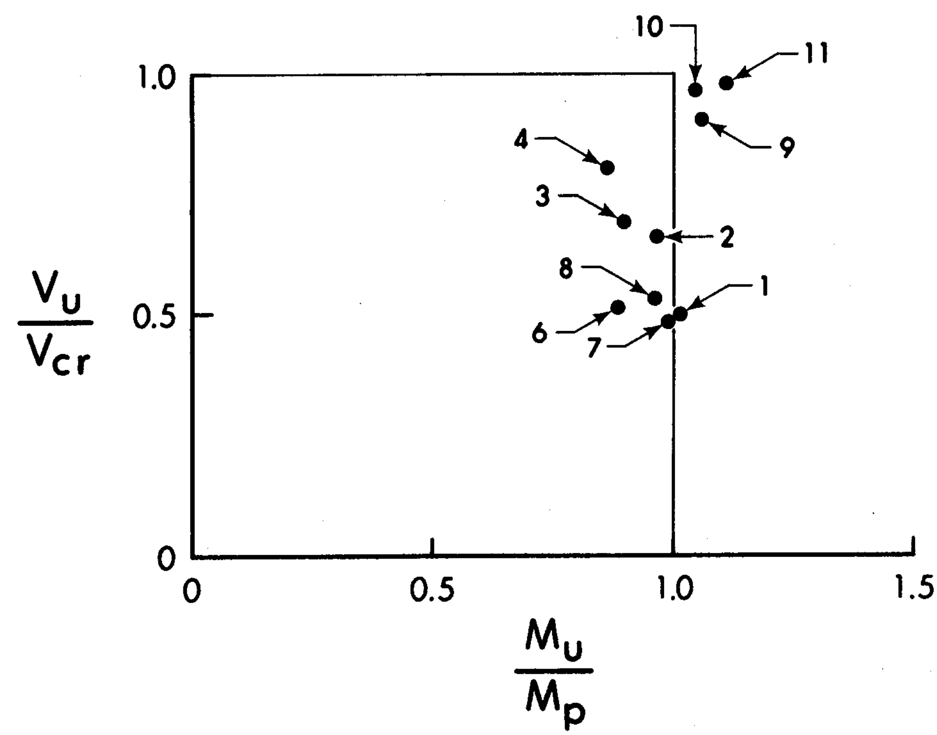
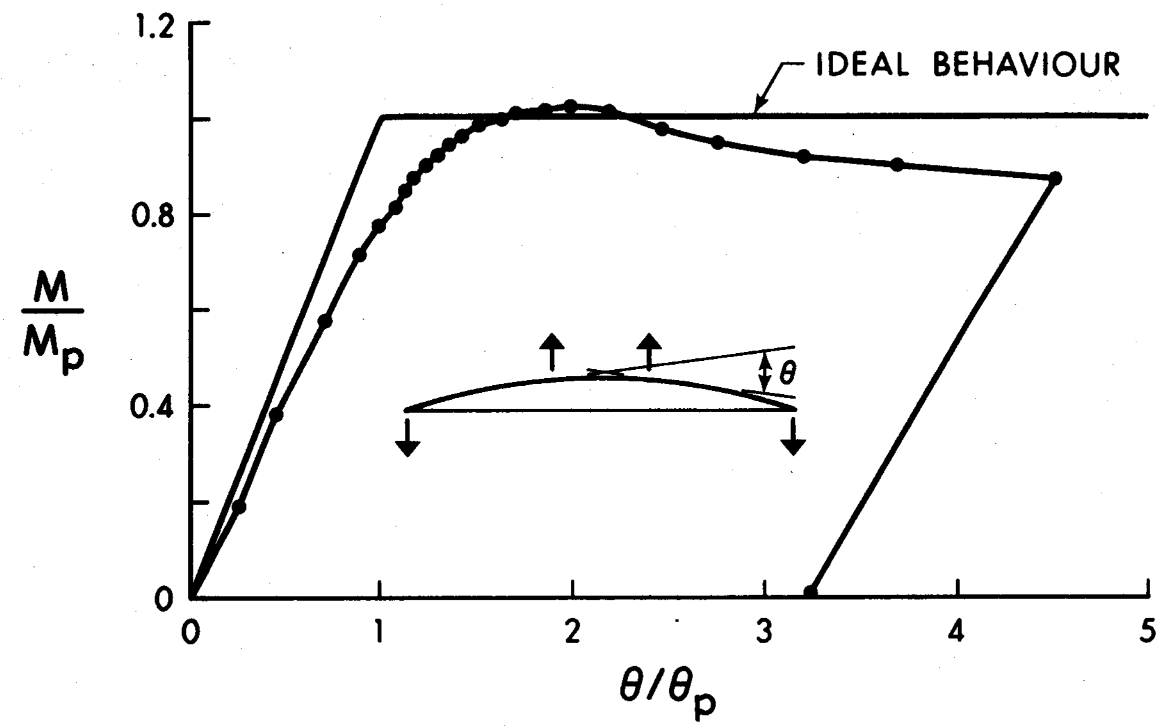
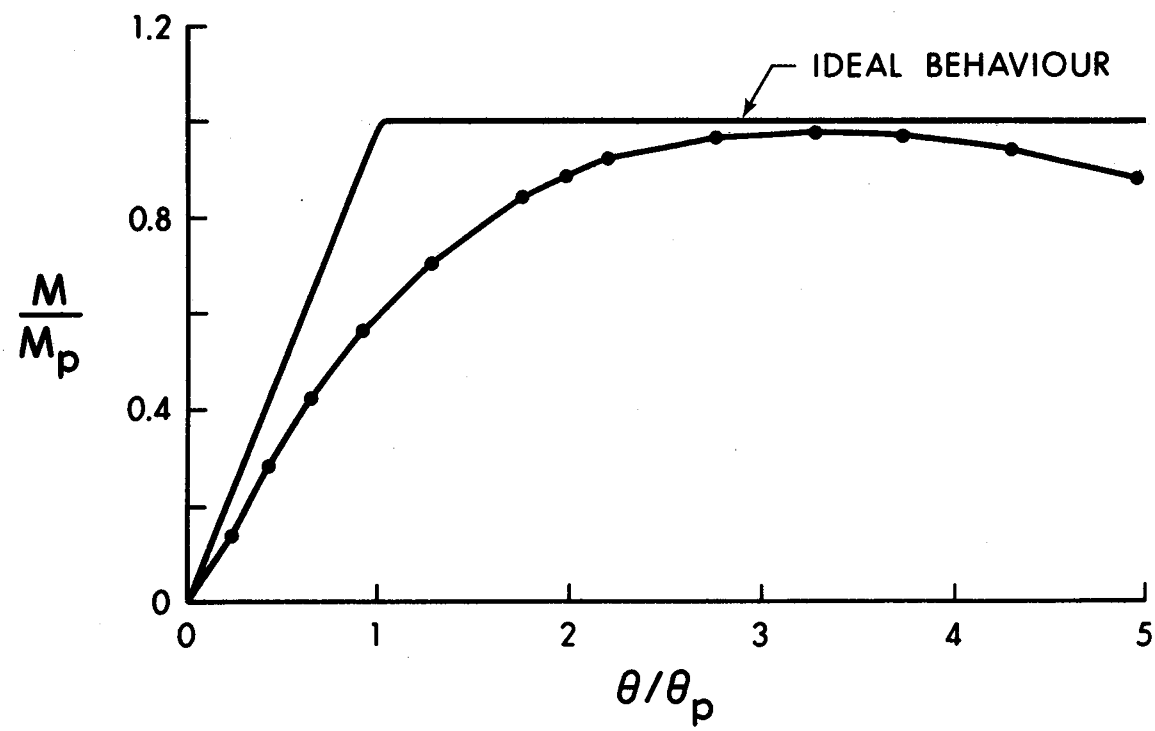


