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Expanded Draft Final Report

on

Effect of Yield-Tensile Ratio on Structural Behavior - High Performance Steels for Bridge Construction

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Abstract

As higher strength steels are considered for the design of structures, a question arises concerning the influence of the yield-tensile ratio on the behavior of members and connections. The question arises because, with traditional methods of steel production, the yield-tensile ratio typically increases with increasing strength levels. However, with newer production methods, steels with lower ratios can be produced. Consequently, a review and interpretation of the literature was made to shed light on this subject.

Most of the recent work related to the effect of the yield-tensile ratio has been done in Japan. The work was largely directed at developing steels for framed structures with improved resistance to severe earthquakes. Yield points of higher strength steels developed include 50, 65, and 100 ksi levels, with yield-tensile ratios from 0.75 to 0.85. For such severe applications, members must demonstrate the capability to deform well into the inelastic range. For example, seismic design of buildings in the United States requires a flexural member rotation capacity of about seven times that required to reach the yield moment. Load and resistance factor design for general bridge and building applications requires a capacity of three. Thus, the required deformation capacity depends on the application.

Because the shape of the stress-strain curve varies for different steels, several related effects may be implied when reference is made to the effect of the yield-tensile ratio. Important properties may include the length of the yield plateau (if one exists), the slope of the stress-strain curve in the inelastic range, local elongation (elongation in the necking region), and uniform elongation (total elongation less local elongation). In general, steels with different yield-tensile ratios have different values for these properties, and in experimental work, it is difficult to isolate individual effects. Thus, the yield-tensile ratio tends to serve as an umbrella for related properties.

Studies on bending members indicate that the rotation capacity tends to decrease with increasing yield-tensile ratio, and that it can affect the final failure mode. However, all applications do not require the same level of rotation capacity, and it is not necessary to maximize the rotation capacity for each one. If required levels are established, analytical and experimental studies can determine the maximum yield-tensile ratio that would provide that capacity. Also the mode of failure can be controlled. For example, flange width-to-thickness ratios and web depth-to-thickness ratios can be selected that will allow a member to reach its required strength and rotation level, but that will ensure inelastic local buckling before reaching the strain required for tension flange rupture.

The strength of columns with yield-tensile ratios up through about 0.95, based on measured properties, can be reasonably predicted with the present relationships that are used for other steels. The material stress-strain curve must be reasonably linear, with the proportional limit in a tension test at least 85 percent of the yield strength. The same conclusion holds for local buckling strength for stresses up to the yield point. However, if a short compression member of given proportions is compressed beyond the yield point level, the maximum average stress that can be reached in proportion to its yield strength, tends to increase with decreasing yield-tensile ratios of the steel. Also, when ultimate load is controlled by inelastic local buckling, beams and columns of steels with low yield-tensile ratios will withstand larger deformations. However, if such post-yield behavior is needed, it can be realized by decreasing width-to-thickness ratios of flange and web elements.

For bolted tension members, if it is desirable for gross section yielding to control rather than net section rupture, greater yield-tensile ratios require greater ratios of net-to-gross section area. However, most specifications do not require that gross section yielding control the design. Apparently, adequate ductility for most applications is provided by bearing deformations at bolt holes and shear deformations in bolts. However, the fracture of diagonal braces through end joints of many buildings in Japan in earthquakes in 1968 and 1978, which had material with an "unusually high" yield-tensile ratio, led to seismic code revisions. Regarding the effect of joint length on strength, the effect is similar for steels with different yield-tensile ratios.

The yield-tensile ratio is a not a significant factor in determining the fatigue strength of fabricated members.

One of the essential assumptions in structural design is that each member and each connection have a capacity for rotation or deformation adequate to ensure that its intended function can be fulfilled. For example, if a structure is designed to develop strength as a mechanism, the rotation at a hinge must not be terminated prematurely by local buckling or by fracture. Thus, in the development of high performance steels, it is essential that adequate notch toughness and fracture ductility be provided, both in the parent material and in weldments. This is a critical part of the development process. The notch toughness required depends on environmental conditions, loading characteristics, and the fabrication details employed.

The required deformation capability depends on the application. Cold-formed structural members fabricated from steels with yield-tensile ratios up to 0.93 have been used successfully for many years. Indeed, members with yield-tensile ratios up to 1.00 have performed adequately in tests and used for a limited range of applications.

Research has shown that, ignoring effects of strain concentrations, pressure vessels of higher-strength steels with higher yield-tensile ratios tend to burst at a higher percentage of their tensile strength than vessels of lower strength steels. For pressure vessels with notches, Royer and Rolfe (1974) have shown that the reduction in burst pressure is directly proportional to the reduction in wall thickness at the notch, provided the notch depth does not exceed about 25 percent of the vessel wall. The tests included material with a yield-tensile ratio up to 0.93.

The highest strength structural steel, A514, has performed well in bridge and building applications, even though the yield-tensile ratio based on measured properties ranges up to 0.93 or 0.95. Thus, provided a reasonable level of total ductility is maintained, such as the 16 to 18 percent (depending on thickness) minimum elongation in 2 in. specified for A514, steels with yield-tensile ratios up to about that level can be effectively utilized in most design applications. Steels with a yield strength greater than about 70 ksi have been designed as non-compact members. Additional work should be undertaken to define compactness in this range.

Japan has developed steels for seismic applications that provide a maximum yield-tensile ratio of 0.80 for yield strengths from 50 to 65 ksi, and 0.85 for a 100 ksi yield strength steel. However, the studies reviewed did not specifically show that such ratios were the highest values that might be acceptable for the application. Additional studies would be desirable to set more precise limits.

Information for developing statistical parameters for material properties (such as yield strength, tensile strength, and yield-tensile ratio) for the new steels produced in Japan was not available in the references.

Answers were provided to various questions previously posed concerning the application of steels as related to the yield-tensile ratio. Also, simple illustrative models concerning were offered to show trends in behavior that are influenced by the yield-tensile ratio. Included were local yielding and stress redistribution as well as end rotation of a beam with a moment gradient.

The studies reviewed suggest that new high performance steels can probably be included in the AASHTO Bridge Specifications with little modification. This will depend to some extent on the characteristics of the steel and the design treatment that is desired.

Introduction

As higher strength steels are considered for the design of structures, a question arises concerning how the behavior of members and connections might be affected by the yield-tensile ratio, the ratio of the yield strength to the tensile strength. The question arises because, with traditional methods of steel production, the yield-tensile ratio typically increases significantly with increasing strength levels. However, with newer production methods, steels with lower ratios can be produced. Thus, the question of the importance of, and appropriate level of, the yield-tensile ratio has received renewed attention. Consequently, a review and interpretation of the literature was made to shed light on this subject.

Figure 0.1 shows the general relationship between the yield-tensile ratio and the tensile strength for steels made by traditional processes and steels made by the newer processes (Ohashi et al, 1990). As indicated, there is some overlap between the two for steels with a tensile strength less than about 100 ksi. The primary higher strength structural steels (A572 Grade 50, A588, A852, and A514) are located on the chart based on specified minimum yield and tensile strength, but actual ratios tend to be higher. The yield-tensile ratios for the first three steels based on minimum properties (0.77, 0.71, 0.78) compares with a <u>specified maximum</u> ratio of 0.80 for the new Japanese steels of this strength level. The yield-tensile ratio of A514 steel based on minimum properties (0.91) compares with a <u>specified maximum</u> value of 0.85 for the new Japanese steel at this strength level.

To illustrate the effect of the new processes on the shape of the stress-strain curve, consider Figure 0.2 (Ohashi et al, 1990). The lower curve is for Nippon Steel's HT80 grade with a minimum yield strength of 100 ksi, a tensile strength of 114/135 ksi, and a maximum yield-tensile ratio of 0.85. It is made by the DQ-L-T (direct quenched, lamellarized, and tempered) process. The upper curve is for the same composition, a low-nickel-microalloyed steel, but produced by the QT (quenched and tempered) process. The curves are terminated at 10 percent strain. The new process results in a more rounded curve but about the same total elongation (24 vs. 23%). The actual yield-tensile ratios in this case were 0.83 and 0.95.

The following sections consider how structural behavior is affected by the yield-tensile ratio, and the shape of the stress strain curve in general.

Part 1 - Review of Literature

Research in Japan

Kato (1990) made a comprehensive review of the effect of the yield-tensile ratio on the structural performance of steel tension members, flexural members, and beam columns. He reminded us that in design, we anticipate that structural members have sufficient capacity to deform axially or rotate at the limit load without premature failure. He then proceeded to establish simple theoretical relationships between the plastic deformation capacity of structural members and the shape of their material stress-strain curve. Some limited test information and numerical analysis was offered in support. He concluded that, to secure sufficient deformation capacity, the yield-tensile ratio must be "reasonably low". Because this paper does an excellent job of focusing on the issues, it will be reviewed in some depth.

<u>Tension Members.</u> A common detail for tension members is a bolted splice which results in some length of the member having a reduced section (net section) because of the bolt holes. Kato simulated this with a plate of varying width as shown in Figure 1.1. He considered a stress-strain curve with arbitrary values of yield strength (ϕ_y), yield strain (ε_y), tensile strength (ϕ_u), strain at tensile strength (ε_u), and strain at initial strain hardening (ε_{st}). The elongation of the member at ultimate load depends on the yield-tensile ratio ($Y=\sigma_y/\sigma_u$). If the ratio is 1.0, and if the member is subjected to increasing loads, the length of the member that yields approaches zero and total elongation is limited. At smaller yield-tensile ratios, a zone of increased length is able to reach the yield strength while the minimum section is reaching the tensile strength, and thus the total elongation before rupture increases. Kato derived the following expression for maximum elongation (δ_{max}) of a plate of length 2L:

$$\delta_{\max} = 2\left\{\left[\varepsilon_{st} - \frac{Y(\varepsilon_u - \varepsilon_{st})}{1-Y}\right]X + \frac{\varepsilon_u - \varepsilon_{st}}{1-Y}\int_0^X \frac{A_o}{A_x}dx + \frac{\varepsilon_y}{Y}\int_X^L \frac{A_o}{A_x}dx\right\} \quad \text{........................ (1.1)}$$

A_0 is the cross section area at mid length and A_x is the area at point X which defines the limit of yielding ($A_x\sigma_y=A_o\sigma_u$ or $A_x\sigma_y=A_o/Y$).

In Figure 1.2, the above equation for a plate with a reduced section is compared to test data for a 10 mm plate fabricated from four steels with a yield-tensile ratio from about 0.65 to 0.85. At applied net section stresses of $\sigma = 0.95\sigma_u$ and $\sigma = \sigma_u$ the agreement with test data is reasonable. The trend of decreasing elongation with increasing yield-tensile ratio is clearly shown for these conditions. In Figure 1.3, a similar result is shown for a rectangular plate with a central hole. The dashed line is for Equation 1.1, the solid line for the result from an inelastic finite element solution, and the steels are the same grades as before. In both of these simple cases the theoretical relationship shows that at yield-tensile ratios greater than about 0.85, the elongation rapidly reduces. The experimental data are over a smaller range (0.65<Y<0.85) but tend to support the trend. (The four grades of steel referred to in Figures 1.2 and 1.3 as SS41, SM50, SM50Y and SM58 are presumed to be designated by their tensile strength in kgf/mm^2. These are equivalent to 58, 71, and 82 ksi.)

Kato also noted that the diagonal braces of many buildings in Japan fractured through end joints in earthquakes in 1968 and 1978 without developing substantial elongation. An investigation showed that the yield-tensile ratio of the material was "unusually high", and this led to seismic code revisions in 1981.

<u>Bending Members.</u> To illustrate the effect of the yield-tensile ratio on bending members, Kato examined the case of an end braced cantilever, Figure 1.4. Under an increasing load, the moment at the fixed end reaches the plastic moment (M_p) and, depending on the yield-tensile ratio, increases to some higher value (M_u) as maximum load is reached. The plastic region extends over some distance between the points where M_u and M_u are reached, thus increasing the rotation capacity of the beam as the end slope increases from θ_y to θ_u. If the material is elastic-perfectly plastic (Y=1.0), the moment can not increase above M_p, there is no extension of the plastic region away from the support, and the tensile flange will fracture as the plastic moment is reached. Kato cited as verification the tests reported by McDermott (1969b) on A514 beams with Y=0.9 which fractured immediately after tension flange yielding.

To numerically show the effect of various yield-tensile ratios on the rotation capacity of the cantilever, Kato idealized the stress-strain curve with linear segments. He assumed that the member was proportioned to eliminate local and lateral buckling. Idealizing the beam cross section as a two-flange no-web model with a depth h_e that gives the same plastic moment and area as a real beam of depth h, he derived the following expression for end rotation (R_θ) at maximum load:

$$R_q = KY\left[\frac{C_3}{C_1} + \frac{1}{2}\left(\frac{C_2}{C_1}\right)\right] \text{ .... (1.2)}$$

where $K = h_e/h$, $C_1 = \sigma_y\varepsilon_y/2$, $C_2 = (s-1)\sigma_y\varepsilon_{st}$, $C_3 = (s-1)^2\sigma_y^2/2E_{st}$ and $s = 1/Y$. The elastic modulus is E and the modulus in the strain-hardening range (assumed constant) is E_{st}. For a reference material he selected a steel with the following values: σ_y = 235 MPa (34 ksi), Y = 0.60, $\varepsilon_{st}/\varepsilon_y$ =10, and E/E_{st} = 50. He then varied each of these variables in turn (σ_y, Y, $\varepsilon_{st}/\varepsilon_y$, and E/E_{st}) to show their individual effect while holding the others constant.

Figures 1.5(a) through 1.5(d) show the results. The ordinates have been normalized by the rotation for the variable held constant. For example, in Figure 1.5(a), increasing the yield strength from 235 MPa (34 ksi) to 520 MPa (75 ksi) decreases the end rotation before failure of the cantilever beam to about 0.87 times that of the reference steel. However, the effect of increasing the yield-tensile ratio is much more dramatic. As shown in Figure 1.5(b), the rotation decreases rapidly with increasing Y so that at values of 0.80 and 0.90 the rotation is only about 0.25 and 0.10 times that of the reference steel. Figure 1.5(c) shows that the rotation increases with the $\varepsilon_{st}/\varepsilon_y$ ratio, a parameter that corresponds to an increased horizontal plateau in the stress-strain curve. Figure 1.5(d) shows that the rotation increases with the E/E_{st} ratio, a parameter that corresponds to a flatter slope in the strain hardening region.

To verify the predicted rotation relationship, tests were run on beams of two higher strength trial-production steels. The stress-strain curves are shown in Figure 1.6 and the properties were as follows:

Steel	Yield Strength, σ_y, MPa	Yield Strength, σ_y, ksi	Tensile Strength, σ_u, MPa	Tensile Strength, σ_u, ksi	Yield-Tensile Ratio,Y	Strain Ratio, $\varepsilon_{st}/\varepsilon_y$	Modulus Ratio, E/E_{st}
A	661	96	716	104	0.92	6.2	131.6
B	483	70	656	95	0.74	3.2	28.3

Note that steel A has higher $\varepsilon_{st}/\varepsilon_y$ and E/E_{st} ratios, which should increase rotation, but a higher Y ratio, which should decrease rotation. Bending tests showed that steel A had a rotation capacity at maximum load 0.41 times that of steel B, which compared favorably with a predicted value of 0.38. Thus, the yield-tensile ratio effect dominated the behavior and reduced rotation capacity.

<u>Beam Columns.</u> Kato also considered the case of a member subjected to an end moment and an axial load. He used a specific I-section member (400 x 400 x 31 x 21 mm) with pinned ends and a slenderness ratio of 30. The basic concept for calculating rotation capacity was similar to the preceding but the effects of the axial stress were accounted for. The rotation capacity was limited by the post-buckling capacity of the flange and web, which depends on width-to-thickness ratios and material parameters. See also Kato (1989).

The steels investigated were as follows:

Steel	Yield Strength, σ_y, MPa	Yield Strength, σ_y, ksi	Tensile Strength, σ_u, MPa	Tensile Strength, σ_u, ksi	Yield-Tensile Ratio,Y	Strain Ratio, $\varepsilon_{st}/\varepsilon_y$	Modulus Ratio, E/ E_{st}
Y90	617	89	686	99	0.90	5	200
Y75-A	470	68	627	91	0.75	5	50
Y75-A'	470	68	627	91	0.75	1	50
SM50	372	54	529	77	0.70	9	70
BSS41	304	44	441	64	0.70	12	120

The rotation capacity was defined as $h = (q_{max} / q_p) - 1$ where q_{max} is the maximum end rotation and q_p is the rotation at full plastic moment. The axial load was normalized as p = P/P_y where P is the load and P_y is the yield load in pure compression. Figure 1.7 shows how the analytically determined rotation capacity at maximum load varies with the axial force ratio for each of the steels. At low axial force ratios the rotation capacity for steels with a high Y is much less than that for steels with a low Y. As the axial force ratio increases, the rotation capacity decreases and difference in rotation capacity for the different steels also decreases. However, the steel with the highest yield-tensile ratio still shows the least rotation capacity.

<u>Effect of Yield Strength Scatter.</u> Kato also reported on an extensive plastic analysis of a nine story building with steel moment frames subjected to lateral loads to show the effect of yield strength variability. A log-normal distribution of yield strength was assumed with a coefficient of variation of 0.10. A total of 200 frames were analyzed. The study showed that the variation in yield strength could affect the failure mechanism pattern and decrease total deflection and maximum horizontal load. The yield-tensile ratio was not a part of this study. However, it shows the desirability of minimizing the yield strength variation in any new higher strength steels developed for such applications.

Kato (1989) investigated the inelastic rotation capacity of I-section members subjected to bending or bending and axial compression. He derived theoretical moment-rotation relationships for various conditions of bending and compression in terms of the critical stress ratio ($s = s_{crit} / s_y$), and stress-strain properties ($\varepsilon_{st}/\varepsilon_y$ and E/E_{st} ratios). For the critical stress ratio he used a correlation of the results of 68 stub-column tests of SS41 and SM50 steels (34 and 47 ksi yield strength). He was then able to write equations to predict the rotation capacity and compare them with the results of 30 tests of beams and beam columns found in the literature. The agreement was fairly reasonable. Finally, for any required member rotation capacity, interaction equations were written for maximum values of the flange width-to-thickness ratio and the web depth-to-thickness ratio that will permit such rotation before inelastic local buckling failure.

This mainly theoretical study focused on the effects of compression. The material parameters used in the study included the $\varepsilon_{st}/\varepsilon_y$ and E/E_{st} ratios, but not the yield-tensile ratio. Rotation increased with increasing values of these ratios as discussed above. In subsequent studies, Kato replaced ($s = s_{crit} / s_y$) with ($s = s_u / s_y$) where the limiting condition was the tensile strain.

Kuwamura and Kato (1989) reviewed studies made on the inelastic behavior of high strength steels with low yield-tensile ratios. They discussed the tensile behavior of rectangular and tapered tensile specimens, and the effect of holes. The results were similar to those presented by Kato (1990). They presented a derivation of expressions for the length of plastic hinge that can develop in beam columns under moment gradient and showed that the hinge length increases with decreasing yield-tensile ratio. Theoretical results presented for rotation capacity were the same as subsequently given by Ohashi (1990).

The authors discussed an experimental investigation of local buckling behavior in which two steels were used to fabricate numerous stub column I-section specimens with various depth-to-thickness (d/t_w) and flange width-to-thickness (b/t_f) ratios. The properties of the steels were as follows:

Property	Steel A	Steel B
Tensile Strength, σ_u, kgf/mm^2	66.9	72.9
Yield Strength, σ_y, kgf/mm^2	51.6	68.1
Yield-Tensile Ratio, Y	0.77	0.93
Strain at Tensile Strength (Uniform Elongation)	12.8	9.5
Tensile Strength, σ_u, ksi	95.1	103.7
Yield Strength, σ_y, ksi	73.4	96.8

As indicated, steel A had a much lower yield-tensile ratio and higher uniform elongation than steel B. The stub columns were compressed past yielding and the stress and strain reached at maximum load were noted (σ_{max} and ε_{max}). Figure 1.8 shows the compression curves for the stub columns, as well as plots relating the d/t_w and b/t_f ratios to σ_{max}/σ_y and $\varepsilon_{max}/\varepsilon_y$ for each steel. It is apparent that higher stress and strain ratios were reached for steel A, which had the lower yield-tensile ratio. This led the authors to conclude that, when ultimate load is controlled by inelastic local buckling, beams and columns of steels with low yield-tensile ratios will withstand larger deformations. However, the behavior observed may well have been controlled by the combined effects of the $\varepsilon_{st}/\varepsilon_y$ and E/E_{st} ratios, not the yield-tensile ratio per se. The stress-strain curves show that, compared to steel B, steel A had a lower E/E_{st} ratio but a much higher $\varepsilon_{st}/\varepsilon_y$ ratio. It is difficult to separate these effects experimentally.

Kuwamura and Kato also discuss an experimental investigation of the hysteresis behavior of cantilever beams subject to a dynamic end load. Under the test conditions imposed, the member with the steel having the lowest yield-tensile ratio absorbed much more energy than the other steels. However, the authors did not arrive at a general conclusion as to its application for earthquakes because of the complicated nature of inelastic behavior under earthquake excitation.

Bessyo et al (1991) discussed Sumitomo Steels' development of SM50B steel for large buildings. The emphasis was on weldability and toughness. Plates 100 mm thick were rolled (TMCP) and tested. The yield-tensile ratio was 0.74 to 0.78 and the uniform elongation 11 to 16 percent. Compared to a reference normalized steel with a yield-tensile ratio of 0.65, the new steel had greater reduction of area and Charpy toughness, as well as improved weldability. No structural tests were reported. Tensile property specifications were as follows:

Property	SM50B
Tensile Strength, σ_u, MPa	490 - 608
Yield Strength, σ_y, MPa	294 min.
Tensile Strength, σ_u, ksi	71 - 88
Yield Strength, σ_y, ksi	43
Total Elongation, %	23

Nagayasu et al (1991) summarized considerations in Kawasaki Steel's development of steel plates with high strength and low yield-tensile ratio for building frameworks. Steels developed included grades 53 and 60. The designation refers to tensile strength in kgf/mm^2 and is equivalent to 75 and 85 ksi. The tensile property targets met and set were as follows:

Property	HT53	HT60
Tensile Strength, σ_u, kgf/mm^2	53 - 65	60 - 73
Yield Strength, σ_y, kgf/mm^2	36 min	45 min

Yield-Tensile Ratio, Y	0.75 max	0.80 max
Tensile Strength, σ_u, ksi	75 - 92	85 - 104
Yield Strength, σ_y, ksi	51 min	64 min

Note that the yield-tensile ratio for grade 53 steel, 0.75, is lower than that for Nippon Steel's HT50 steel, 0.80. Also, the range in tensile strength for grade 60 is somewhat wider than for HT60 (13 vs. 8 kgf/mm^2). The strain hardening modulus, E_{st}, for grade 60 steel was set at 1/30 of the elastic modulus (E/30). No target value for E_{st} was set for grade 53 steel, but the values reported were favorably low (E/48 to E/53).

The plates were developed to meet the following impact properties and composition parameters:

Property	HT53	HT60
Impact (t/4, T dir.)	2.8 kgf·m @ 0 ^{0}C	4.8 kgf·m @ -5 ^{0}C
Carbon Equivalent, C_{eq}	0.37	0.43
Weld Cracking Parameter, P_{em}	0.22	0.23
C_{eq}=C+Si/24+Mn/6+Ni/40+Cr/5+Mo/4+V/14 P_{em}=C+Si/30+(Mn+Cu+Cr)/20+Mo/15+V/10+Ni/60+5B		

Welding studies included tensile and impact properties of material up to 100 mm (4 in.) thick welded by SAW, CES (consumable electroslag), and GMAW with high heat input. Tensile, bending, and impact tests met targeted values for the base metal although there was some degradation of strength and impact properties due to welding.

Low cycle fatigue tests were run on specimens simulating a beam flange welded to the face of a column flange, with a continuity or diaphragm plate on the opposite side, Figure 1.9(a). The results are shown in Figure 1.9(b) in terms of applied axial strain amplitude (half the strain range, e.g., 2 represents +2 to -2) and cycles to failure. For strain amplitudes from 1 to 4 percent, which is well into the inelastic range, the specimens with grade 53 flanges withstood 74 to 7 cycles. Those with grade 60 flanges withstood about twice as many cycles. These results were judged to be adequate for severe inelastic loads such as earthquake loadings. Figure 1.9(c) shows a specimen after testing.

The flexural behavior of a full scale cruciform frame fabricated from grade 60 steel was investigated. The frame was comprised of a box column (600 mm square, 60 mm plates) and intersecting I-section beams (800 by 300 mm, 22 and 40 mm plates). The load-deformation curve showed ductile behavior with the load increasing in the inelastic range until the bending moment was 1.36 times the plastic moment. In the beam at the beam-to-column joint, the compression flange buckled locally as maximum load was reached and subsequently the tension flange at the same location fractured. The rotation at maximum strength was 8.29 times the rotation at yield load. Thus the deformation capacity was deemed satisfactory

Ohashi et al (1990) summarized considerations in Nippon Steels' development of steel plates with high strength and low yield-tensile ratio for building structures. To show the importance of the yield-tensile ratio, he reviewed the simple case of diagonal brace in tension, Figure 1.10. To absorb significant energy, the member must yield in the gross section (A_g) before it fractures in the net section through the bolt holes (A_e). This leads to the requirement that $Y < A_e/A_g$. The energy absorbed is the area under the load-displacement curve. Because the gross section is much longer than the net section, it absorbs much more energy as it yields. The simple expression above can be modified because the rupture stress on the net section tends to be higher than the tensile strength as discussed by Kulak (1987).

Studies were also reviewed that showed how the plastic deformation capacity of beam-columns depends on the yield-tensile ratio, Figure 1.11. The deformation capacity was shown to depend on the column load ratio, and to be much larger for the steel with of a low yield-tensile ratio. The study was based on steels with a yield-tensile ratio of 0.77 and 0.93 (see Kuwamura and Kato, 1989). The authors also claimed experimental verification.

Properties targets were set based on the work of Kato (1990) and related studies. Steels developed included HT50, HT60, and HT80 grades. The designation refers to tensile strength in kgf/mm^2 and is equivalent to 71, 85, and 114 ksi.

Tensile property targets, which were subsequently met, were as follows:

Property	HT50	HT60	HT80
Tensile Strength, σ_u, kgf/mm^2	50 min	60 - 68	80 - 95
Yield Strength, σ_y, kgf/mm^2	33 min	45 - 55	70 min
Yield-tensile Ratio	0.80 max	0.80 max	0.85 max
Tensile Strength, σ_u, ksi	71 min	85 - 97	114 - 135
Yield Strength, σ_y, ksi	47 min	64 - 78	100 min

The yield strength variation for the new steels was reduced to insure the likelihood of a collapse mechanism for laterally loaded frames in which the beams yield before the columns. This is efficient because it negates the need to over-design the columns to allow for the effect of an over-strength beam. Apparently efforts were made to reduce the COV of the yield strength from 10 percent to 5 or 2.5 percent.

Results are reported of bending tests of two beams of HT60 steel with yield-tensile ratios of 0.72 and 0.93. These are probably the two beams referred to by Kato (1990) although the reported properties are a little different:

Steel	Yield Strength, σ_y, kgf/mm^2	Yield Strength, σ_y, ksi	Tensile Strength, σ_u, kgf/mm^2	Tensile Strength, σ_u, ksi	Yield-Tensile Ratio, Y	El., %
1	67.6	96	73.0	104	0.926	3.2
2	49.2	70	68.0	97	0.724	6.2

The dimensionless moment-rotation curves resulting from the tests are shown in Figure 1.12. Note that the steel with the lower yield strength, tensile strength, and yield-tensile ratio, resulted in the greater maximum moment ratio (M/M_p) by a factor of about 1.25, and the greatest angle of rotation ratio (θ/θ_p) at maximum load by a factor of about 1.8. Web buckling limited the maximum load.

Information is given on production methods, welding characteristics and toughness. The HT50 steel is designated as a high-weldability steel with a carbon equivalent of 0.40 percent or less. This is based on the equation

$$C_{eq}=C+Si/24+Mn/6+Ni/40+Cr/5+Mo/4+V/14 \quad \text{........................} \quad (1.3)$$

Ohki (1991) of Nippon Steel reviewed market trends in Japan for steel plates used in the construction of buildings, bridges, and transmission towers, as well as the underlying technology. The information on buildings includes that discussed previously on high-strength low yield-tensile ratio steels for earthquake resistance and will not be repeated here.

The information on bridges includes a brief discussion of the Honshu-Shikoku bridge project. This is a very large project involving three separate routes and numerous bridges. At the time of the paper presentation, bridges on one route had been completed and those on a second route started. The specification for the steel was agreed upon after much study. It is interesting to note that there was no reported discussion or requirement given for the yield-tensile ratio. The minimum specified properties given below suggest that it was likely about 0.90.

Property	HT70		HT80	
Thickness range, mm	$t \le 50$	$50 < t \le 100$	$t \le 50$	$50 < t \le 100$
Tensile Strength, σ_u, kgf/mm^2	70 - 85	68 - 83	80 -95	78 -93
Yield Strength, σ_y, kgf/mm^2	63	60	70	68
Tensile Strength, σ_u, ksi	100 - 121	7 - 118	114 - 135	111 -132
Yield Strength, σ_y, ksi	90	85	100	97

Impact properties and composition parameters were as follows:

Property	HT70		HT80	
Thickness range, mm	$t \le 50$	$50 < t \le 100$	$t \le 50$	$50 < t \le 100$
Carbon Equivalent, C_{eq}	0.49	0.52	0.53	0.56
Impact	4.8 kgf·m @ -15 ^{0}C	4.8 kgf·m @ -15 ^{0}C	4.8 kgf·m @ -15 ^{0}C	4.8 kgf·m @ -15 ^{0}C
Charpy Energy Transition Temperature, ^{0}C	-35	-35	-35	-35

Otani (1992) summarized some of the work done by Nippon Steel in the development of steel plates for building construction. Properties for the three steels with low yield-tensile ratios are the same as given by Ohashi (1990). Tensile strength designations HT50, HT60, and HT80 (kgf/mm^2) are referred to here as HT490N, HT590N, and HT780N (N/mm^2).

Otani discusses the relative importance of the yield-tensile ratio and uniform elongation in increasing plastic deformation capacity. (As subsequently discussed, total elongation in a tensile specimen can be considered to be comprised of two parts, local elongation in the region that necks and fractures, and uniform elongation in the remainder of the specimen. See Dhalla and Winter (1974).) Uniform elongation is related to the yield-tensile ratio, tending to decrease with increasing ratios as shown in Figure 1.13. Otani refers to Japanese studies that show that the most effective way to increase deformation capacity is to (1) decrease the yield-tensile ratio for steels that have over about 10 percent uniform elongation, and (2) increase the uniform elongation in other cases. The first category would include a 85 ksi tensile, 64 ksi yield strength steel (HT60 or HT590N, Y=0.74), and the second a 114 ksi tensile, 100 ksi yield strength steel (HT80 or HT780N, Y=0.83).

Otani refers to a common standard for heavy steel plates. Tensile property specifications for Grades HT50 and HT53 were as follows:

Property	HT50	HT53
Tensile Strength, σ_u, kgf/mm^2	50 - 62	53 - 65
Yield Strength, σ_y, kgf/mm^2	33 min	36 min
Yield-Tensile Ratio	0.80 max	0.80 max
Tensile Strength, σ_u, ksi	71 - 88	75 - 92
Yield Strength, σ_y, ksi	47 min	51 min

Impact properties and composition parameters were as follows:

Property	Thickness range, mm	HT50	HT53
Impact (t/4, T dir.)	All	2.8 kgf·m @ 0 ^{0}C	2.8 kgf·m @ 0 ^{0}C
Carbon Equivalent, C_{eq}	41 - 50	0.38	0.40
	51 - 100	0.40	0.42
Weld Cracking Parameter, P_{em}	41 - 50	0.24	0.26
	51 - 100	0.26	0.27
C_{eq}=C+Si/24+Mn/6+Ni/40+Cr/5+Mo/4+V/14 P_{em}=C+Si/30+(Mn+Cu+Cr)/20+Mo/15+V/10+Ni/60+5B			

The specifications listed by Otani for HT53 steel for the yield-tensile ratio, C_{eq} and P_{em} are somewhat different than those referred to earlier by Nagayasu (1991).

Research at U.S. Steel

McDermott (1969a) investigated local plastic buckling of flanges in A514 steel members. Specifically, he conducted axial compression load tests on 12 cruciform sections with projecting width-to-thickness ratios from 2.77 to 9.80. The material yield-tensile ratio varied from 0.89 to 0.92. The specimens failed in local buckling. He determined that the maximum width-to-thickness ratio of an out-standing element to permit hinge rotation at yield adequate for plastic design was 5.0. This is equivalent to the value of 8.5 specified by AISC (1989) for plastic design with A36 steel, extrapolated by inverse proportion to the square root of the yield strength. However, for all except seismic applications, AISC (1993) now permits higher values ($65/\sqrt{65}$ = 10.8 for A36).

McDermott also made theoretical analyses, building on prior work by Lay (1965), which is discussed subsequently under Lehigh University Research. McDermott was able to show a close correlation between experimental maximum stresses and theoretical predictions from the deformation theory of plasticity.

The expression for the tangent modulus in shear (G_t) from deformation theory is

$$G_t = (E_s/E)G \quad (1.4)$$

For steel, G/E = 0.385 so

$$G_t = 0.385E_s \quad (1.5)$$

He approximated the secant modulus (E_s) for a steel with a stress-strain curve such as that of A514 steel as

$$E_s = \frac{\sigma_c}{0.005 + \dfrac{\sigma_c - \sigma_y}{E_t}} \quad \text{...} \quad (1.6)$$

Combining Equations 1.5 and 1.6 with the expression for the buckling stress of a flange element in the inelastic range, $\sigma_c = (t/b)^2 G_t$, he arrived at the following equation:

$$\sigma_c = \sigma_y + E_t\left[0.385\,(t/b)^2 - 0.005\right] \quad \text{.............................} \quad (1.7)$$

Experimental values for various b/t were in close agreement with this equation. The equation shows that, to reach some value of σ_c past yield to allow for hinge rotation, the limiting b/t would decrease with decreasing tangent modulus. The tangent modulus would typically decrease with increasing yield-tensile ratios.

McDermott (1969b) also investigated the plastic bending behavior of A514 steel beams. Seven simply supported beams were tested under a pair of central loads to obtain a region of uniform moment, and two were tested under a single central load to study the effects of a moment gradient. The yield strength of the flange material was 111 ksi or greater, and the yield-tensile ratio varied from 0.90 to 0.93.

For the beams under uniform moment, No. 1 through 5 had short unsupported flange lengths (slenderness ratio, L/r_y, ratio of unbraced length to radius of gyration, from 11.6 to 5.4) and were designed to fail by local buckling of the compression flange, either before or after reaching yield. No. 1 and 2 (ratio of half flange width to thickness, b/t of 12.3 and 8.02) failed by local flange buckling at maximum moments 0.78 and 0.97 times the plastic moment. No. 3, 4, and 5 (b/t of 7.3, 6.0, and 5.4) failed by local flange buckling at maximum moments 1.01, 1.02, and 1.02 times the plastic moment. No. 6 and 7, which had longer unsupported flange lengths (L/r_y of 24.9 and 23.9) and small b/t (b/t of 3.18 and 4.80), failed by combined lateral and local buckling of the compression flange at 1.02 and 1.00 times the plastic moment. Figure 1.14 shows dimensionless moment-rotation curves for specimens 3 through 7. (M/M_p is the ratio of the experimental moment to the plastic moment; θ is the experimental rotation over the length L; and $\phi_p L$ is the hypothetical rotation from an elastic analysis with $M = M_p$.) All of these beams continued to strain and deflect after buckling and eventually unload as the applied deformation increased.