FIG. 4.2 INTERACTION OF SHEAR AND MOMENT

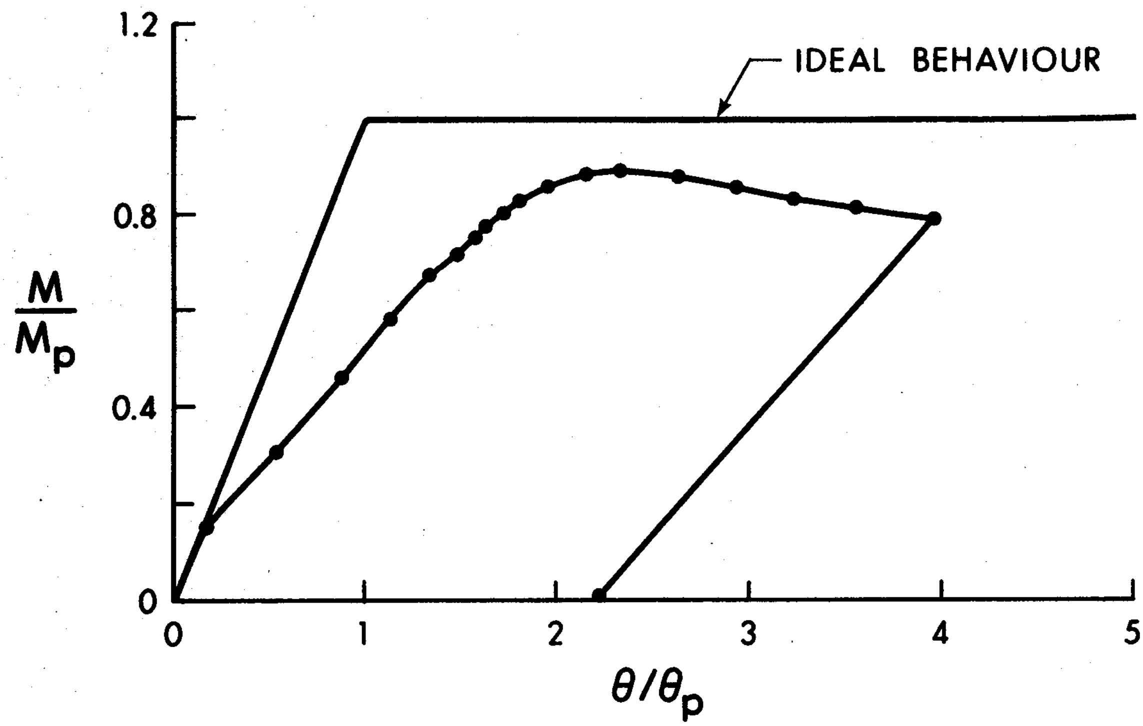


(a) Specimen WS-1

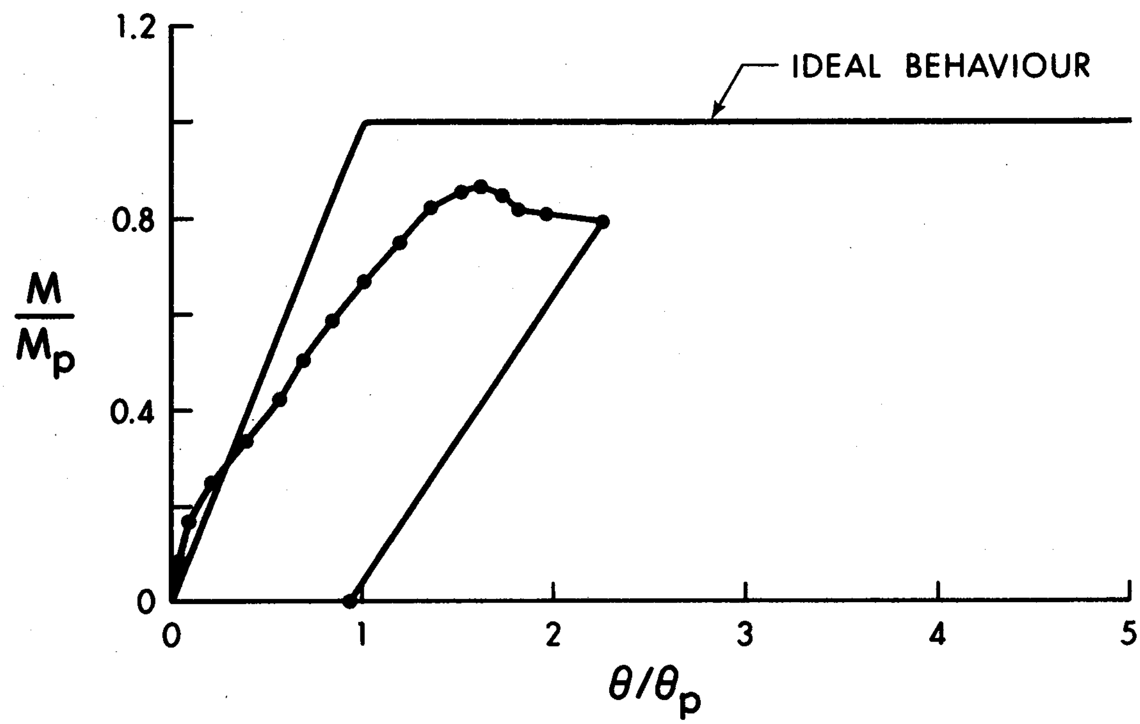


(b) Specimen WS-1

FIG. 4.3 MOMENT ROTATION BEHAVIOR