The beams under moment gradient, Designated A (L/r_y=37.5, b/t=3.23) and B (L/r_y=35.4, b/t=4.82), were designed to reach yield and subsequently fail by lateral or local buckling. Both Beams A and B developed plastic hinges in the vicinity of the concentrated load as expected. They sustained continuously increasing loads and reached M/M_p values of 1.17 and 1.14. Unlike the other specimens, the final failure mode was that of tension flange rupture which caused abrupt unloading. Both ruptures were preceded by tension flange necking. The ruptures extended through the webs and terminated in a compression zone. Figure 1.15 shows the curvatures developed and the extent of the plastic hinge zone.

McDermott developed the following equation for the hinge rotation (θ_{HA}) that could be developed in a centrally loaded simple beam:

$$\theta_{HA} = \frac{2M_pL}{EI}\left(1-\frac{M_p}{M_m}\right)\left[s+\frac{h}{2}\left(\frac{M_m}{M_p}-1\right)\right] \quad \text{.............................. (1.8)}$$

where M_p is the plastic moment, M_m is the maximum moment in the beam, L is the unbraced length, E is Young's modulus, I is the moment of inertia, $s = \varepsilon_{st}/\varepsilon_y$ (ratio of strain at beginning of strain hardening to the yield strain), and $h = E/E_t$ (ratio of Young's modulus to the tangent modulus in the strain hardening range). M_m=M_p for an elastic perfectly plastic material and $M_m/M_p > 1.0$ for a strain hardening material. Thus, the equation indicates a need for strain hardening to obtain hinge rotation. Also $\varepsilon_{st}/\varepsilon_y$ and E/E_t should be as large as possible to maximize the rotation. The rotation depends on the moment gradient. For a moment that varies linearly from $0.5M_m$ to M_m in span L, multiply the above equation by 2.0; for a moment that varies from $-M_m$ to $+ M_m$, multiply by 0.50.

The experimental rotations for the beams with moment gradients (the more severe condition) were about 1.6 times the calculated rotations, reaching a value about two times the product of the elastic curvature and the unbraced span length. Since earlier studies had shown that this was about twice the required rotation, McDermott concluded that A514 steel had sufficient rotation capacity for use in plastically designed structures with either uniform moments or moment gradients. For local buckling and lateral buckling considerations, he recommended that b/t be limited to 5 and L/r_y to 21 (uniform moment regions) or 36 (regions with linear moment gradient).

McDermott (1970) continued his investigation of the plastic bending behavior of A514 steel with tests of five additional beams. The yield strength of the flange material was 107 ksi or greater, and the yield-tensile ratio varied from 0.90 to 0.93. In these tests, two beams had a linear variation in moment with end moment ratios in the unbraced length of 0.5, two beams had a linear variation in moment with end moment ratios in the unbraced length of -1.0 (reversed bending), and the fourth simulated a uniformly loaded three-span continuous beam. Both beams with end moment ratios of 0.50 failed by lateral buckling (L/r_y=25) after attaining a moment greater than the plastic moment (M_m/M_p=1.17 and 1.14) but reaching only about 3/4 the desired rotation. Both beams with end moment ratios of -1.0 failed by tension flange rupture, one (M_m/M_p=1.18) after reaching about 3/4 the desired rotation and the other (M_m/M_p=1.14) after attaining the desired rotation; both had the same slenderness ratio (L/r_y=60) and the reason for the premature rupture of one of the beams was not known. The three-span beam failed by tension flange rupture after sustained a loading 6 percent greater than the calculated mechanism loading, thus exhibiting the required strength and rotation.

McDermott speculated that, with reduced slenderness ratios, the buckled beams would have attained the desired rotation. Based on the entire 14-beam test program, he concluded that A514 steel was suitable for use in plastically designed structures, but that its suitability might be marginal in certain cases.

It should be noted that, despite these research efforts, A514 steel was never accepted for plastic design. Even though the test beams sustained significant rotation, some may not have been comfortable with the eventual failure mode of tension flange rupture exhibited under moment gradient.

Research at Lehigh University

Dexter et al (1993a) described the results of compression tests sponsored by the U.S. Navy on cellular sections representative of double-hull ship construction. These large specimens were fabricated from HSLA-80 steel, which is similar to ASTM A710, Grade A or C, Class 3. Tensile properties were as follows:

	Yield Strength, σ_y, MPa	Yield Strength, σ_y, ksi	Tensile Strength, σ_u, MPa	Tensile Strength, σ_u, ksi	Yield-Tensile Ratio, Y	El., %	Unif. Strain, %
Long.	623.3	90.4	692.3	100.4	0.90	15.21	7.4
Tranv.	579.9	84.1	660.5	95.8	0.88	13.86	6.7
Avg.	601.9	87.3	676.4	98.1	0.89	14.53	7.1

The tests included single-cell and multiple-cell box sections with width-to-thickness (b/t) ratios from 48 to 96. Most tests were of the stub column type, but some column and beam column tests were included. The ultimate compression stress observed ranged from 38 to 72 percent of the yield stress. Where column behavior was not involved, the strength could be predicted by simple empirical equations based on b/t, yield strength, and modulus of elasticity, which had been developed previously from tests on lower strength steels. Where column behavior was involved, the strength could be reasonably predicted by either the tangent modulus method or by inelastic finite element analysis, including the input of residual stresses and initial imperfections. Thus, the behavior of the HSLA-80 components was generally predictable by traditional methods used for other steels.

Dexter et al (1993b) conducted fatigue tests on over 170 large-scale welded HSLA-80 steel fabricated I-section and box-section beams with weld details characteristic of double-hull ship construction. Material properties were similar to those given above. Details evaluated included longitudinal fillet welds, transverse groove welds, bulkhead attachment details, and longitudinal stiffeners left unwelded at their butted ends. The lower confidence limits of the S-N curves were not significantly affected by mean stress, and were not significantly different than those for similar weld details in traditional bridge steels. This confirms the results of earlier tests by others which shows that the fatigue strength of welded details in air can be considered to be independent of the type and strength of steel. Stress range and detail type are the primary factors. Therefore, if larger stress ranges are encountered as a result of the use of higher strength steels, a greater incidence of fatigue cracking may be expected unless fatigue resistance is specifically addressed in design.

Fisher and Dexter (1994) reviewed problems with present high-strength steels and discussed potential applications for new high-performance steels. The main problem noted with present steels was susceptibility to hydrogen cracking during fabrication. In contrast, the high-performance steels are reportedly virtually immune to hydrogen cracking in the heat-affected zone. This characteristic allows them to be welded without preheat in most cases. Also, the toughness of the newer steels is superior, which could lead to easing of restrictions on fabrication of fracture critical members.

It was noted that weld metals that adequately resist hydrogen cracking are available up to 690 MPa (100 ksi) in tensile strength, sufficient to overmatch base metal with 560 MPa (81 ksi) yield strength. Thus, for effective utilization of higher yield strength steels, either new weld metals must be developed or designs must be developed that employ undermatched weld metal.

Some potential applications noted for high-performance steels because of high toughness and resistance to defects were as follows: increased use of one-sided welds, such as for tee joints; increased use of field welding; and retrofitting to improve resistance to fatigue or seismic loadings.

Changes in design configuration were suggested to effectively use the thinner plates of higher strength steels. Examples included, greater use of box sections, possibly stiffened with lightweight concrete or foams; corrugated webs; sandwich panels; and cellular tension flanges on box girders.

Reference is made in the paper to work in progress at the University of Texas (Prof. Karl Frank) on net section fracture through bolt holes, which shows that resistance equations are sensitive to the yield-tensile ratio of the steel. Also, reference is made to recent tests at Lehigh University which show that the bending ductility of a steel with a high yield-tensile ratio is far less than that for mild steels.

Lay (1965) discussed theoretical fundamentals for flange buckling in the inelastic range. This is of importance in establishing maximum flange width-to-thickness ratios (b/t) for compact sections or plastic design.

In the elastic range, the buckling stress (σ_c) for a plate simply supported along one edge is

$$\sigma_c = (t/b)^2 G \quad \text{...} \quad (1.9)$$

where G is the elastic shearing modulus. This can be extended to the inelastic range by substituting the tangent shear modulus (G_t) for G:

$$\sigma_c = (t/b)^2 G_t \quad \text{..} \quad (1.10)$$

For buckling at a stress equal to the yield strength then, the maximum b/t is

$$b/t = (G_t/\sigma_y)^{1/2} \quad \text{...} \quad (1.11)$$

A stress greater than σ_y would be assumed to ensure rotation past first yield. G_t can be approximated based on the deformation theory of plasticity (see discussion under McDermott, 1969a) or slip plane theory. Using slip plane theory, G_t can be expressed in terms of the tangent modulus of elasticity (E_t, the slope of the stress-strain curve in the inelastic range) as

$$G_t = \frac{2G}{1 + \dfrac{E / E_t}{4(1 + \boldsymbol{u})}} \quad \text{..} \quad (1.12)$$

where ν is Poisson's ratio. Considering a given yield strength, with higher yield-tensile ratios, E_t/E will decrease. Thus, Equations 1.11 and 1.12 suggest that G_t will decrease, and the limiting b/t will decrease, as the yield-tensile ratio increases.

However, since tests by McDermott showed that the b/t limit was proportional to (σ_y)$^{1/2}$ for A514 steel with a high yield-tensile ratio, equivalent to assuming a constant G_t in Equation 1.11, the effect may be small.

Research at Cornell University

Dhalla et al (1971) conducted tests as part of an AISI research program to determine the influence of two factors on the suitability of a steel for cold-formed steel members and connections under static loading: (1) ductility and (2) the spread between the yield strength and the tensile strength. The work was directed at bolted and welded connections because these were considered one of the most critical problem areas for low-ductility steels. Tests were made on specimens of five steels including material that had been cold-reduced 45 percent (some was annealed to restore ductility) (Steel X), material that had been cold-reduced 33 percent and not annealed (Steel Y), and commercial A446 Grade E product (Steel Z). Thicknesses ranged from 0.183 in. to 0.038 in. Tensile strengths ranged mostly from 73 to 100 ksi, and the yield-tensile ratio from 0.7 to 1.0. Figure 1.16shows typical stress-strain curves (strain over 2 in. gage length). Note that the major portion of the strain in X or Y steel occurs after ultimate load is reached, whereas much of it occurs before ultimate load in the Z steel.

Tensile tests were first conducted on rectangular specimens with holes to determine behavior under stress concentrations. All steels were able to strain locally and develop the material tensile strength through the net section, except for two specimens of Z steel in the transverse direction (94 percent). Next tests were conducted on lapped specimens with bolted connections and compared to tests on steels of normal ductility. Modes of failure involved shear and bearing, similar to previous tests on traditional material. It was determined that the elongation capacity of the connections was adequate, and that the low ductility of the special steels reduced the strength of the connections by about 15 percent. Finally, tests were conducted on specimens with either longitudinal or transverse fillet welded connections. Ultimate strengths were generally predictable by the usual methods with no reduction for low ductility. However, there was some strength loss in transverse fillet welded specimens fabricated from cold worked material because of the partial annealing effect of the welding.

Dhalla and Winter (1974) reviewed the preceding work, formability, and other studies to develop suggested minimum ductility requirements for steel for cold-formed steel members. They emphasized the difference between local elongation, as measured over a 1/2 in. gage length that includes the necking region, and uniform elongation as measured over a 2-1/2-in. gage length that excludes that region. Average values for materials used in the tests were as follows:

Property	Steels X and Y, long.	Steel Z, long.	Steel Z, transv.
Yield-Tensile Ratio, Y	0.99	0.93	1.00
Local Elongation, 1/2 in. gage, %	24	10	0.4
Uniform Elongation, 2-1/2-in. gage, %	0.6	2.7	0.5
Standard Elongation, 2-in. gage, %	5 - 6	4.4	1.3

In the above mentioned tension tests of specimens with holes, it was observed that fracture occurred more rapidly after maximum load (P_u) than with steels of the usual ductility. In X and Y steels fracture occurred on the descending load-deflection curve at $0.6P_u$; in longitudinally loaded Z steels at $0.8P_u$; and in transversely loaded Z steels at $1.0P_u$. It was concluded that local ductility was more important than uniform ductility for alleviating stress concentrations, and that about 20 percent in a 1/2 in. gage length was sufficient to produce ductile member behavior. The tests of bolted connections confirmed this value.

An analytical study using inelastic finite element methods indicated that a uniform elongation of 3 percent was needed to insure plastification of perforated and notched specimens in tension. This was in reasonable agreement with the results of tests.

Thus it was concluded that for satisfactory structural performance of thin sheet steel members under essentially static loads, the following material requirements should suffice: a minimum uniform elongation of 3 percent, a minimum local elongation (1/2 in. gage length) of 20 percent, and a maximum yield-tensile ratio of 1/1.05 = 0.95. It was shown that these elongation requirements are equivalent to 7 percent elongation as measured in a standard test.

Based on this investigation, the AISI "Specification for the Design of Cold-Formed Steel Structural Members" adopted (1980 or earlier) slightly more restrictive requirements for a maximum yield-tensile ratio of 1/1.08 = 0.93 and a minimum elongation of 10 percent in a 2 in. gage length., or 7 percent in a 8 in. gage length, all based on standard specimens.

Macadam et al (1988) described an investigation made to permit the use of a low-strain-hardening ductile steel (LSHD steel) in cold-formed sheet steel members. Such material has been used in building construction for members such as purlins and girts. It is manufactured by hot rolling, followed by a light cold rolling to improve strength and surface finish, with no annealing. Figure 1.17 shows stress strain curves for the material used for structural tests. Steel R was produced to a thickness of 0.101 in., and Steel R was further reduced to a thickness of 0.096 in. Properties determined using a standard tensile specimen, as well as a special specimen for the compressive yield strength, are given below. Note that, because of the cold reduction, the compressive yield strength was 84 to 79 percent of the tensile yield strength.

Property	Steel O	Steel R
Yield Strength, σ_y, ksi	72.5	82.2
Tensile Strength, k, ksi	75.5	83.4
Yield-Tensile Ratio, Y	0.96	0.99
Local Elongation, %	48.0	33.8
Uniform Elongation, %	4.0	0.8
Standard Elongation, 2-in. gage, %	15.0	9.0
Compressive Yield Strength, σ_{yc}, ksi	61.1	64.8

Tests included simple beam uniform-moment tests to check effective width equations for stiffened and unstiffened elements, to check inelastic reserve strength, and to check inelastic lateral buckling strength. Also included were compression tests on stub columns and intermediate length columns, with cross sections that were not fully effective. Beam cross sections were either boxes or back-to-back C shapes; column cross sections were boxes.

Figure 1.18 shows the response in the inelastic simple beam bending tests. The ultimate moment was 0.92 to 1.15 times the plastic moment, depending on whether the tensile yield strength or the compressive yield strength was used for the compressive elements. In accord with specifications, the calculations assume a maximum strain in the compression flange of three times the yield strain.

The stub column results agreed closely with predictions based on the compressive yield strength. The strength of the intermediate length columns was somewhat underpredicted, but this has been encountered for other steels in this slenderness range.

In general, the structural response was typical of that observed for more traditional steels, characterized by local buckling patterns, gradual yielding, gradual descending load-deflection curves after maximum load, and no fractures. The observed strengths were in reasonable agreement with predictions from the AISI Specification or with the results of tests on members of traditional steels.

As a result of this work, the AISI Specification was amended in 1989 to allow the use of LSHD steels. Specifically, if the standard requirements (maximum yield-tensile ratio of 1/1.08 = 0.93 and minimum elongation of 10 percent in a 2 in. gage length., or 7 percent in a 8 in. gage length) can not be met, the following criteria must be satisfied: (1) local elongation of at least 20 percent in a 1/2 in. gage length across the fracture and (2) uniform elongation outside the fracture of at least 3 percent. This is similar to the original recommendations of Dhalla. It was not felt necessary to impose a maximum yield-tensile ratio under these alternative requirements. However, their use was limited to members such as purlins and girts which support loads principally by bending.

Research at University of Missouri - Rolla

Santaputra and Yu (1986) investigated web crippling and web buckling in cold-formed beams of five different high-strength sheet steels as part of an AISI program to develop information for the design of automotive structural components. Average material properties were as follows:

Material	Yield Strength, σ_y, ksi	Tensile Strength, σ_u, ksi	Yield-Tensile Ratio, Y	Elong. in 2 in, %	Thickness, in.
80DK	58.2	87.6	0.66	25.7	0.048
80XF	88.3	98.7	0.84	22.8	0.082
100XF	113.1	113.1	1.00	8.1	0.062
140XF	141.2	141.2	1.00	4.4	0.047
80DK-2	58.2	86.6	0.67	24.8	0.047
80XF-2	77.1	89.1	0.87	20.4	0.088
100XF-2	116.9	116.9	1.00	10.1	0.065
140SK	165.1	176.2	0.94	4.3	0.046

As indicated, the materials included yield-tensile ratios up to 1.00 and yield strengths up to 165 ksi. Tests were conducted on 264 beam type specimens under central concentrated loads or opposing concentrated loads. The specimens behaved similar to previous specimens of lower strength material, failing by yielding or buckling with no rapid unloading or brittle type behavior. The data were used to develop new equations for predicting strength. The equations were developed in terms of the yield strength, elastic modulus, and geometry. There were no special provisions needed related to the yield-tensile ratio.

Other Structural Research

Davies and Cowen (1994) presented the results of tests in England on large rack structures fabricated from steel with a high yield-tensile ratio. Beam and column components of open-box configuration were tested using both hot-rolled and cold-reduced steel, and a full-size structure using cold-reduced steel was tested to ultimate load. Material properties were as follows (gage length for elongation is unknown):

Thick., mm	Type of Steel	Yield Strength, σ_y, N/mm^2	Yield Strength, σ_y, ksi	Tensile Strength, σ_u, N/mm^2	Tensile Strength, σ_u, ksi	Yield-Tensile Ratio, Y	El., %
2.02	CR	474	69	474	69	1.00	-
2.07	HR	493	71	499	72	0.99	20
3.27	CR	505	73	512	74	0.99	10
3.25	HR	410	59	477	69	0.86	31

The beam tests were simple span with a concentrated load applied through a stub column at midspan; the beam halves had end devices with lugs that snapped in to perforations in the stub column. The failure mode for all specimens was local distortion of the stub column and eventual tearing around the perforations. Thus, these tests were really connector tests. For both material thicknesses tested, the hot-rolled material resulted in ultimate moments about 10 percent higher than those for the cold reduced material, even though the yield strength was nearly 20 percent less in one case. In the compression tests, which included short and intermediate length columns, maximum loads were nearly the same for both types of materials.

A final test was made of a two-bay two-story rack with vertical and horizontal loads. Cold reduced material was used for all components except the welded-on lug-type connectors at the beam ends, which were of hot-rolled material. The loading on the beams simulated a uniform load. The failure mode was sidesway accompanied by ductile moment failures in the beam-to-column connectors, with some lugs eventually shearing. The ultimate loads were close to those predicted from an elastic-plastic second order computer analysis.

These results led the authors to conclude that there is no need for a formal ductility requirement in cold-formed section specifications. A material bend test to ensure formability was deemed more useful than a specified elongation or yield-tensile ratio.

Kulak et al (1987) summarized the extensive testing done over the years on bolted joints and made design recommendations under the auspices of the Research Council on Structural Connections. Ultimate load tests have been conducted on large bolted tension splices with steels ranging from carbon steel through A514 steel. Because the observed behavior was generally similar, the design recommendations made, in terms of the yield strength and tensile strength of the connected material, were common for all steels. However, there are some subtle effects of higher yield and tensile strengths.

In designing a splice for a given load, the bolt shear area is fixed, but the plate cross section area decreases with increasing tensile properties. Thus, the A_n/A_s ratio (net plate cross section area over total bolt shear area) also decreases. This has the effect of decreasing the average bolt shear strength at ultimate load as illustrated in Figure 1.19. As shown, the effect is more pronounced for longer joints.

Regarding joint length, the data referred to by Kulak et al includes tests of A514 steel double-lapped joints with lengths from 10.5 to 63 in. The results correlated well with a mathematical model which was subsequently used to study joints up to 100 in. long. For shorter joints, the bolt shear stress has a reasonably uniform distribution between the fasteners. As the joint length increases, the average shear stress at ultimate load decreases because the end fasteners are subjected to higher strains and fail first. The research showed that, for bearing type connections, an average bolt shear stress of 0.60 times the tensile strength of the bolt could be reached for joint lengths of 50 in. or less, and that for longer joints, a reduction factor of 0.80 should be applied. These results were found to be suitable for steels from A36 to A514. The experimental work included A514 steel with a yield-tensile ratio as high as 0.95. The issue of joint length involves both base metal ductility (bearing deformations) and bolt ductility (shear deformations), but the latter effect is apparently dominant.

As illustrated in Figure 1.20, in a plate with bolt holes, the average tensile stress at failure through the net section, σ_u, is slightly greater than the tensile strength of the material measured in a coupon test, $(\sigma_u)_{coup}$. The apparent strength increases with the A_n/A_g ratio (net plate cross section area over gross plate cross section area). At an A_n/A_g ratio of 0.70, the increase is about 8 percent for A514 steel and about half that much for carbon steel. In Figure 1.20, "g" represents the specimen width and "d" the diameter of the single line of holes.

Because differences in strength attributable to tensile properties were not considered large, lower bounds that are suitable for all the steels were used in developing design criteria. Specific effects of the yield-tensile ratio on strength or ductility, as opposed to strength per se, have not been identified. However, the following general effect may be observed.

Kulak et al noted that it is sometimes desirable for a member to yield in the gross cross section before fracture in the net section. Under severe conditions, this would enable distortion or geometrical adjustment before failure. This requirement would be satisfied theoretically if $A_n/A_g \geq \sigma_y/\sigma_u$. However, typical σ_y/σ_u ratios, not ones based on specification minimum strengths, must be considered. Also, as noted above, the tensile stress through the net section at fracture is greater than the material tensile strength. The expression recommended for design was $A_n/A_g \geq \sigma_y/0.90\sigma_u$. With a steel with a high yield-tensile ratio such as A514, this would require $A_n/A_g \geq 0.91/0.90 = 1.0$, an impossible condition. If cast in terms of design equations to account for reliability effects, such as AASHTO-LRFD (1993), such a requirement would become even more impossible. Equating the expressions for net section tensile strength ($0.80\ \sigma_u A_n$) and gross section yielding ($0.95\ \sigma_y A_g$), it is found that for gross section yielding to govern, $A_n/A_g \geq 1.19\sigma_y/\sigma_u$. For A514 steel, this would require the impossible condition, $A_n/A_g \geq 1.08\ \sigma_y/\sigma_u$. Consequently, for steels with higher yield-tensile ratios, the limit state of net section fracture will generally prevail over gross section yielding unless special details are employed.

Specifications such as AASHTO - LRFD (1993) do not allude to a yield-before-fracture criterion except for the commentary discussion on connecting elements such as splice plates. The specification requirement is that A_n not be taken as greater than A_g in calculations The Commentary indicates that yield will occur before fracture if $A_n/A_g \leq 0.85$; the $\geq$ sign appears to have been inadvertently reversed. The information presented by Kulak et al do not support this for steels with a yield-tensile ratio greater than about 0.77, the value for A572 Grade 50 steel. AASHTO ASD Specifications (1992) use an allowable net section stress in tension splices for A514 steel of $0.46\sigma_u$ versus $0.50\sigma_u$ for other steels.

Miller (1994) discussed the role of welding in the structural distress observed after the Northridge earthquake. Although there were no structural collapses of steel buildings, beam-to-column connections were severely damaged in an estimated 100 buildings in the area. These connections involved hot-rolled wide flange beams, typically with the flanges of the beams groove-welded directly to the column flanges and the web bolted (or bolted and fillet welded) to a shear tab on the column. Under the loads resulting from the earthquake, cracks developed in the region of the flange welds and often propagated into the column. Most material was A36 or A572 Grade 50. Although several factors were involved in this damage, which will not be reviewed here, Miller's comments on the role of the yield-tensile ratio are of interest.

Most designs of this type of beam-to-column connection assume that the beam flanges transfer all or most of the bending moment. and the web transfers the shear force. Thus, to develop the plastic moment of the beam, the material in the beam flange will have to strain harden to a stress greater than the yield strength to make up for the portion of the bending moment that is carried by the web adjacent to the connection. Therefore, if Z is the plastic section modulus of the entire beam cross section and Z_f is the contribution of the flanges, $\sigma_u Z_f$ must be greater than $\sigma_y Z$, to avoid a tensile failure of the beam flange. Consequently,

$$\sigma_y/\sigma_u < Z_f/Z \quad \text{...} \quad (1.13)$$

Most rolled shapes have a Z_f/Z ratio of 0.6 to 0.9. Materials commonly used for such structures have yield tensile-ratios of 0.62 and 0.77 based on specified minimum properties, but values based on actual properties reportedly ranged up to 0.83. The inference is that lower ratios would have been beneficial.

There is no clear answer given as to what role the yield-tensile ratio played, if any, in the cracking that occurred in the Northridge earthquake. However, this discussion serves to remind us of an important interrelationship between design assumptions and material properties that must be considered.

Pressure Vessel Research

Royer and Rolfe (1974) presented the results of work done to study the effects of strain hardening and the yield-tensile ratio on the burst strength of pressure vessels. This was part of a program directed at more efficient utilization of the yield strength of higher strength steels. Their tests of thin-wall pressure vessels, as well as that of other investigators in the program, confirmed the validity of the following modified Svensson equation for predicting the burst pressure (P_B) based on plastic instability:

$$P_B = \sigma_u F_{cyl} \ln W \quad \text{...} \quad (1.14)$$

where σ_u is the material tensile strength, F_{cyl} is a correction factor for cylinders (a slightly different equation for F holds for a spherical vessel), and W is the ratio of the OD to the ID. The correction factor is defined as

$$F_{cyl} = \left(\frac{0.250}{\varepsilon_u + 0.227}\right)\left(\frac{\mathrm{e}}{\varepsilon_u}\right)^{\varepsilon_u} \quad \text{..................................} \quad (1.15)$$

where e is the logarithmic base and ε_u is the true strain at maximum load in the tensile test. In this expression, ε_u serves as a close approximation of the strain-hardening exponent n, where the true stress and true strain in a tensile test are related by

$$\sigma = A\varepsilon^n \quad \text{..} \quad (1.16)$$

where A is a constant.

Figure 1.21 shows how the correction factor varies with ε_u. For thin-wall pressure vessels, a correction factor of 1.0 essentially means that the vessel bursts when the membrane stress reaches the tensile strength of the material. The figure shows that the correction factor, and therefore the burst pressure, decreases as ε_u increases. For high-strength steels with a high yield-tensile ratio, ε_u will typically be less than that for lower strength steels. Thus, ignoring any effects of strain concentrations, pressure vessels of higher strength steels tend to burst at a higher percentage of their tensile strength than do lower strength steels.

The validity of the above approach was confirmed in tests on cylindrical pressure vessels with moderate strain concentrations such as welds, nozzles, and end closures. Also, tests were run on cylindrical vessels with a severe strain concentration, specifically a longitudinal notch with a length 30 percent of the vessel length and a depth up to 35 percent of the wall thickness. Tests by others included in the data had notches up to 50 percent of the wall thickness. Figure 1.22 shows the ratio of the actual burst pressure to the calculated burst pressure of a pressure vessel without a notch, P_A/P_B. For most tests, the reduction in burst pressure was directly proportional to the reduction in wall thickness at the notch. However, for the A517 steel with a high yield-tensile ratio, the reduction in burst pressure exceeded the reduction in wall for the greater notch depth (35 %). This led the researchers to speculate that at some notch depth greater than 25 percent, the reduction may be material dependent and be limited by some measure of ductility or toughness. This was not pursued further because it was felt that inspection programs would certainly likely limit flaws to 25 percent of the wall thickness or less.

Material properties for the A517 steel and three other steels used in the program are given below. (Tests on HY-140(T) and A106B were reported by others and properties are not readily available.)

Steel	Yield Strength, σ_y, ksi	Tensile Strength, σ_u, ksi	Yield-Tensile Ratio, Y	True Strain at Maximum Load, ε_u	CVN at 70F, ft-lb
304SS	31.9	83.7	0.38	0.585	107
A516	52.8	75.8	0.70	0.189	38
A517-Ht. 1	117	126	0.93	0.085	37
A517-Ht. 2	122	132	0.92	0.085	64

Langer (1970) presented a discussion of the design-stress basis for pressure vessels with an emphasis on the effect of different materials on the burst pressure. This earlier review also supported the use of the modified Svensson equation discussed above and will not be further elaborated on here. Additionally, to show the effect of stain hardening on behavior at discontinuities, Langer derived expressions for strain concentrations (K_ε) in a tapered bar and at the end of a cantilever beam in terms of the strain-hardening exponent, n. The strain concentration was defined as the actual peak strain divided by the peak strain calculated for completely elastic behavior on the assumption that the maximum deflections are the same. As shown in Figure 1.23, the strain concentration increases as n decreases. (Reemsnyder (1995) has offered an alternative treatment using Neuber's rule that shows the stain concentration factor may be less for a steel with a high yield-tensile ratio.)

Thus, under identical geometric conditions, a steel with a high yield-tensile ratio will tend to have a lower n and a higher strain concentration. With a lower strength steel with a higher n, the surrounding material tends to control the strain in the plastified zone. Thus, in general, more attention must be paid to the effects of discontinuities in higher strength steels, particularly when the yield-tensile ratio is high.

Part 2 - Summary and Interpretation of Literature

Overview

In the design of steel structures it is assumed that members have the capability to rotate and deflect adequately before collapse or fracture. The required level depends in part upon the design assumptions. For example, a bending member may be designed to reach the yield moment, the plastic moment at one section, or to redistribute moments and develop a mechanism. In addition, some degree of robustness is usually required to allow for unexpected events, and this requirement generally varies with the application. In the design of compact section flexural members, including those used for plastic design, LRFD specifications [AISC(1993) and AASHTO(1993)] provide for a rotation capacity of three times that required to reach the yield moment. In the design of such members for seismic applications, AISC provides for a rotation capacity of approximately seven times that required to reach the yield moment.

One method for assessing the behavior of members made from different steels is to compare the capacity for energy absorption as measured by the area under the moment-rotation curve of bending members, or the load-deflection curve of axially loaded members. Steels of higher strength hold the potential for a favorable comparison on this basis. However, if the rotation or deflection is limited by premature buckling or fracture, even in the inelastic range, the area under the curve is limited and the comparison may be unfavorable.

In this regard, as higher strength steels are considered, the influence of the yield-tensile ratio on the behavior of structural members is often questioned. The question arises because, with traditional methods of steel production, the yield-tensile ratio typically increases with increasing strength levels. However, with newer production methods, steels with lower ratios can be produced. Thus, the question of the importance of, and appropriate level of, the yield-tensile ratio has received renewed attention.

Most of the recent work related to the effect of the yield-tensile ratio has been done in Japan. The work was largely directed at developing steels for framed structures with improved resistance to severe earthquakes. Yield points of higher strength steels developed include 50, 65, and 100 ksi levels, with yield-tensile ratios from 0.75 to 0.85. For such severe applications, members must demonstrate the capability to deform well into the inelastic range. For other applications, the required deformation capacity may be much less.

Implications of Yield-Tensile Ratio

It is very convenient to focus on the yield-tensile ratio when comparing different steels. However, it is somewhat of an over-simplification because the shape of the stress-strain curve is different for different steels. For example, in compression, inelastic buckling is influenced by the length of the yield plateau, if one exists, and the slope of the stress-strain curve in the inelastic range. In tension, both the local elongation (elongation in the necking region) and the uniform elongation (total elongation less local elongation) are important. In general, steels with different yield-tensile ratios have different values for these properties, and in experimental work, it is virtually impossible to isolate individual effects. Thus, several related effects may be implied when reference is made to the effect of the yield-tensile ratio.

Member Behavior

Bending Members

Bending members involve consideration of both tension and compression. If the member is proportioned so that local and lateral instability is prevented, the maximum rotation is reached when the flange reaches its tensile strength. Otherwise, if the member is braced to prevent torsional-flexural buckling, the rotation will be limited by local buckling of the flange and web.

The rotation capacity corresponding to tensile flange rupture can be determined by analysis. Kato (1990) made theoretical analyses to determine the maximum rotation in a cantilever beam with an I cross section. The material stress-strain curve was characterized by an initial linear slope (E) to the yield strength (σ_y), a yield plateau (extending from a strain of ε_y to ε_{st}), and a linear slope (E_{st}) in the strain hardening range until reaching the tensile strength (σ_u). These variables are sufficient to define the strain at which the tensile strength is reached. Kato determined how the rotation capacity was affected by independently varying the ratios σ_y/σ_u, $\varepsilon_{st}/\varepsilon_y$, E/E_{st}, and the yield strength. It was shown that the rotation capacity decreases rapidly with increasing yield-tensile ratio, and decreases gradually with increasing yield point. The rotation capacity increases as the yield plateau increases in length (increasing $\varepsilon_{st}/\varepsilon_y$ ratio) and as the strain hardening modulus decreases (increasing E/E_{st} ratio), as also shown by McDermott (1969b). The linearized stress-strain relationship was used to approximate the behavior of two steels (σ_y/σ_u = 0.93 and 0.74) in beam tests described further by Ohashi (1990). The rotation capacity of the steel with the lower yield-tensile ratio was 2.43 times that of the other, in reasonable agreement with the predicted value of 2.61.

Kato (1989) also derived a more comprehensive set of relationships for the rotation capacity of I-sections that are limited by inelastic local buckling of the flange and web. The relationships involved the same parameters as discussed above, but the maximum stress was limited by a critical stress determined by correlations with stub column tests. The correlation involved only steels with yield strengths of 50 ksi or less and the only material parameter was the yield strength. However, local buckling studies on A514 steel by McDermott (1969a) showed that maximum flange width-to-thickness ratios for this steel can be extrapolated from those for lower strength steels in proportion to the square root of the yield strength.

The general effect of the yield-tensile ratio on the behavior of a beam can be considered as follows. In a beam with a moment gradient, if the yield strength is less than the tensile strength, the plastic region of a beam can extend over some length of the beam as the bending moment at the critical section increases above the plastic moment by virtue of strain hardening. The spread of the plastic region contributes greatly to the rotation capacity. On the other hand, if the yield strength and tensile strength are equal (yield-tensile ratio of 1.0), there can be no extension of the plastic region because the tension flange can rapidly reach its ultimate strain and rupture as the plastic moment is reached. Thus, the inelastic rotation capacity of a beam with a moment gradient approaches zero as the yield-tensile ratio approaches 1.0.