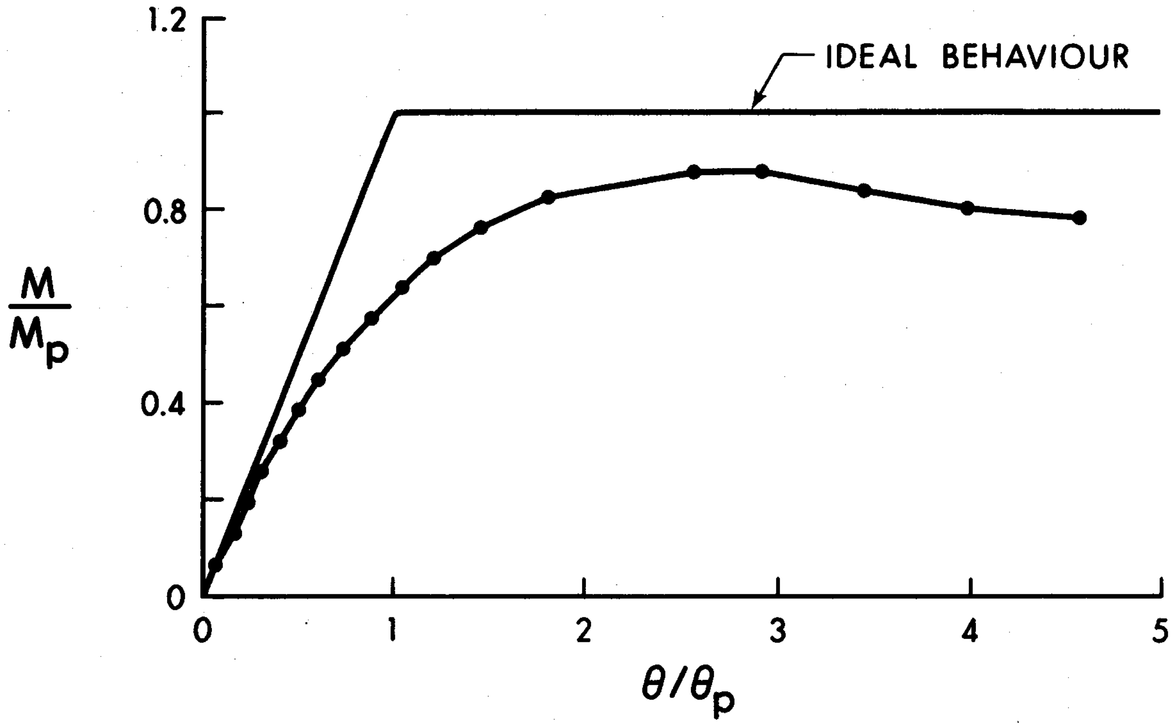


(c) Specimen WS-3

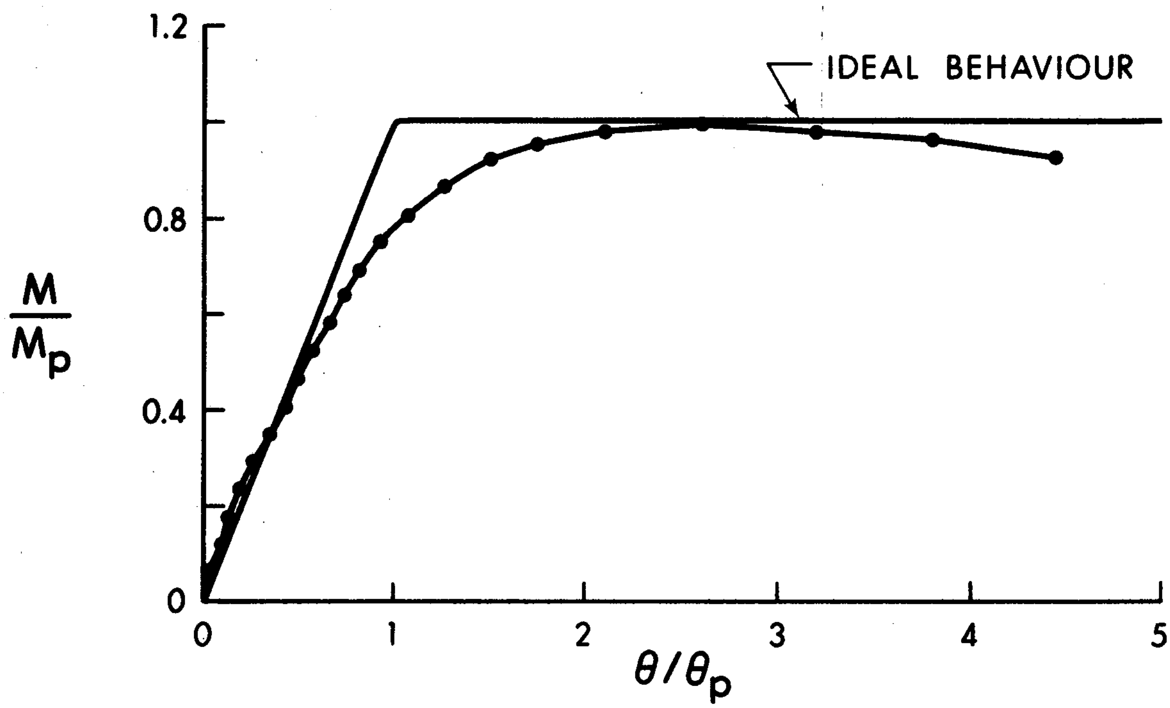


(d) Specimen WS-4

FIG. 4.3 (cont.) MOMENT ROTATION BEHAVIOR