This trend has been observed experimentally. McDermott (1969b, 1970) tested several I-section beams of A514 steel with a yield-tensile ratio of 0.90 to 0.93. Beams under uniform moment and moment gradient were included. The maximum moment reached generally exceeded the plastic moment (by up to 17 percent), and all showed considerable inelastic rotation. The maximum rotation of most of the beams was limited by local or lateral buckling. However, for several of the beams under moment gradient, the failure mode was abrupt rupture of the tension flange after inelastic rotation and tension flange necking. This behavior had not been observed in tests of beams of lower strength steels. However, Nagayasu et al (1991) discussed the results of a test involving a cantilever I-section beam framing into a box column. The steel had a minimum yield strength of 45 ksi and a maximum yield-tensile ratio of about 0.80. This beam exhibited extensive inelastic rotation and reached a moment 1.36 times the plastic moment. However, it also eventually exhibited a failure mode of tension flange rupture. In view of the strength level and rotation achieved, the performance was deemed satisfactory.

As previously stated, the yield-tensile ratio per se is not the only parameter that affects rotation capacity. Otani (1992) refers to studies by Toyoda et al that show the deformation capacity is affected by both the yield-tensile ratio and the uniform elongation of the steel. They found that the most effective way to increase deformation capacity was to decrease the yield-tensile ratio for steels with over 10 percent uniform elongation, such as a 64 ksi yield strength grade with a ratio of 0.74, and to increase the uniform elongation in other cases, such as a 114 ksi yield strength grade with a ratio of 0.83. However, data for numerous steels shows that the uniform elongation tends to increase naturally with decreasing yield-tensile ratio.

Cold-formed steel members with yield-tensile ratios up to 1.00 have performed adequately in bending. This has been demonstrated in tests conducted by Macadam et al (1988) on beams under uniform moment and by Davies and Cowan (1994) on full-size rack structures.

Columns and Beam Columns

If a short compression member is compressed beyond the yield point level, the maximum average stress that can be reached in proportion to its yield strength, increases with decreasing width-to-thickness ratios of flange and web elements and decreasing yield-tensile ratios of the steel. Also, when ultimate load is controlled by inelastic local buckling, beams and columns of steels with low yield-tensile ratios will withstand larger deformations. This was demonstrated in tests reported by Kuwamura and Kato (1989) on steels with a yield-tensile ratio of 0.93 and 0.77.

The strength of columns with yield-tensile ratios up through about 0.95, based on measured properties, can be reasonably predicted with the present relationships that are used for other steels. The material stress-strain curve must be reasonably linear, with the proportional limit in a tension test at least 85 percent of the yield strength. Compression member strength has been demonstrated in earlier tests of columns of A514 steels, and in more recent tests of columns of HSLA-80 steel by Dexter et al (1993a).

Tests on cellular HSLA-80 construction by Dexter et al (1993a), as well as tests of thin cold-formed sections by Macadam et al (1988) and Santaputra and Yu (1986), showed that the local buckling strength at stresses below and approaching the yield strength level can also be predicted from relationships used for lower strength steels. The tests on cold-formed steel involved material with a yield-tensile ratio of as high as 1.00.

Beam-columns show the combined effects of bending and axial compression. As the material yield-tensile ratio increases, both the maximum strength and the rotation capacity of the member decrease. The severity of the effect decreases with increasing axial load ratios (P/P_y). These trends are based on numerical analysis as discussed by Kato (1990) and Ohashi et al (1990). Experimental verification is claimed.

Tension Members

Tension members can be considered in two groups, those without a reduced section such as welded members and those with a reduced section (net section) such as bolted members.

If there is no reduced section, the entire length of the member will stretch when the yield strength is reached and the deformation will tend to be large. The total deformation of the member will depend on the strain at which the tensile strength is reached. However, as the yield-tensile ratio approaches 1.0, the load required to completely fail the member will be the same as the yield load, although the deformation will be large. In structural configurations, as opposed to isolated hangers for example, the situation would likely be relieved by a redistribution of loads.

On the other hand, if there is a reduced section, the behavior will depend on the ratio on the net section area to the gross section area (A_n/A_g) and the yield-tensile ratio. The significance of these parameters has been discussed by Kato (1990), Ohashi et al (1990), and Kulak et al (1987). If the member is proportioned so that it yields in the gross section before the tensile strength in the gross section is reached, it will behave much like the welded member. However, if the product of the net section area times the tensile strength of the material is less than the gross section area times the yield strength, the member deformation will be limited. Some experimental correction to the tensile strength must be made, since the stress at which a member with bolt holes ruptures is somewhat greater than the tensile strength measured in a standard test.

Thus, the effect of the yield-tensile ratio depends on the design philosophy used. If it is desirable for gross section yielding to control, greater yield-tensile ratios require greater A_n/A_g ratios. But, most specifications do not require that gross section yielding control the design. In this case the yield-tensile ratio may not have a significant effect on the member design. Apparently, adequate ductility for most applications is provided by bearing deformations at bolt holes and shear deformations in bolts. However, more stringent demands may prevail for some seismic applications. The fracture of diagonal braces through end joints of many buildings in Japan in earthquakes in 1968 and 1978, which had material with an "unusually high" yield-tensile ratio, led to seismic code revisions.

Fatigue

Based on fatigue tests of details fabricated from steels with strength levels up through that of A514 steel, present fatigue design criteria for high cycle fatigue

(20,000 cycles or greater) use stress range and type of detail as the primary variables, not the strength of the steel. Dexter et al (1993b) conducted fatigue tests on over 170 large-scale welded details of HSLA-80 steel (yield-tensile ratio of about 0.90) representative of double-hull ship construction. The lower confidence limits of the S-N curves were not significantly different than those for similar welded details in traditional bridge steels. Thus, there is no reason to suspect that the yield-tensile ratio is a factor in the fatigue of fabricated members.

Low cycle fatigue is a consideration where a detail may be subjected to repeated cycles above the yield point, such as in earthquake loadings. Nagayasu et al (1991) conducted tests on a beam-to-column connection in which the specimens were cycled well into the inelastic range (strain amplitudes from 1 to 4 percent). The cycles to failure ranged from 74 to 7 and were judged adequate. The material with the greatest yield-tensile ratio (0.80) and tensile strength (85 ksi) withstood about twice as many cycles as the other material (0.75 and 75 ksi). Thus, there is no indication that tensile strength or yield-tensile ratio has any negative effect in low cycle fatigue, at least within the limitations of this data.

Toughness

One of the essential assumptions in structural design is that each member and each connection have a capacity for rotation or deformation adequate to ensure that its intended function can be fulfilled. For example, if a structure is designed to develop strength as a mechanism, the rotation at a hinge must not be terminated prematurely by local buckling or by fracture. Thus, in the development of high performance steels, it is essential that adequate notch toughness and fracture ductility be provided, both in the parent material and in weldments. This is a critical part of the development process.

The notch toughness required depends on environmental conditions, loading characteristics, and the fabrication details employed. Information on notch toughness of new steels developed in Japan was provided by Ohashi et al (1990). For HT80 steel (100 ksi minimum yield strength, yield-tensile ratio of 0.85 maximum) the target minimum V-notch toughness (transverse specimens) was 4.8 kgf·m @ -15 ^{0}C, and test values ranged from 11.9 to 20.2 kgf·m @ -15 ^{0}C. For HT50 steel, 47 ksi minimum yield strength, yield-tensile ratio of 0.80 maximum) target values were not given but test values (average of three specimens) were 29.1 (longitudinal) and 29.7 (transverse).

Ductility Demand in Cold-Formed Steel Structures

Early interest in utilizing sheet steels with high yield-tensile ratios led to investigations into ductility requirements by Winter and his associates under AISI sponsorship. The work involved tests of sheets with holes and with bolted and welded connections conducted by Dhalla et al (1971), as well as investigations on formability and inelastic finite element analyses of stress concentrations. Specific suggestions for ductility requirements were made by Dhalla and Winter (1974). Based on these investigations, the following requirements were adopted in national specifications for the design of cold-formed members: a yield-tensile ratio of 1/1.08 = 0.93 and a minimum elongation of 10 percent in a 2 in. gage length or 7 percent in a 8 in. gage length. These criteria have served well for many years.

More recently, based on a review of Winter's work and new tests by Macadam et al (1988), somewhat more liberal criteria were adopted for members such as purlins and girts. For these members, criteria adopted in the specifications eliminated the yield-tensile requirement and instead required a local elongation of at least 20 percent in a 1/2 in. gage length across the fracture and a uniform elongation outside the fracture of at least 3 percent. Davies and Cowen (1994) have also called for the elimination of yield-tensile requirements in cold-formed steel structures.

Pressure Vessels

Research has shown that, ignoring effects of strain concentrations, pressure vessels of higher-strength steels with higher yield-tensile ratios tend to burst at a higher percentage of their tensile strength than vessels of lower strength steels. For pressure vessels with notches, Royer and Rolfe (1974) have shown that the reduction in burst pressure is directly proportional to the reduction in wall thickness at the notch, provided the notch depth does not exceed about 25 percent of the vessel wall. The tests included material with a yield-tensile ratio up to 0.93. For deeper notches, the data are inconclusive. Langer (1970) showed that a material with a low strain-hardening exponent, such as a higher strength steel with a high yield-tensile ratio, will tend to have a higher strain concentration in the presence of a notch. In a steel with a higher exponent, the material surrounding the notch tends to control the strain in the plastified zone.

Yield-Tensile Ratio vs. Yield-Tensile Difference

Some have questioned whether structural behavior can be better viewed in terms of the difference between the tensile strength and the yield strength rather than the yield-tensile ratio. To reflect on this matter, consider the paper by Kuwamura and Kato (1989). They addressed the behavior of a beam-column, a member subjected to an axial load and an increasing end moment similar to that shown in Figure 1.10(a).

The rotation capacity depends on the length of the plastic region developed at the moment end of the beam as discussed earlier. Using an idealized stress-strain curve, Kuwamura and Kato give the following expression for the plastic length fraction, α, where the beam length is L and the length of the plastic region is αL:

$$\alpha = 1-Y(C_y/C_u) \quad \text{1.17}$$

Y is the yield-tensile ratio. In the absence of axial load, $\alpha = 1-Y$. Cy and Cu are modification factors for the axial load effect; they contain terms for the cross section geometry, the ratio of the axial stress to the yield strength (p_y), and the axial stress to tensile strength. Equation 1.17 can be rewritten in terms of the yield strength (σ_y) and the difference between the tensile strength and the yield strength ($\Delta\sigma = \sigma_u - \sigma_y$) as follows:

$$\alpha = 1-(C_y/C_u)(1+\Delta\sigma/\sigma_y)^{-1} \quad \text{1.18}$$

This allows the plastic length fraction to be portrayed in terms of either the yield-tensile difference or the yield-tensile ratio. Such calculations were made for an arbitrary I-shaped cross section (12 by 1 in. flanges and 10 by 0.60 in. web) and axial load ratio of p_y=0.20. The calculations were made for $\Delta\sigma$ values of 5 to 20 ksi and σ_y values of 50 and 100 ksi; these selections correspond to a Y range of 0.71 to 0.95.

The results are shown in Figure 1.24. As indicated, the plastic length decreases with either decreasing yield-tensile difference or increasing yield-tensile ratio. The effect may be viewed in either manner, but the yield-tensile ratio is the more fundamental variable.

Conclusions

Because the shape of the stress-strain curve is different for different steels several related effects may be implied when reference is made to the effect of the yield-tensile ratio. Important properties may include the length of the yield plateau (if one exists), the slope of the stress-strain curve in the inelastic range, local elongation (elongation in the necking region), and uniform elongation (total elongation less local elongation). In general, steels with different yield-tensile ratios have different values for these properties, and in experimental work, it is virtually impossible to isolate individual effects. Thus, the yield-tensile ratio tends to serve as an umbrella for related properties. The yield-tensile ratio appears to be a more fundamental variable than the yield-tensile difference. Inelastic numerical or finite element analysis, combined with structural testing verification, can be used to predict the structural behavior that can be expected for a steel with a defined stress-strain curve.

Studies on bending members indicate that the rotation capacity tends to decrease with increasing yield-tensile ratio, and that it can affect the final failure mode.

However, all applications do not require the same level of rotation capacity, and it is not necessary to maximize the rotation capacity for each one. Instead, if required levels are established, analytical and experimental studies can determine the maximum yield-tensile ratio that would provide that capacity. Also the mode of failure can be controlled. For example, flange width-to-thickness ratios and web depth-to-thickness ratios can be selected that will allow a member to reach its required strength and rotation level, but that will ensure inelastic local buckling before reaching the strain required for tension flange rupture.

The strength of columns with yield-tensile ratios up through about 0.95, based on measured properties, can be reasonably predicted with the present relationships that are used for other steels. The material stress-strain curve must be reasonably linear, with the proportional limit in a tension test at least 85 percent of the yield strength. The same conclusion holds for local buckling strength for stresses up to the yield point. However, if a short compression member of given proportions is compressed beyond the yield point level, the maximum average stress that can be reached in proportion to its yield strength, tends to increase with decreasing yield-tensile ratios of the steel. Also, when ultimate load is controlled by inelastic local buckling, beams and columns of steels with low yield-tensile ratios will withstand larger deformations. However, if such post-yield behavior is needed, it can be realized by decreasing width-to-thickness ratios of flange and web elements.

For bolted tension members, if it is desirable for gross section yielding to control rather than net section rupture, greater yield-tensile ratios require greater ratios of net-to-gross section area. However, most specifications do not require that gross section yielding control the design. Apparently, adequate ductility for most applications is provided by bearing deformations at bolt holes and shear deformations in bolts. However, the fracture of diagonal braces through end joints of many buildings in Japan in earthquakes in 1968 and 1978, which had material with an "unusually high" yield-tensile ratio, led to seismic code revisions. Regarding the effect of joint length on strength, the effect is similar for steels with different yield-tensile ratios.

The yield-tensile ratio is a not a significant factor in determining the fatigue strength of fabricated members.

One of the essential assumptions in structural design is that each member and each connection have a capacity for rotation or deformation adequate to ensure that its intended function can be fulfilled. For example, if a structure is designed to develop strength as a mechanism, the rotation at a hinge must not be terminated prematurely by local buckling or by fracture. Thus, in the development of high performance steels, it is essential that adequate notch toughness and fracture ductility be provided, both in the parent material and in weldments. This is a critical part of the development process. The notch toughness required depends on environmental conditions, loading characteristics, and the fabrication details employed.

Most of the recent work on heavy structures related to the effect of the yield-tensile ratio has been done in Japan. The work was largely directed at developing framed structures with improved resistance to severe earthquakes. For such severe applications, members must demonstrate the capability to deform well into the inelastic range. The work reported indicates that a low yield-tensile ratio enhances this capability. However, most structures do not require such a high deformation capability.

Cold-formed structural members fabricated from steels with yield-tensile ratios up to 0.93 have been used successfully for many years. Indeed, members with yield-tensile ratios up to 1.00 have performed adequately in tests and used for a limited range of applications.

Research has shown that, ignoring effects of strain concentrations, pressure vessels of higher-strength steels with higher yield-tensile ratios tend to burst at a higher percentage of their tensile strength than vessels of lower strength steels. For pressure vessels with notches, Royer and Rolfe (1974) have shown that the reduction in burst pressure is directly proportional to the reduction in wall thickness at the notch, provided the notch depth does not exceed about 25 percent of the vessel wall. The tests included material with a yield-tensile ratio up to 0.93.

The highest strength structural steel, A514, has performed well in bridge and building applications, even though the yield-tensile ratio based on measured properties ranges up to 0.93 or 0.95. Thus, provided a reasonable level of total ductility is maintained, such as the 16 to 18 percent (depending on thickness) minimum elongation in 2 in. specified for A514, steels with yield-tensile ratios up to about that level can be effectively utilized in most design applications. As an example, the large Honshu-Shikoku bridge project in Japan, which involves three separate routes and numerous bridges, is apparently using 90 and 100 ksi yield point steels with a yield-tensile ratio of 0.90.

Steels with a yield strength greater than about 70 ksi have been designed as non-compact members. This penalizes their design efficiency by about 10 percent. Additional work should be undertaken to define compactness in this strength range.

Japan has developed steels for seismic applications that provide a maximum yield-tensile ratio of 0.80 for yield strengths from 50 to 65 ksi, and 0.85 for a 100 ksi yield strength steel. However, the studies reviewed did not specifically show that such ratios were the highest values that might be acceptable for the application. Additional studies would be desirable to set more precise limits.

Part 3 - Application Considerations

Answers to Specific Questions

1. How is the ability to yield locally and redistribute stresses under static loads affected by the yield-tensile ratio?

In the most common structural connection, a bolted joint, the stress distribution is seldom uniform. The material around the most highly stressed bolt must yield to allow redistribution of stresses to other bolts. Similar situations occur in welded joints.

Dhalla and Winter (1974) showed that the most important factor is the local ductility and that an elongation of 20 percent in a 1/2 in. gage length (including the necking region) is sufficient to ensure ductile behavior. On the other hand, Kato (1990) showed that the elongation capacity does increase with a decreasing yield-tensile ratio.

2. Under dynamic loads, how is the ability to yield locally and redistribute stresses affected by the yield-tensile ratio?

Connections and members often have fabrication imperfections that cause stress concentrations. Examples include weld tabs left in place or gouges left unrepaired. Such imperfections may prove harmless under static loads, but trigger crack propagation under dynamic loads.

No data were found on structural impact tests related to the yield-tensile ratio. However, it appears that the key factor is the fracture toughness or V-notch impact toughness of the steel. The Japanese did address toughness in the development of new steels as reported by Ohashi et al (1990). The toughness should not be viewed as a consequence of the strength level or the yield-tensile ratio, but as a consequence of the steel chemistry and processing employed to develop the grade. The notch toughness required depends on environmental conditions, loading characteristics, and the fabrication details employed.

3. What is the effect of the lower strain hardening on behavior in compression, specifically local-buckling and column buckling?

Resistance to buckling is important in both flexural members and compression members. Various theories have been developed to relate inelastic properties, such as the strain-hardening exponent or the secant modulus, to buckling in the inelastic range. For efficient design, compression members are generally designed to buckle in the inelastic range.

Based on the work of Dexter et al (1993a) and earlier studies, the strength of columns with yield-tensile ratios up through about 0.95, based on measured properties, can be reasonably predicted with the present relationships that are used for other steels. The material stress-strain curve must be reasonably linear, with the proportional limit in a tension test at least 85 percent of the yield strength. The same conclusion holds for local buckling strength for stresses up to the yield point. However, as reported by Kuwamura and Kato (1989), if a short compression member of given proportions is compressed beyond the yield point level, the maximum average stress that can be reached in proportion to its yield strength, tends to increase with decreasing yield-tensile ratios of the steel. Also, when ultimate load is controlled by inelastic local buckling, beams and columns of steels with low yield-tensile ratios will withstand larger deformations. However, if such post-yield behavior is needed, it can be realized by decreasing width-to-thickness ratios of flange and web elements.

4. What is the effect of the lower strain hardening on the moment-rotation behavior?

Recent efforts to develop more efficient design methods for beams and girders has been related to defining moment-rotation curves that describe behavior up through maximum load. This involves considerations of behavior under both tension and compression.

Studies on bending members by Kato (1990), Ohashi et al (1990), and Mc Dermott (1969b, 1970) indicate that the rotation capacity tends to decrease with increasing yield-tensile ratio, and that it can affect the final failure mode. The general effect of the yield-tensile ratio on the behavior of a beam can be considered as follows. In a beam with a moment gradient, if the yield strength is less than the tensile strength, the plastic region of a beam can extend over some length of the beam as the bending moment at the critical section increases above the plastic moment by virtue of strain hardening. The spread of the plastic region contributes greatly to the rotation capacity. On the other hand, if the yield strength and tensile strength are equal (yield-tensile ratio of 1.0), there can be no extension of the plastic region because the tension flange can rapidly reach its ultimate strain and rupture as the plastic moment is reached. Thus, the inelastic rotation capacity of a beam with a moment gradient approaches zero as the yield-tensile ratio approaches 1.0.

5. What are the consequences of using structural members with a significantly higher strength than anticipated?

In earthquake design, it is desirable for the beams to yield before the columns. However, if the beam material is significantly over-strength, this may not occur. This type of behavior may be important in other applications.

If the yield strength of the beams is likely to be much greater than specified, the columns must be overdesigned to account for this possibility. Ohashi et al (1990) showed that if the coefficient of variation of the beams is 10 percent, the columns must be overdesigned by a factor of 2.0; reducing the COV to 5 percent cuts the required overdesign factor to 1.4. They concluded that reducing the yield point deviation is of utmost importance in enhancing seismic resistance of building structures.

6. What are the consequences of designing with a stress-strain curve with very limited strain hardening?

Inelastic analysis of structures with computer programs to determine ultimate strength or non-linear load-deflection response, requires the input of a stress-strain curve. Even hand calculations for plastic design anticipate a curve with a shape similar to that for traditional structural steels. If steels are developed that have very high yield-tensile ratios, will this have an adverse effect on structural behavior?

The consequences of a high yield-tensile ratio are reflected in the answers to the preceding questions on behavior. In regard to predicting behavior, no particular problems were reported related to the shape of the curve. Thus, the traditional analytical methods and numerical methods can be applied with the new steels.

Simple Illustrative Models

In this section, simple illustrative models are offered to show trends in behavior that are influenced by the yield-tensile ratio.

Local Yielding and Stress Redistribution

Consider two steels having the stress-strain curves depicted in Figure 2.1a. The yield strain is ε_y and the maximum strain reached before unloading is ε_u. Assume such steels are used for a tension strap with a bolt hole at midwidth as shown in Figure 2.1b. The stress-strain distribution across the net section is non-linear with a maximum concentration at the edge of the hole. Assuming a linear representation of strain, Figure 2.1c illustrates the strain at maximum elongation for steel A. As the load increases, the strain at the edge of the hole can increase to ε_u, but as soon as the strain at the edge of the strap reaches ε_y, the maximum load is reached because there can be no increase in stress. Figure 2.1d illustrates the strain at maximum load for steel B. In this case the strain can approach ε_u across the full net width as the stress increases because of strain hardening. As shown by the expressions at the bottom Figure 2.1, the inelastic deflection of the strap of steel B would approach two times that of the strap of steel A, and the maximum load ratio would approach 2/(1 + Y), where Y is the yield-tensile ratio, σ_y/σ_u.

Figure 2.2 depicts dimensionless load-deflection curves for a tension strap with yield-tensile ratios of 0.75, 0.95, and 1.00. Because the load increases are small, the curves would be fairly flat as the deflection ratio increases from 1.0 for Y = 1 to 2.0 at lower Y values. The extremities of the plots are dashed to indicate approached values. The figure illustrates the trend that for a tension member, a lower yield-tensile ratio tends to increase the deflection (elongation) significantly and to increase the maximum load slightly.

Moment-Rotation Behavior

Consider two steels having the stress-strain curves depicted in Figure 2.3a. The yield strain is ε_y and the maximum strain reached before unloading is ε_u. Assume such steels are used for an end-loaded cantilever beam as shown in Figure 2.3b. The resulting moment-curvature relationships are shown in Figures 2.3c and 2.3d for steels A and B respectively.

With steel A, the moment at section 1 increases until it reaches the yield moment, M_y, at a curvature, ϕ_y; then increases to the plastic moment, M_p, at a curvature, ϕ_p. At the same time, the moment at section 2 also increases until it reaches the yield moment, M_y, at a curvature, ϕ_y; but it can increase no more because the moment at section 1 has reached its maximum value. The distance between sections 1 and 2 is designated L_{12}, Figure 2.3b.

With steel B, the moment at section 1 increases until it reaches the yield moment, M_y, at a curvature, ϕ_y; then increases to the plastic moment, M_p, at a curvature, ϕ_p; then increases to the maximum moment, M_m, at a curvature, ϕ_m, through strain hardening. At the same time, the moment at section 2 also iincreases until it reaches the yield moment, M_y, at a curvature, ϕ_y; then increases to the plastic moment, M_p, at a curvature, ϕ_p. Meanwhile, the moment at section 3 increases until it reaches the yield moment, M_y, at a curvature, ϕ_y. The distance between sections 1 and 2 is designated L_{12}, and the distance between sections 2 and 3 is designated L_{23}, Figure 2.3b.

The end rotation for beams of steels A and B is given by the relationships for θ_A and θ_B at the bottom of Figure 2.3. The derivation of these expressions is shown in Figure 2.4 and, in Figure 2.5, some simplifications are made to calculate the rotation ratio, θ_A/θ_B, in terms of the yield-tensile ratio, Y. The solid line in Figure 2.6 shows the resultant plot of the rotation ratio versus Y. However, the solid line plot should be viewed as an upper limit because it is not reasonable to expect the entire section to strain harden to reach the maximum stress σ_u. A more reasonable expectation would be to strain harden to reach a maximum stress $(\sigma_u + \sigma_y)/2$. This relationship is shown by the dashed line in Figure 2.6, determined as follows. The dimensionless maximum stress is $(1/\sigma_u)(\sigma_u + \sigma_y)/2 = (Y + 1)/2$ instead of Y. For an assumed yield-tensile ratio Y, calculate the effective ratio $(Y + 1)/2$, and determine θ_A/θ_B for that value from the solid line in Figure 2.6; plot the result at the assumed Y value.

Figure 2.6 illustrates that for a cantilever beam, a lower yield-tensile ratio tends to increase the end rotation significantly. For example, considering a beam from a steel with a yield-tensile ratio of 0.85 versus one from a steel with a ratio of 0.95, the dashed line plot indicates that the end rotation at maximum load would increase by about 40 percent (1.8/1.3 = 1.4). Of course, these relationships assume that the member is proportioned and braced so that premature failure by local or lateral buckling does not occur.

Local Buckling Behavior

Although a detailed model of the behavior is not detailed here, the general behavior discussed above for the cantilever beam can be related to local buckling. For example, consider a short compression member comprised of plate elements that fails by local buckling. As each plate element deflects laterally, it develops bending moments along its length that vary in proportion to the lateral deflection. The rotation that this element with a moment gradient can withstand before failure increases as its yield-tensile ratio decreases. This explains the greater inelastic ductility observed in compression tests of stub columns of steels with lower yield-tensile ratios.

Consequence of Overstrength Beam in Seismic Design

Columns in moment resistant frames are subjected to both axial loads and bending moments. Figure 2.7 shows an elastic moment diagram at a typical exterior beam-to-column intersection for such a frame. Codes such as the AISC Seismic Provisions [AISC (1992)] require that the members be proportioned so that the beams yield before the columns. This is provided by the relationship from AISC shown in Figure 2.7. It is based on specified minimum yield strengths on the assumption that the variation in yield strength above the specified minimum is similar for beams and columns. However, if beams have an actual yield strength that exceeds the minimum by an amount that is typically greater than that of the columns, it should be considered in proportioning members. Thus, overstrength bending members can lead to the requirement of overdesigning columns to compensate.

Impact on AASHTO Bridge Specifications

As discussed below, the studies reviewed suggest that new high performance steels can probably be included in the AASHTO Bridge Specifications [AASHTO (1993)] with little modification. This will depend to some extent on the characteristics of the steel and the design treatment that is desired. Any new steel would likely be more readily acceptable if the yield-tensile ratio did not exceed that of A514 steel, 0.91 based on specified properties and about 0.95 based on measured properties. Also it should have a ductility no less than that of A514 steel, 16 percent in 2 in. These characteristics are generally assumed in the following discussion.

In AASHTO Specifications for Load and Resistance Factor Design (LRFD), all members and connections must satify the following equation:

$$h \Sigma g_i Q_i \leq f R_n \tag{2.1}$$

where g_i is a load factor, Q_i is a force effect, f is a resistance factor, and R_n is the nominal resistance. The factor h is a multiple of three factors, one relating to ductility, h_D, one relating to redundancy, and one relating to operational importance. The factors that could potentially be affected include the ductility factor h_D and the resistance factor f.

The ductility factor is specified as 1.05 for non-ductile components and connections and 0.95 for ductile components and connections. Ductile behavior is characterized by AASHTO as significant (not further defined) inelastic deformation before any loss of load carrying capacity occurs, and must be established by test. There should be no difficulty in demonstrating such behavior.

The resistance factor is a statistically based multiplier applied to the nominal resistance to account for variations in determining the strength of the member or connection. It includes such effects as uncertainties in the theory used to define the strength of the member and variations in material properties and member dimensions. Meaningful statistical data on variations in material properties such as yield strength and tensile strength can be developed after production quantities are available. However, the literature reviewed did not indicate any difficulties in this regard. Thus, there is no reason at this point to suspect that the resistance factors for members or connections of the new steels would be less than the presently specified factors.

As indicated above, acceptance of new steels also depends on the design treatment desired. At present AASHTO limits the use of compact sections to steels with a specified minimum yield point of 70 ksi or less. Also, for steels with a specified mimnimum yield strength that exceeds 50 ksi, certain moment redistribution provisions and inelastic analysis procedures can not be used. To overcome these limitations, structural research studies would be required for new steels that exceed these strength limits.

Non-compact bending members can be treated with present criteria. Additional investigations would be required to define provisions for compact members or to take advantage of moment redistribution provisions or inelastic analysis procedures. A primary area of investigation would be inelastic local buckling studies to define member proportions that would allow increasing levels of performance as discussed subsequently under "Suggested Future Work." Present provisions for compression members, tension members, and connections should hold without modification. However, limited confirmatory structural tests should be conducted to demonstrate and verify behavior. The tests should demonstrate that the overall ductility needed is provided.

Suggested Future Work

As previously discussed the level of structural ductility required depends upon the application and the structural behavior assumed in the design method used. The general approach in planning future work should be to (1.) define structural ductility requirements and (2.) perform analytical and experimental studies to ensure that a proposed new steel can provide that ductility. The work should be targeted at specific steels with defined tensile properties and defined stress-strain curves. It is presupposed that adequate notch toughness and weldability has been established. Research required for various types of structural members can be summarized as follows.

Bending Members. A primary area of investigation here would be inelastic local buckling studies to define member proportions that would allow increasing levels of performance. It would be directed at beams but, depending on the application, some work on beam-columns should be included as well. For I-shaped cross sections, primary variables include the flange width-to-thickness ratio, the web depth-to-thickness ratio, and unbraced length. There is a gain in design efficiency if these proportions are defined so that the member may be considered compact, that is, one which can withstand rotation sufficient to reach and maintain the plastic moment. As discussed previously, steels with a yield strength greater than about 70 ksi have been designed as non-compact members. This penalizes design efficiency by about 10 percent. To take full advantage of the strength provided by high performance steels in this strength range, inelastic member behavior should be well understood. With the behavior of compressive elements defined, the structural ductility as limited by the yield-tensile ratio, such as the spread of the plastic length region, can be quantified. This could best be done by a combination of inelastic analytical work and large scale structural testing. However, it must not be overlooked that high levels of inelastic behavior are not required for all applications. The work should be directed at determining if a particular defined level of need can be achieved for a particular steel.

Compression Members. The stress-strain curves for the newer steels do not appear to be significantly more rounded than those of present steels. Therefore, it is not anticipated that significant work will be required for columns. However, stub-type compression tests of cross section elements should be conducted as part of the inelastic local buckling studies for bending behavior.

Tension Members. Some work should be done to confirm behavior in bolted tension splices. This would include tension tests of plates with bolt holes to determine average net section stress levels at ultimate strength, tests of short joints to determine slip coefficients, and possibly limited tests of long bolted joints to demonstrate behavior. However, it is likely that present design criteria in this area is adequate for the new steels.

Connections. Connections representative of the targeted application should be tested to demonstrate that the expected behavior can be achieved. Static type loadings should suffice but, depending on the application, some low cycle fatigue tests may be desirable as well.

Fatigue. At this time, there does not appear to be a significant need for fatigue testing in view of the information that has been previously developed.

Through-Thickness Properties. In some applications, the strength and ductility in the through thickness direction is very important. One example is a beam-to-column connection, where tensile forces from the beam flange are resisted by through-thickness strains in the column flange. If a new steel is to be used for such applications, it should be ascertained through coupon-type tests and larger tests that the processing used develops adequate properties.

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Figure Sources

Figure 0.1 - 0.2. Ohashi (1990)

Figures 1.1 - 1.7. Kato (1990)

Figure 1.8 - Kuwamura and Kato (1989)

Figure 1.9 - Nagayasu et al (1991)

Figures 1.10 - 1.12 Ohashi et al (1990)

Figure 1.13 - Otani (1992)

Figures 1.14 - 1.15 McDermott (1969b)

Figure 1.16 - Dhalla et al (1971)

Figures 1.17 - 1.18 Macadam et al (1988)

Figures 1.19 - 1.20 Kulak et al (1987)

Figures 1.21 - 1.22 Royer and Rolfe (1974)

Figure 1.23 - Langer (1971)

Figure Captions

Figure 0.1. Relationship of yield-tensile ratio to tensile strength.

Figure 0.2. Stress-strain curves reflecting conventional and new processes.

Figure 1.1. Plate with reduced section and stress-strain curve definition.

Figure 1.2. Effect of yield-tensile ratio on elongation capacity of tension member with reduced section.

Figure 1.3. Effect of yield-tensile ratio on elongation capacity of tension member with central hole.

Figure 1.4. Fundamental behavior of cantilever beam.

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Figure 1.9. Low cycle fatigue tests with grade 53 and grade 60 steels.

(a) Specimens tested.

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Figure 1.12. Effect of yield-tensile ratio on behavior of cantilever beam.

Figure 1.13. Relation between yield-tensile ratio and uniform elongation.

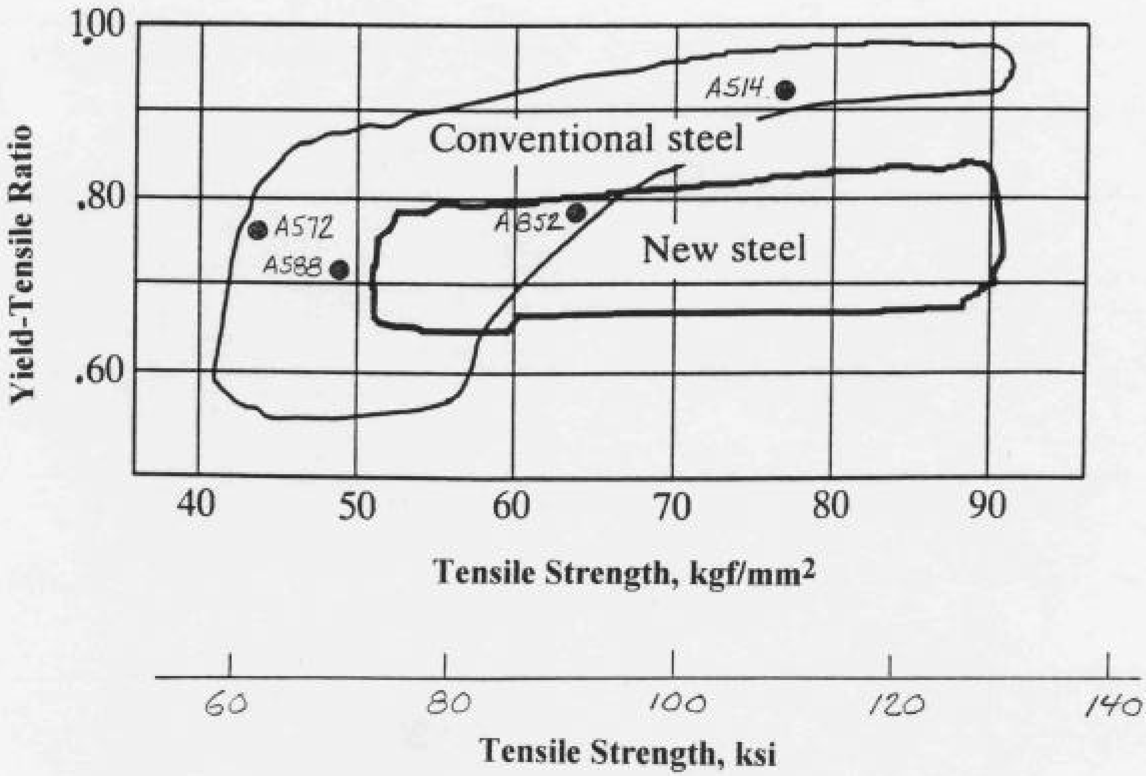
Figure 2.3. Effect of yield-tensile ratio on moment-rotation behavior of cantilever beam. (a) Stress-strain curves. (b) Cantilever beam with end load. (c) Moment-curvature plots for sections 1 and 2 for steel A. (d) Moment-curvature plots for sections 1, 2 and 3 for steel B.