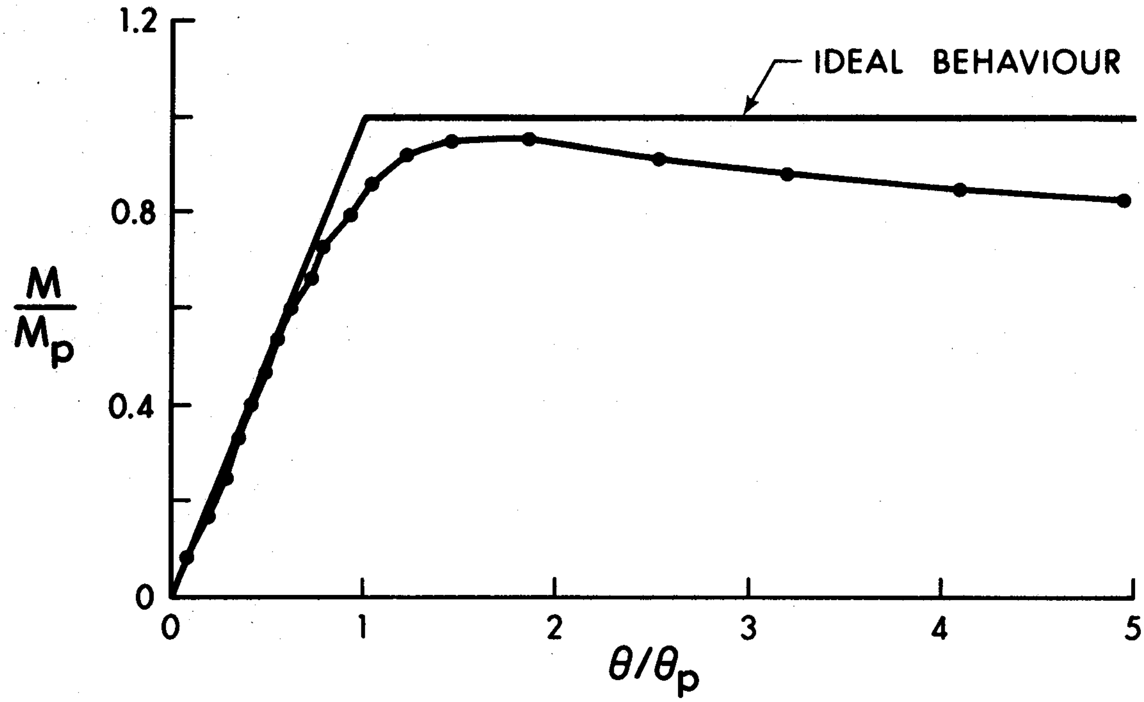


(e) Specimen WS-6

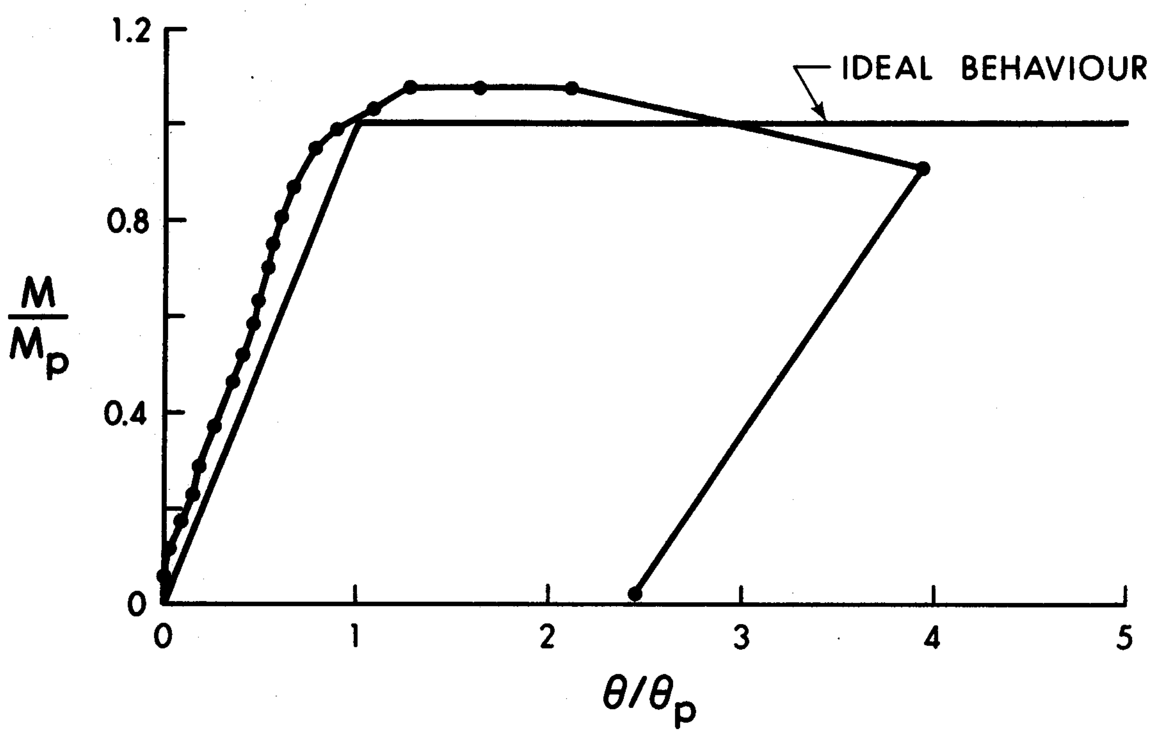


(f) Specimen WS-7-P

FIG. 4.3 (cont.) MOMENT ROTATION BEHAVIOR



(g) Specimen WS-8-P