Figure 2.4. Derivation of relationships for moment-rotation behavior of cantilever beam.

Figure 2.5. Simplification of relationships for moment-rotation behavior of cantilever beam.

Figure 2.6. Illustration of effect of yield-tensile ratio on moment-rotation behavior of cantilever beam.

Figure 2.7. Elastic moment diagram at beam-to-column intersection for moment frame subjected to seismic loading.



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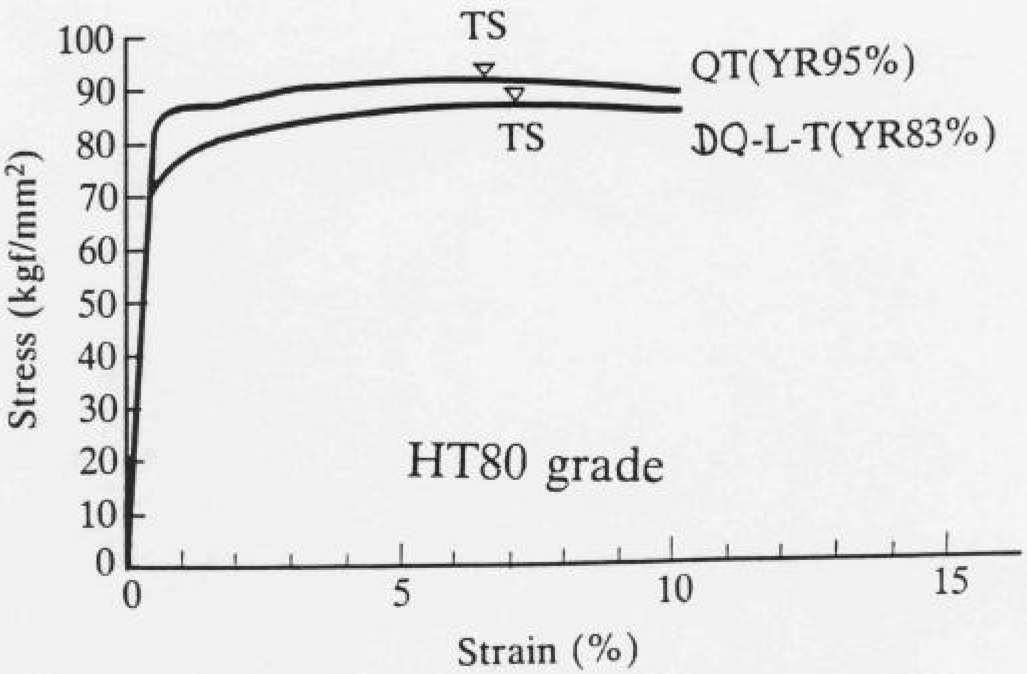
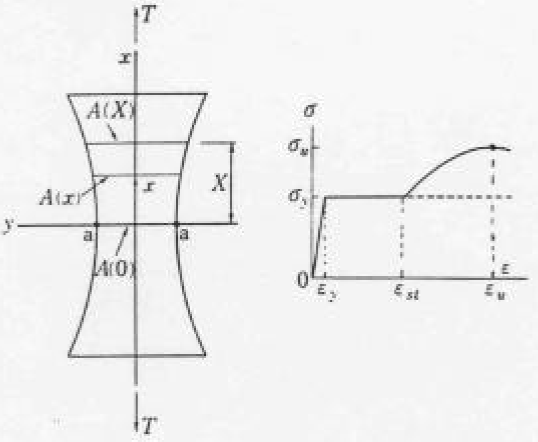


Figure 0.2. Stress-strain curves reflecting conventional and new processes.



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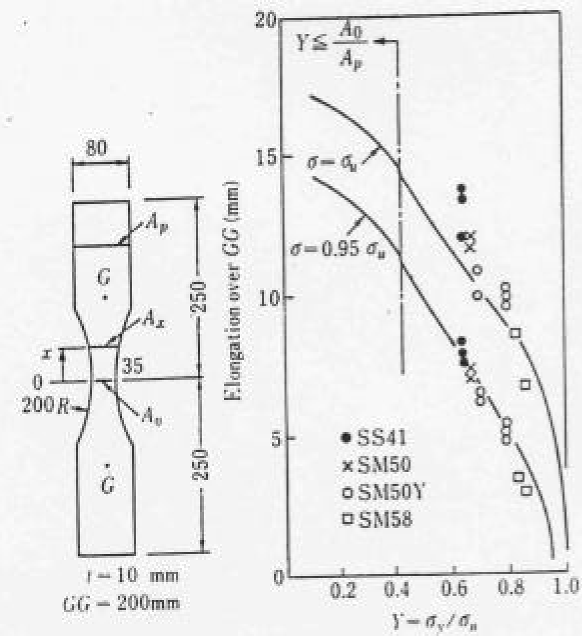


Figure 1.2. Effect of yield-tensile ratio on elongation capacity of tension member with reduced section.

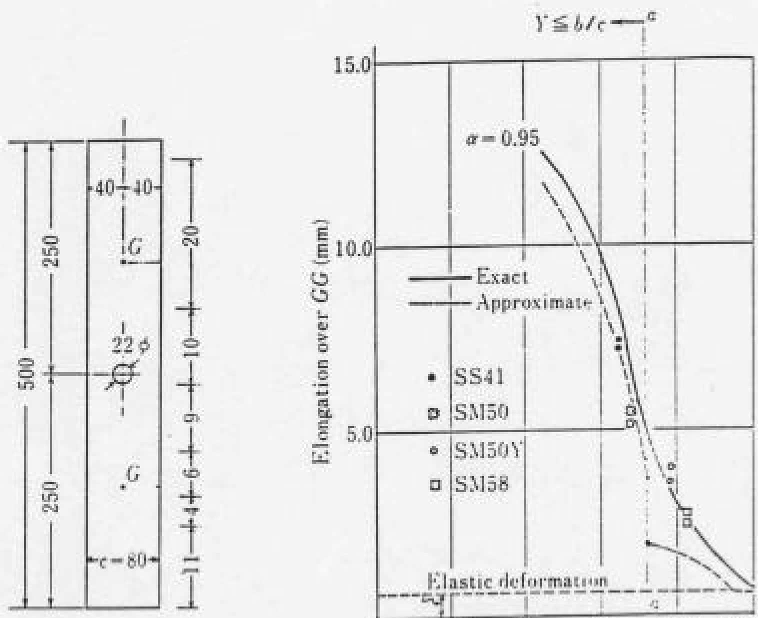
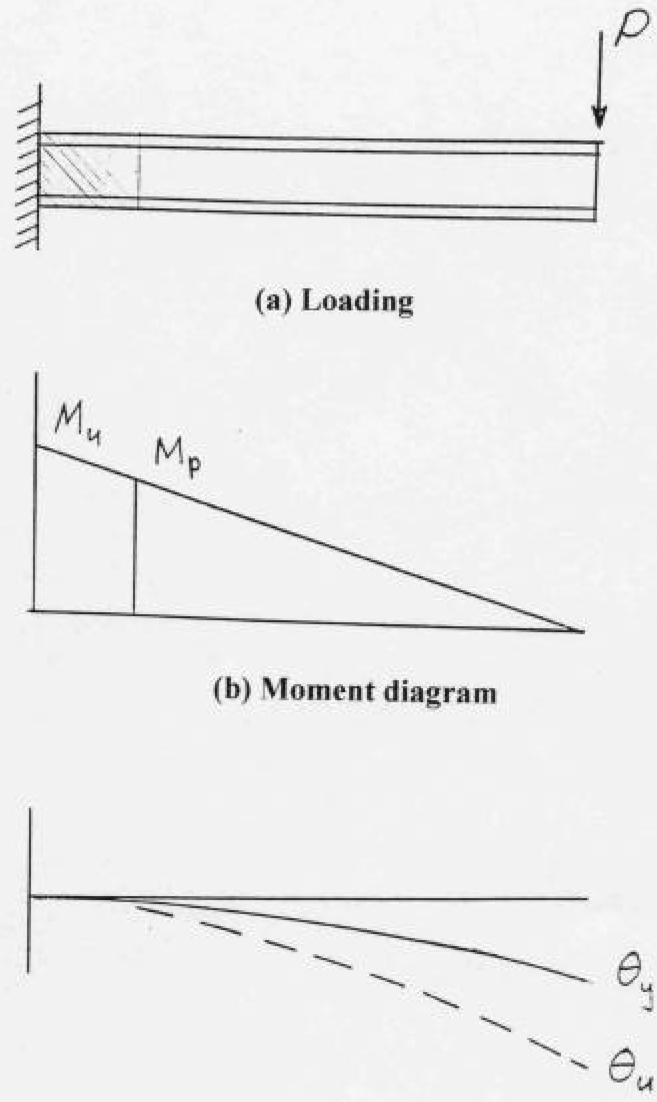


Figure 1.3. Effect of yield-tensile ratio on elongation capacity of tension member with central hole.



(a) Loading

(b) Moment diagram

(c) Rotation

Figure 1.4. Fundamental behavior of cantilever beam.

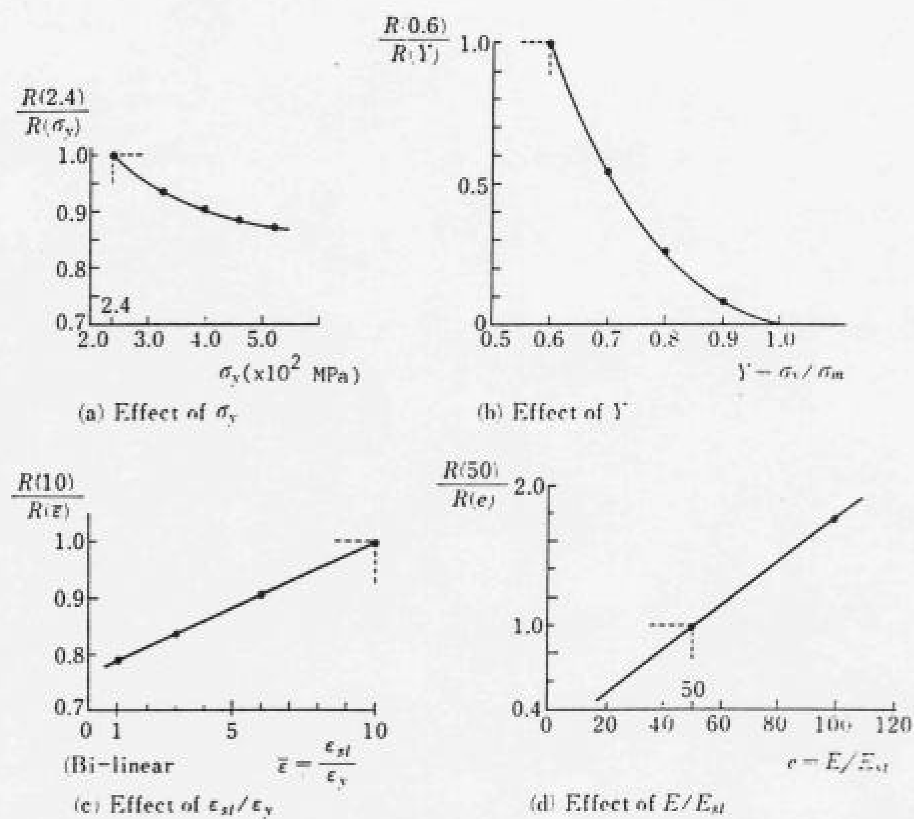


Figure 1.5. Effect of stress-strain curve on rotation capacity.

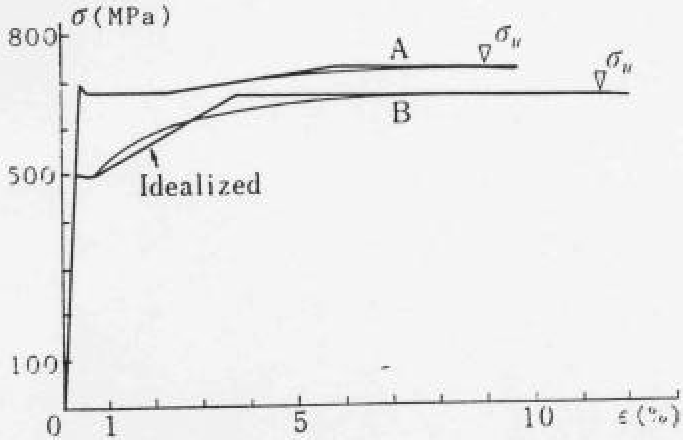


Figure 1.6. Stress-strain curves for steels in evaluation of rotation capacity.

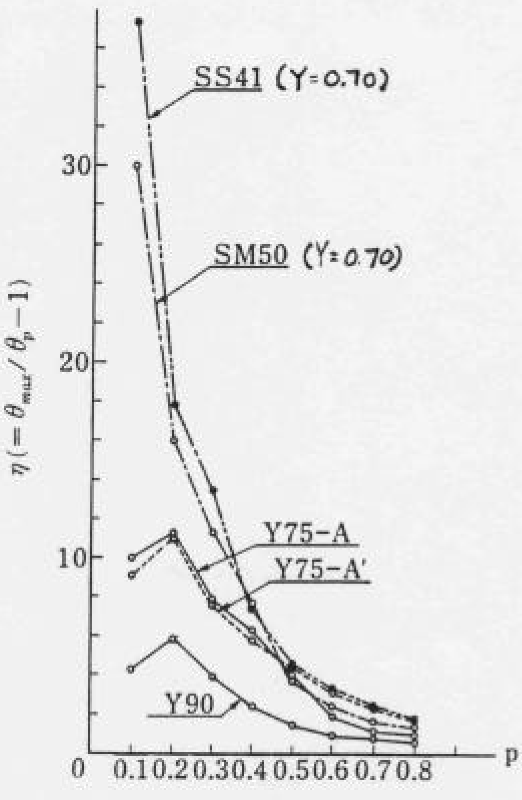
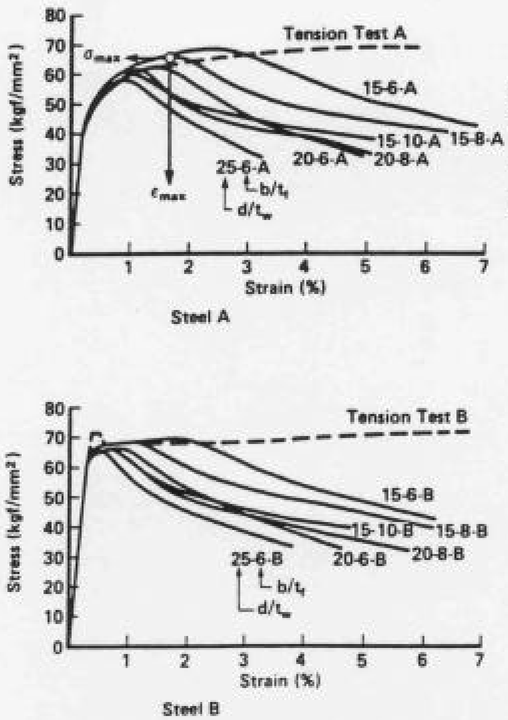
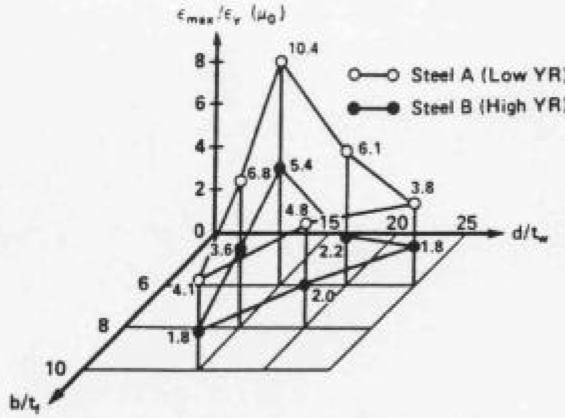


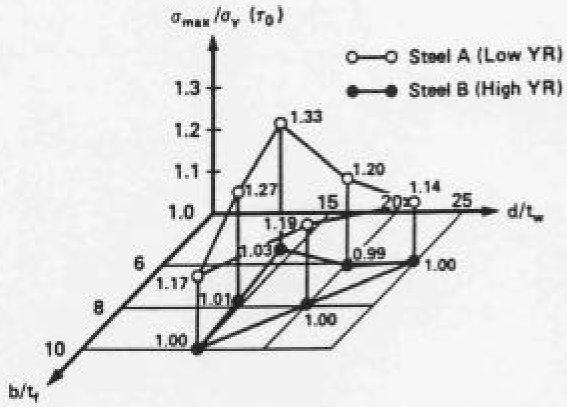
Figure 1.7. Rotation capacity for beam-columns of steels with yield-tensile ratios from 0.70 to 0.90.



Stress-Strain Curves in Compression Obtained from Stub-Column Testing.