(h) Specimen WS-9

FIG. 4.3 (cont.) MOMENT ROTATION BEHAVIOR

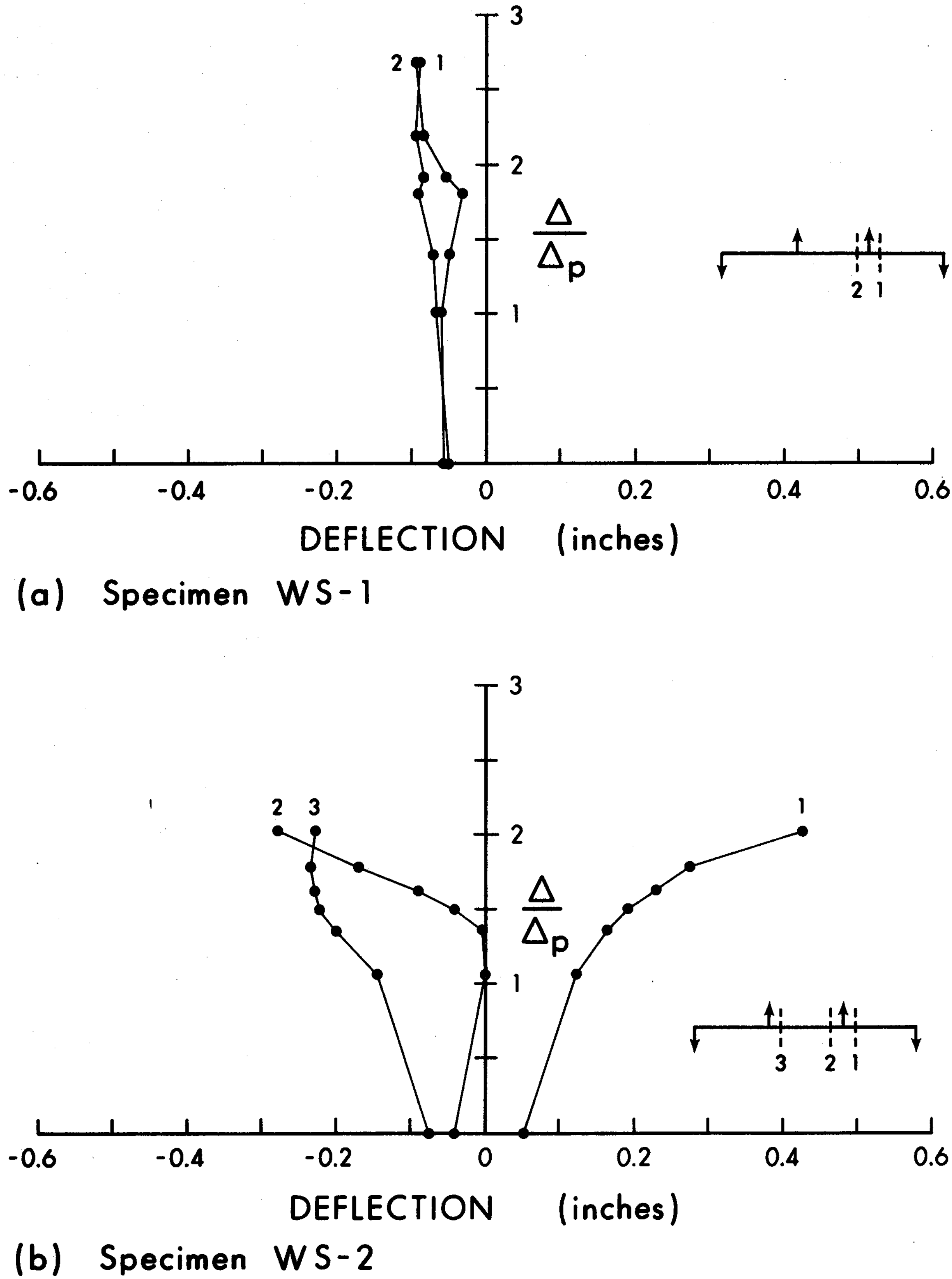
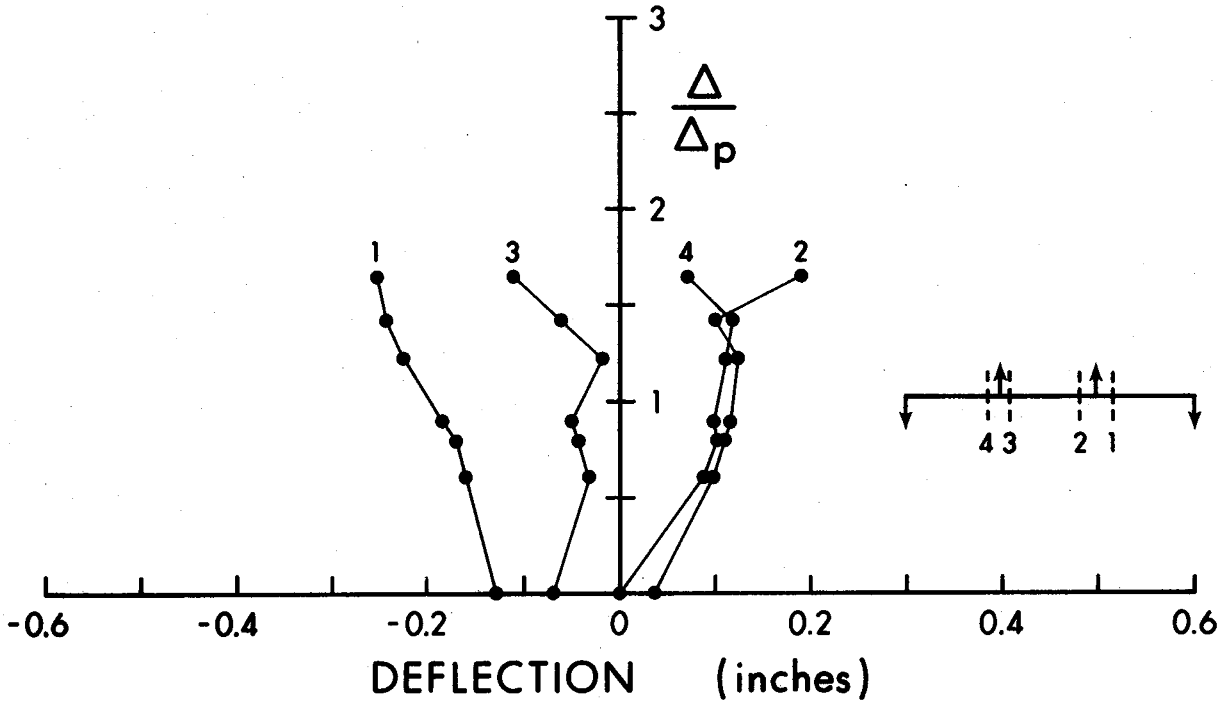
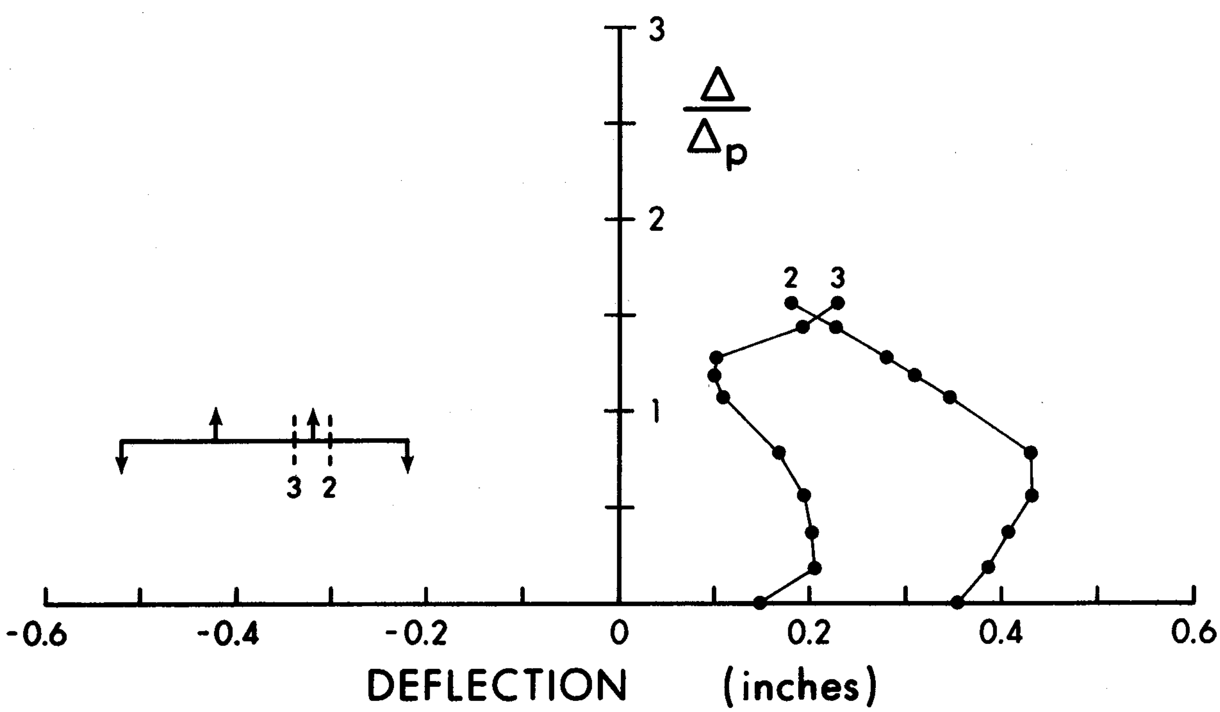


FIG. 4.4 WEB DEFLECTION

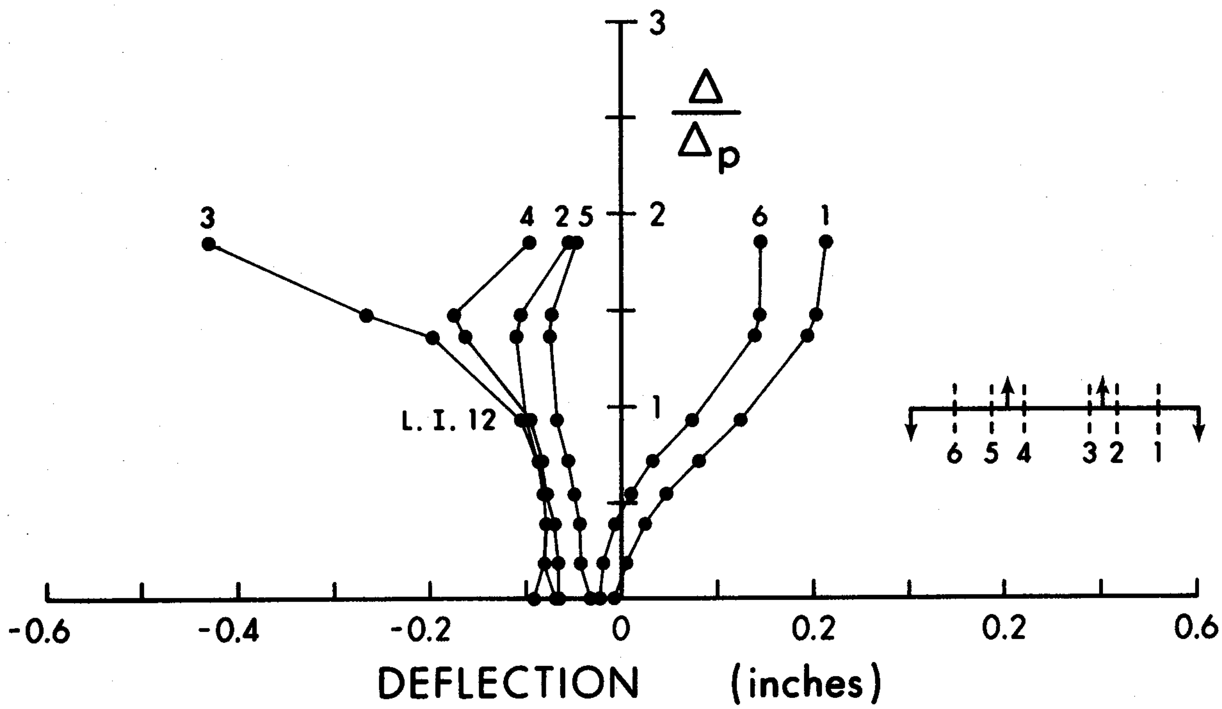


(c) Specimen WS-3

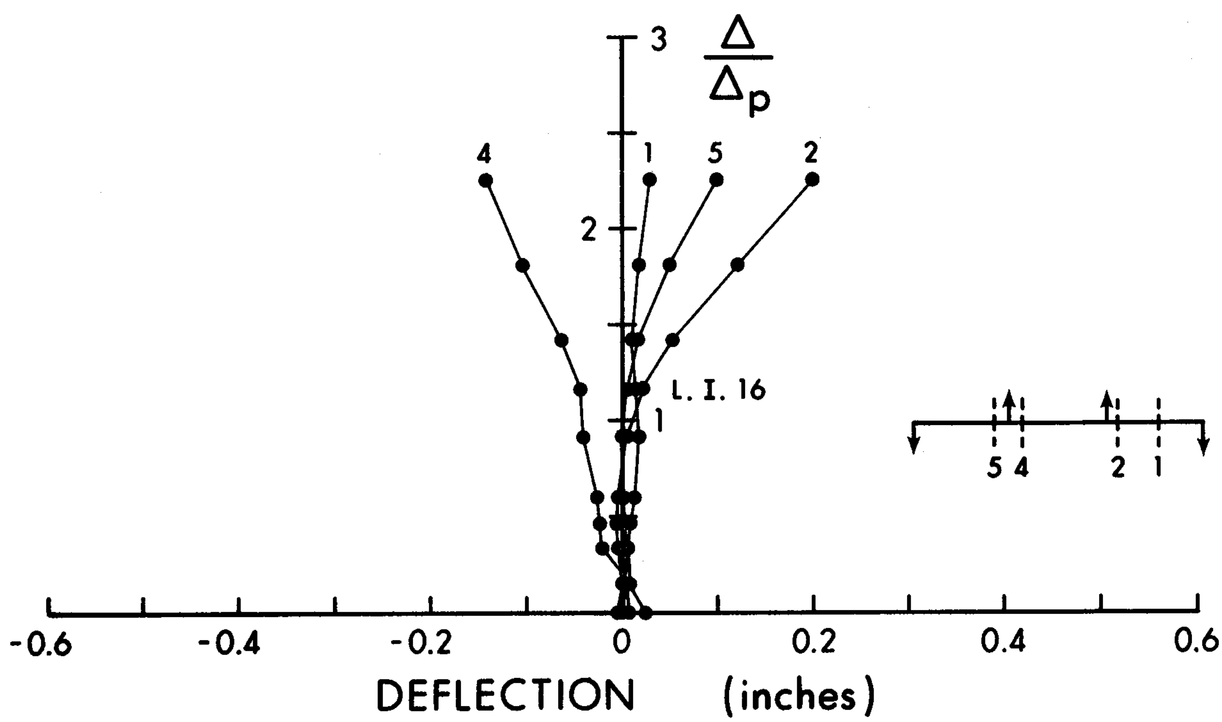


(d) Specimen WS-4

FIG. 4.4 (cont.) WEB DEFLECTION

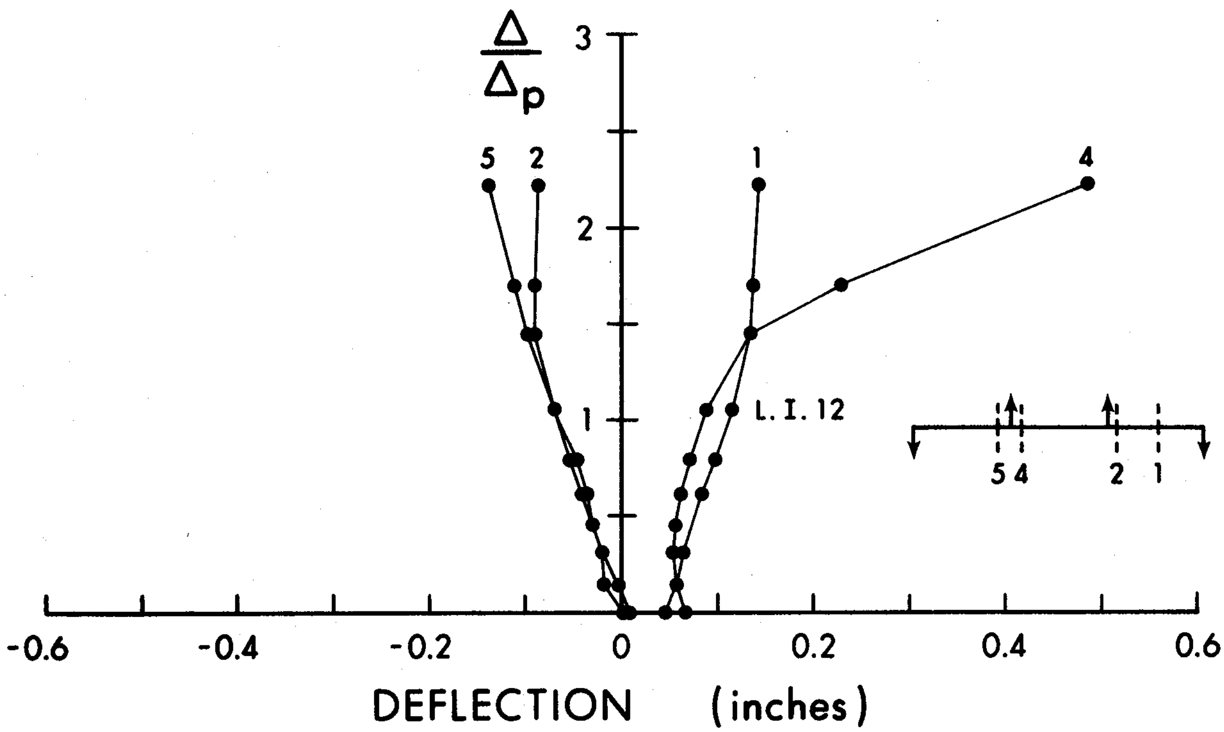


(e) Specimen WS-6

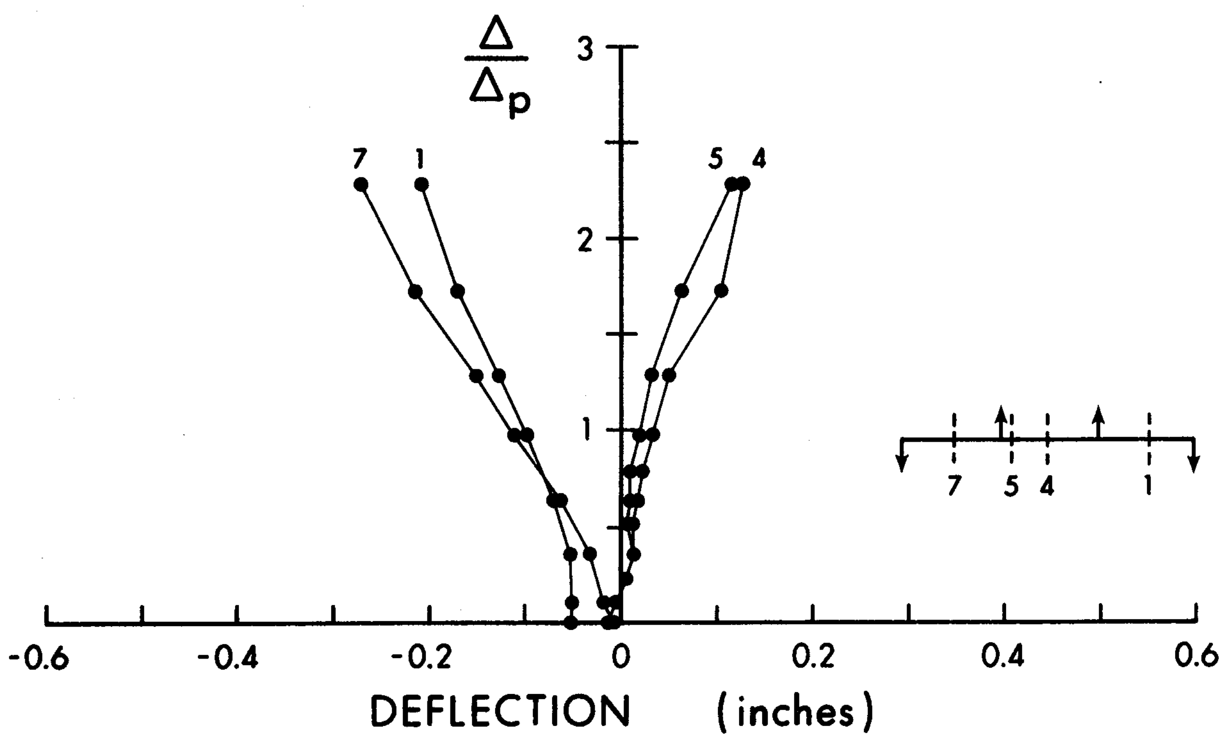


(f) Specimen WS-7-P

FIG. 4.4 (cont.) WEB DEFLECTION

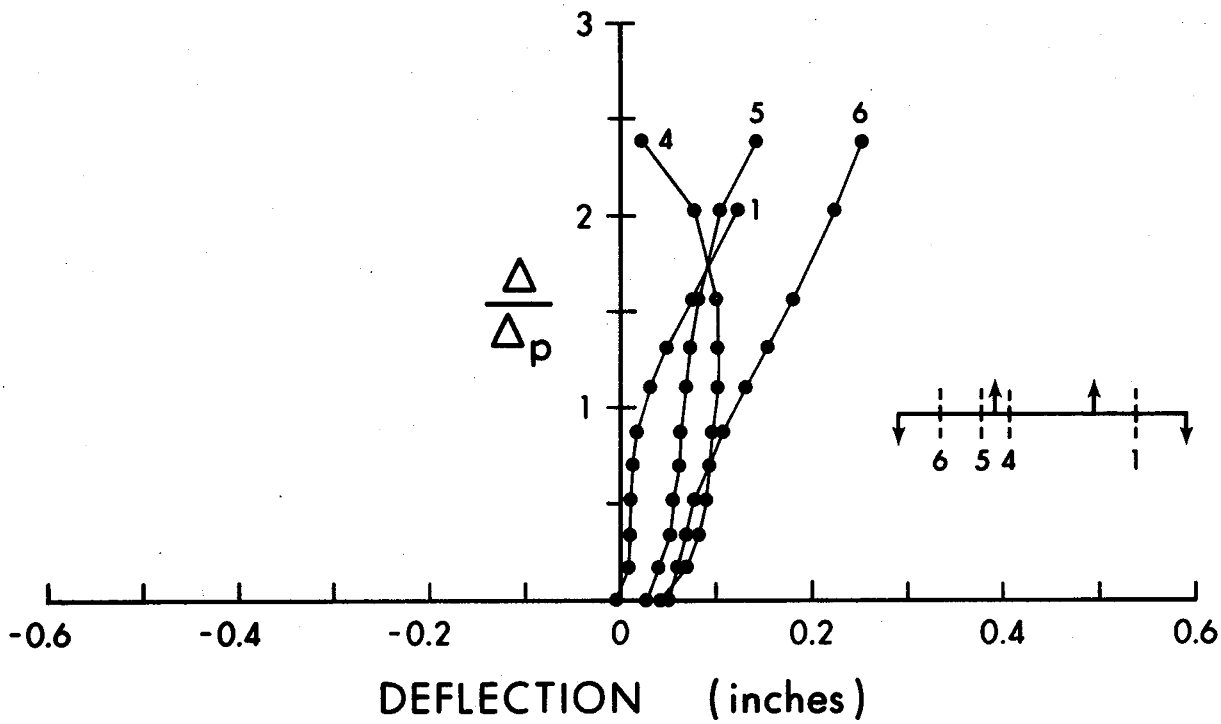


(g) Specimen WS-8-P

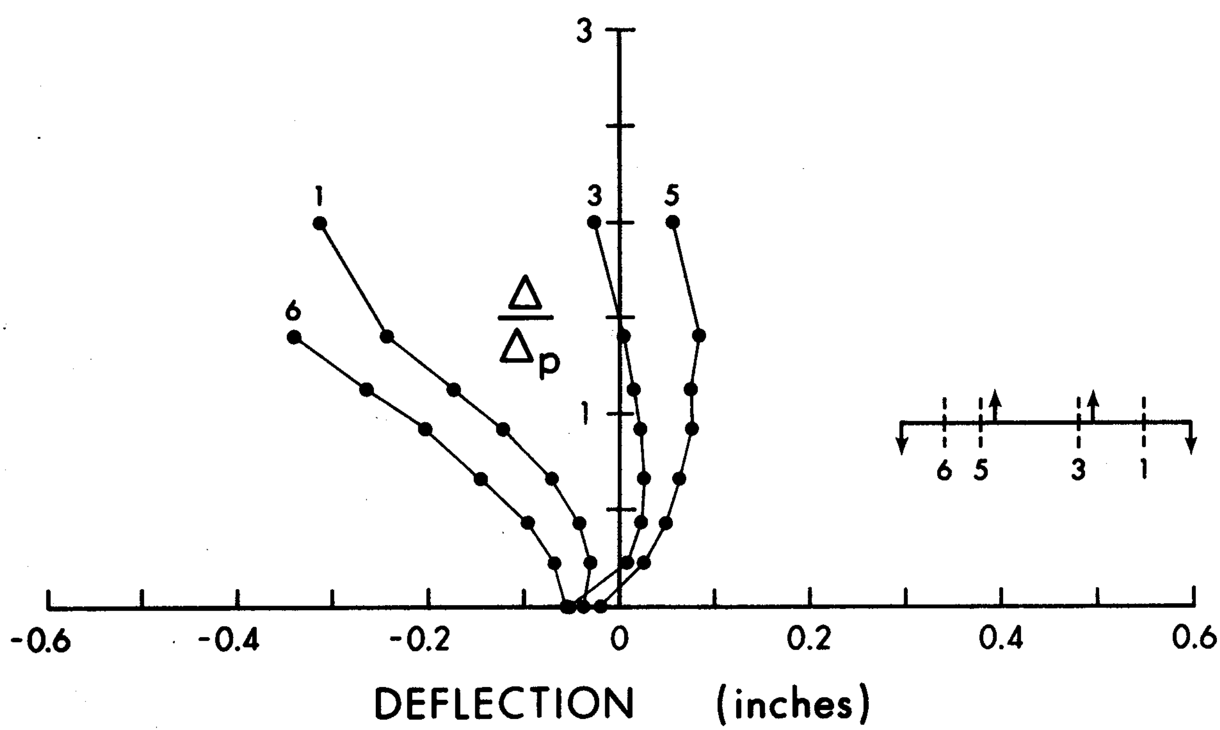


(h) Specimen WS-9

FIG. 4.4 (cont.) WEB DEFLECTION



(i) Specimen WS-10



(j) Specimen WS-11

FIG. 4.4 (cont.) WEB DEFLECTION

5. SUMMARY AND CONCLUSIONS

Tests have been conducted on ten beams with various web slenderness ratios. Eight of the beams tested had flanges proportioned such that the sections qualified as compact sections. The other two specimens had stockier flanges which qualified them as sections suitable for plastic design. The sections tested had varying amounts of shear, ranging from 50% to 98% of the theoretical shear buckling stress. Although not a control parameter, the initial out-of-flatness of the beam web also had an apparent affect on the ultimate strength of the member.

Failure of the beams tested occurred either by web buckling or by flange buckling. Although it is customary to treat these phenomena as independent effects, there is no doubt an interrelationship. However, it can be expected that if the flange is the critical element, its failure will preceed that of the adjoining web and vice versa. In the compact section tests reported herein, there was only one specimen in which a flange buckle occurred before the plastic moment capacity had been reached (WS-2). However, the actual capacity was only about 2% less than M_p and this must be considered within the tolerance of experimental error. One of the two plastic design section also failed by local buckling of the flange but, again, its capacity was very close (99.3%) to the value M_p . Strictly speaking, a beam which reaches its maximum strength on the basis of a flange buckle has not failed in terms of its strength as related to web buckling. The distinction is academic, however. Since the flanges of all beams in this test program were at the slenderness limits prescribed for compact or plastically designed sections, failure by either mode must be considered as the limiting case.

The first four specimens tested showed that an approximately linear decrease in moment capacity occurred as the web slenderness was increased. (See Fig. 4.1). On the basis of these results, a "confirming" test was chosen at a web slenderness value, $(h/w)\sqrt{F_y}$, of 590. This was Specimen WS-6. The amount by which this test fell below M_p , 12%, was exceeded only by Specimen WS-4 which had a web slenderness of 890, higher shear, and a higher initial web deflection. Since this "confirming" test did not clarify the situation, Specimens WS-9, 10 and 11 were tested. These had web slenderness values of 482, 526 and 575, respectively. Shear was high in each of these specimens. All exceeded the required value of M_p (see Table 4.1).

In attempting to assess the effects of the variables, certain comparisons are possible. Specimens 1 and 9 had approximately the same amount of initial out-of-flatness of the web (.0030 vs. .0027, see Table 4.2) and both exceeded the value of M_p (see Table 4.1). The shear in WS-9 was 90.4% of the theoretical shear buckling value while the corresponding value was only 49.9% for WS-1. A similar comparison can be made between WS-2 and WS-11. Again, the significant difference was the amount of shear present and it apparently did not have a marked effect on the moment capacity.

Specimens 2, 6 and 11 can be compared in an attempt to evaluate the effect of initial out-of-flatness. The web slendernesses for these three specimens were 618, 590 and 575, respectively, the actual shear to critical shear ratios were 0.661, 0.521 and 0.968, while the initial out-of-flatness values were 0.0032, 0.0047 and 0.0026. Specimen WS-11 had a strength which exceeded M_p by 7%, WS-2 was 2% under, and WS-6 was 12% under its expected M_p. If it is conceded that shear does not influence the moment capacity and that these three web slenderness values are contained within a fairly narrow band, the examination shows that the initial-out-of-flatness can account for the change in moment capacity. Fig. 5.1 shows the comparison in graphical form. At the same time, it must be noted that a comparison between specimens WS-6 and WS-4, which had nearly the same moment capacities but greatly different web slendernesses (590 vs. 890) does not show the same influence of initial web deflection. Specimen WS-4 had an initial web deflection over three times that of WS-6. It also should be pointed out that both WS-4 and 6 failed by web buckling whereas WS-2 and 11 failed by flange buckling. A definitive answer concerning the exact influence of the initial web deflection upon the moment-carrying capacity of these test specimens obviously cannot be given.

Based on the tests reported herein, it is concluded that a suitable web slenderness limit for compact sections is

$$(h/w)\sqrt{F_y} = 520$$

Reviewing the data shown in Fig. 4.1 and contained in Table 4.1, this would seem to be perhaps unnecessarily conservative since two specimens tested with higher values of web slenderness than this (WS 10 and 11) exceeded the required moment capacity by a significant margin and a third (WS-2) was under its capacity by only a small amount. The choice was tempered, however, by the results obtained for specimen WS-6 which had a substantial under-capacity.

Of the two sections suitable for plastic design, only one (WS-7-P) reached its M_p value. This section had a web slenderness, $(h/w)(\sqrt{F_y})$, of 528. As Fig. 4.3(f) shows, there was not enough rotation of the plastic hinge in order that this specimen would qualify for plastic design. Based on these two tests, therefore, no recommendation can be made that the present web slenderness limit of $(h/w)(\sqrt{F_y}) = 420$ be raised.

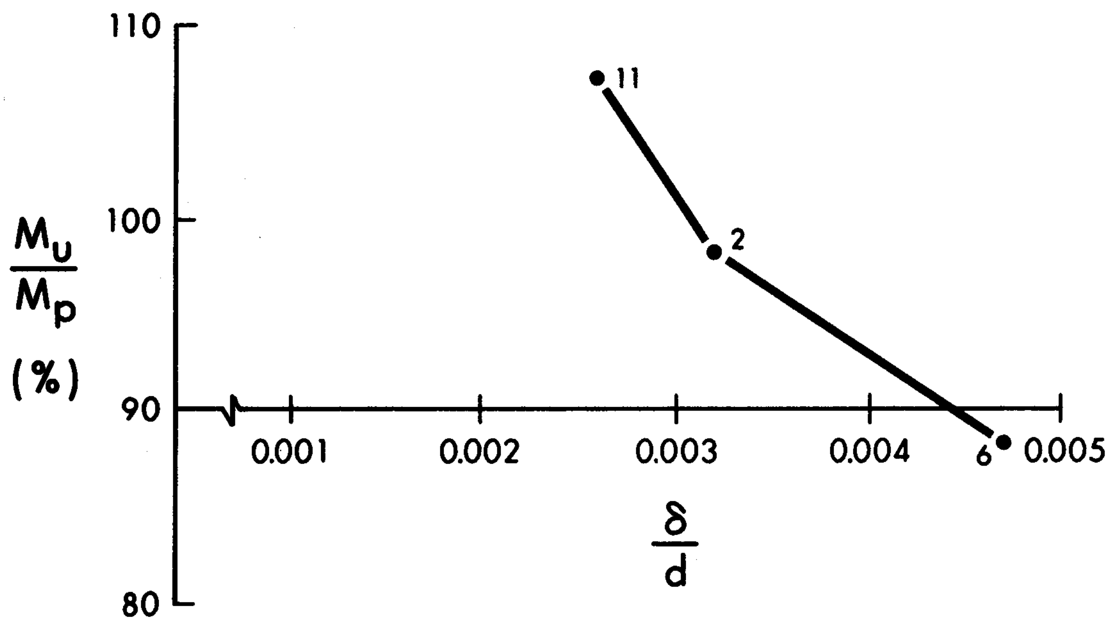


FIG. 5.1 EFFECT OF INITIAL WEB DEFLECTION ON MOMENT CAPACITY

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