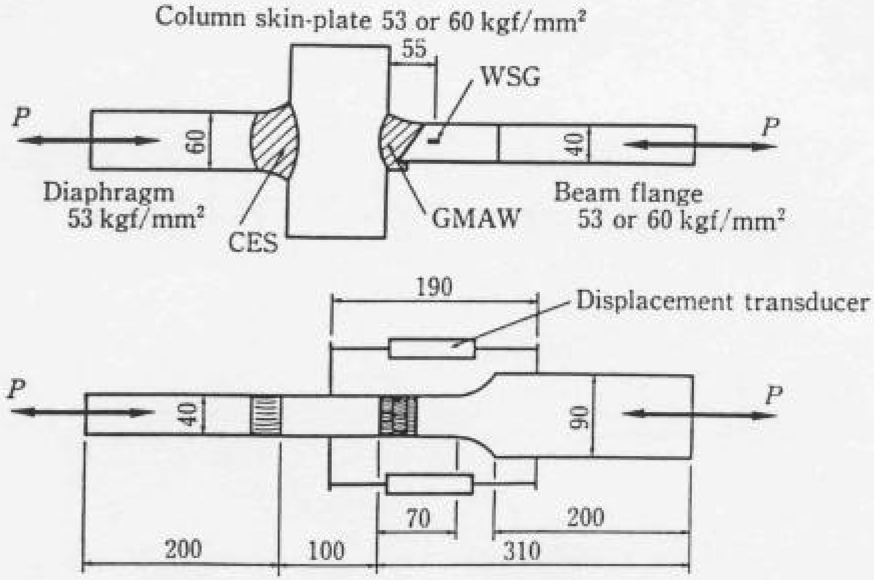


Strain Ductility of H-Shaped Stub-Columns of High-Strength Steels.

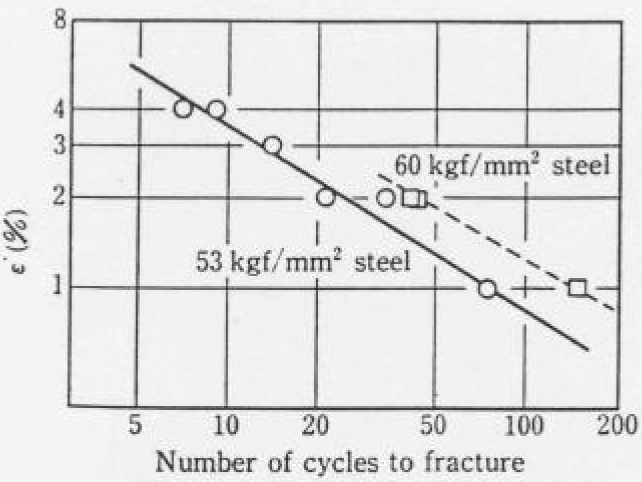


Stress Increase Rate of H-Shaped Stub-Columns of High-Strength Steels.

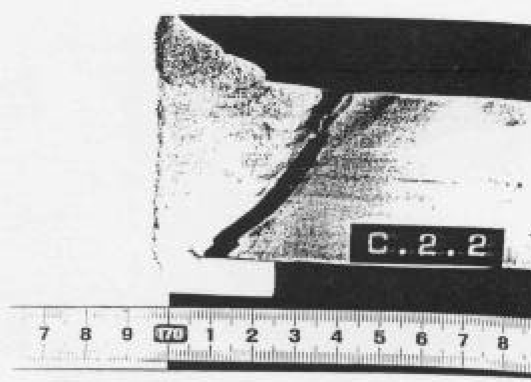
Figure 1.8. Relation of local buckling characteristics to yield-tensile ratio.



(a) Specimens tested.



(b) Test results.



(c) Specimen after test.

Figure 1.9. Low cycle fatigue tests with grade 53 and grade 60 steels.

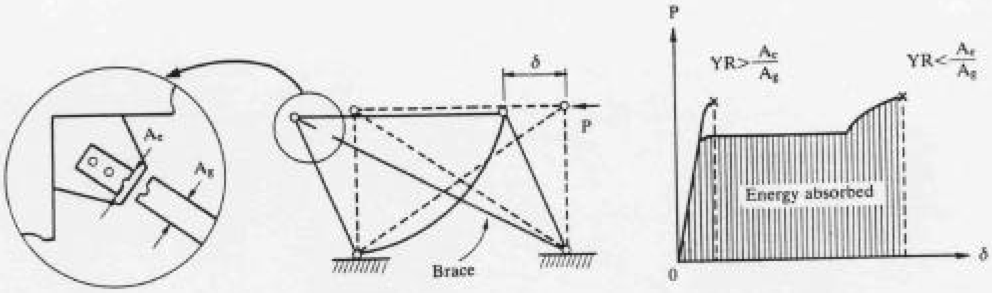


Figure 1.10. Effect of yield-tensile ratio on energy absorbed by brace.

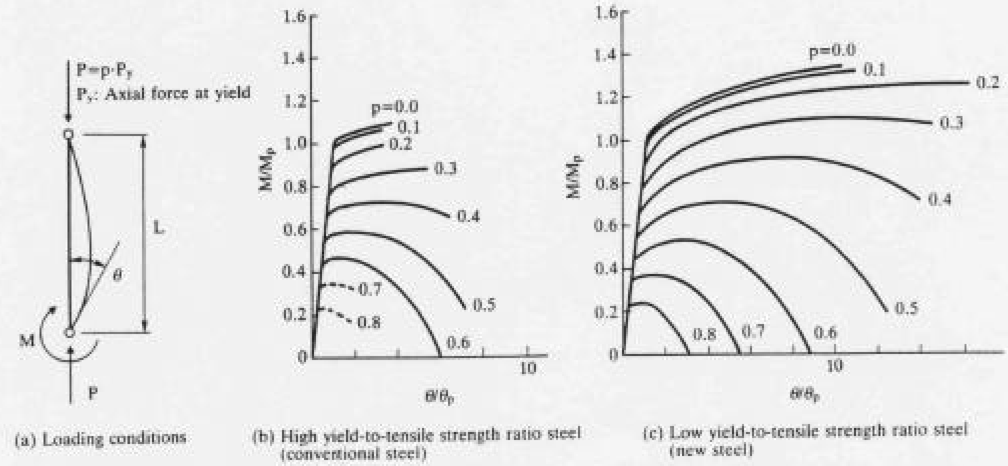


Figure 1.11. Effect of yield-tensile ratio on deformation capacity of beam-column.

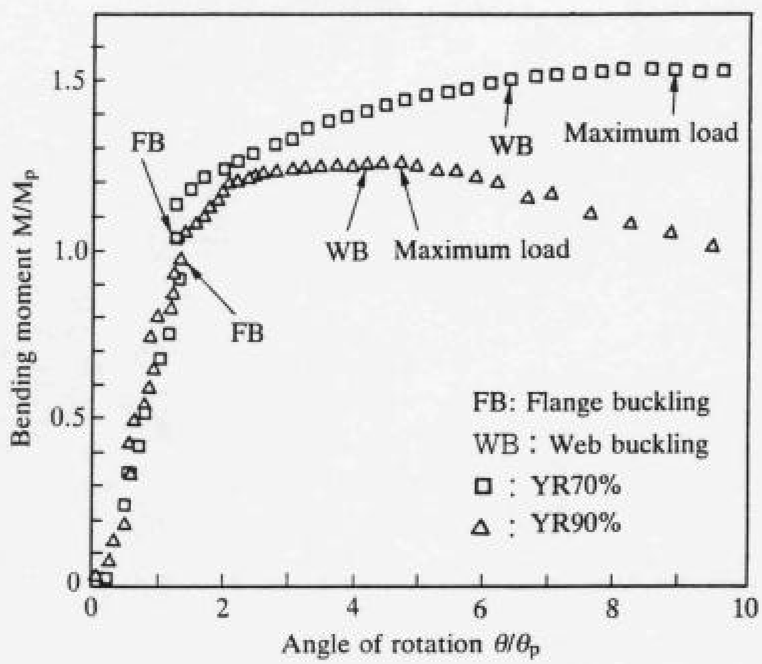


Figure 1.12. Effect of yield-tensile ratio on behavior of cantilever beam.

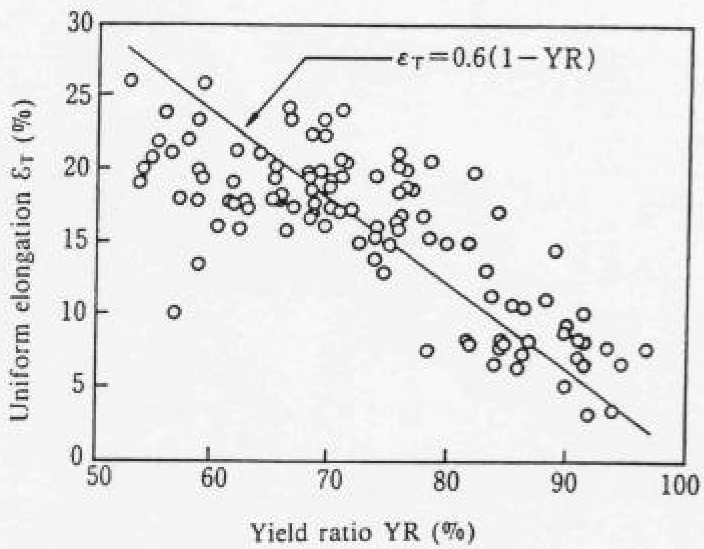


Figure 1.13. Relation between yield-tensile ratio and uniform elongation.

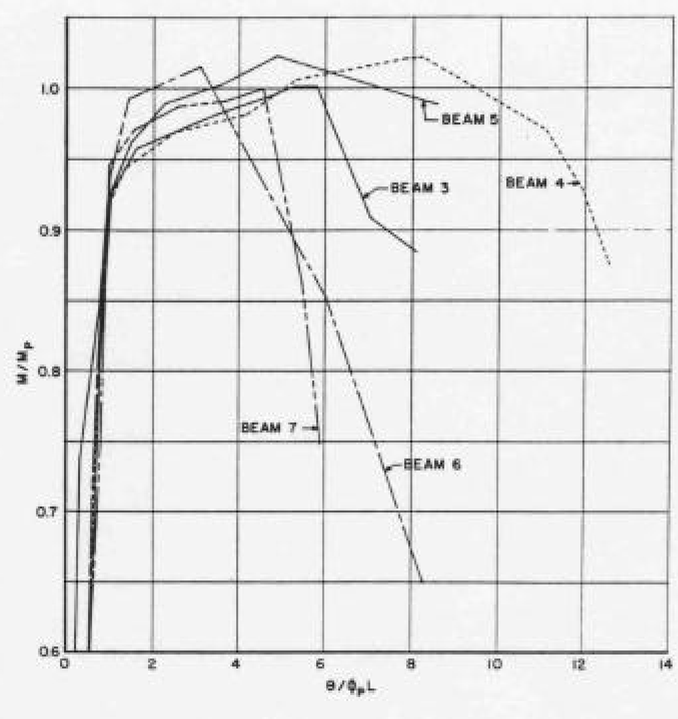
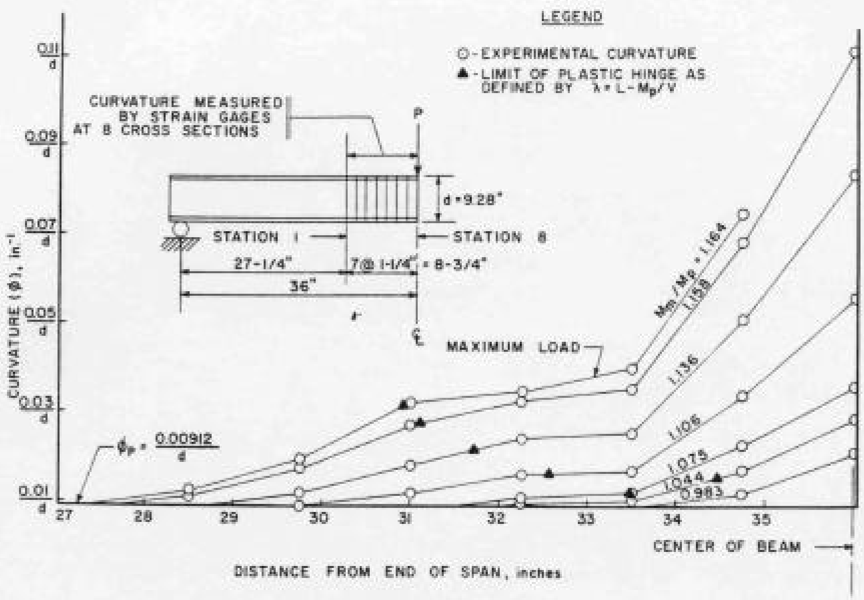
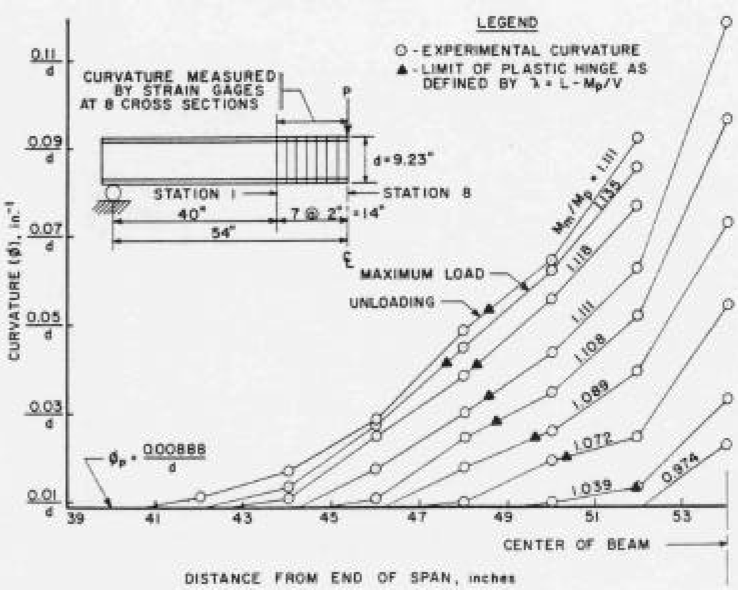


Figure 1.14. Moment-rotation curves for A514 beams with uniform moment.



(a) Beam A



(b) Beam B

Figure 1.15. Curvature of A 514 beams with moment gradient.

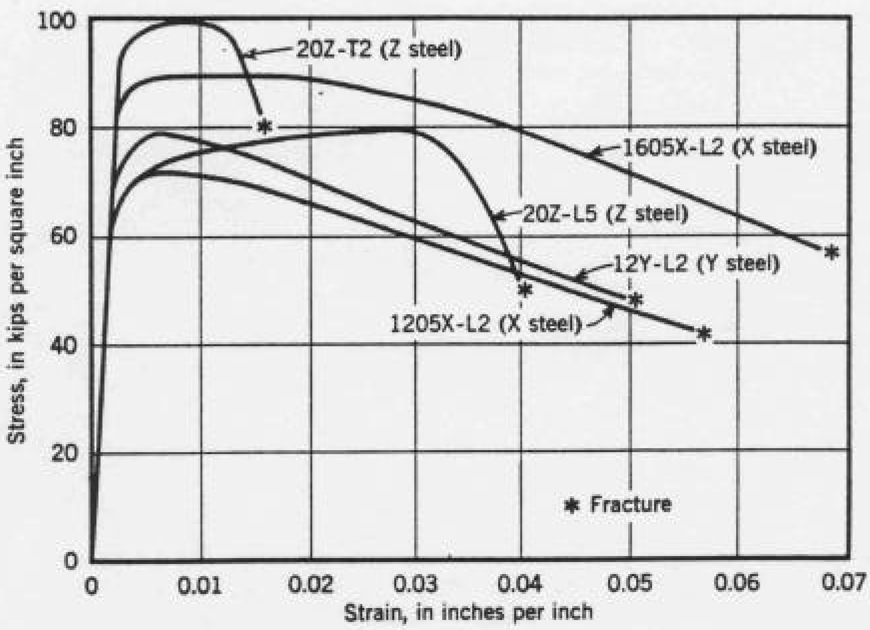


Figure 1.16. Complete stress-strain curves for X, Y, and Z steels.

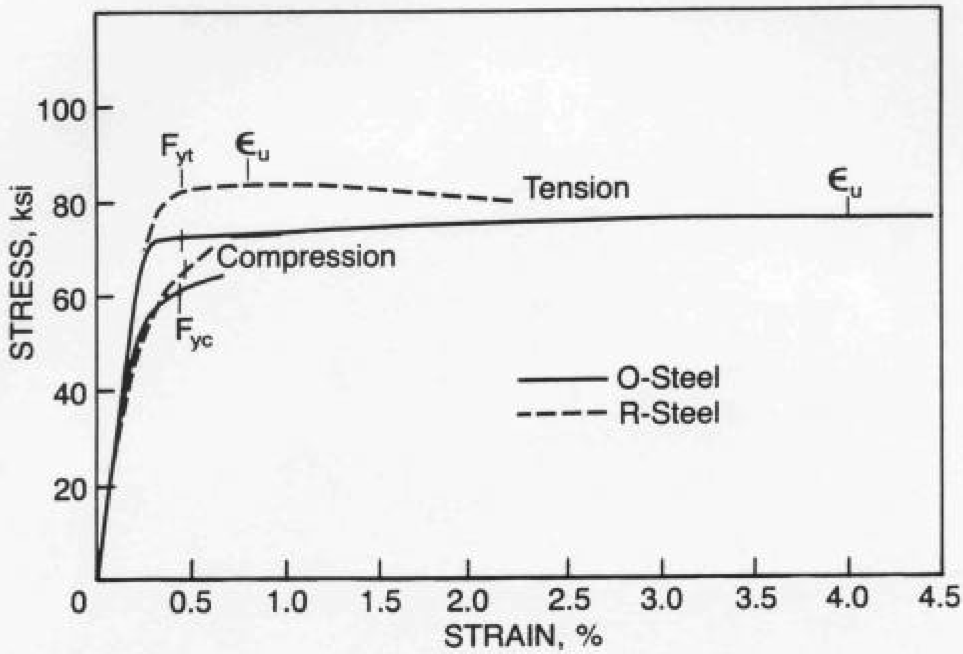


Figure 1.17. Stress-strain curves for LSHD steels.

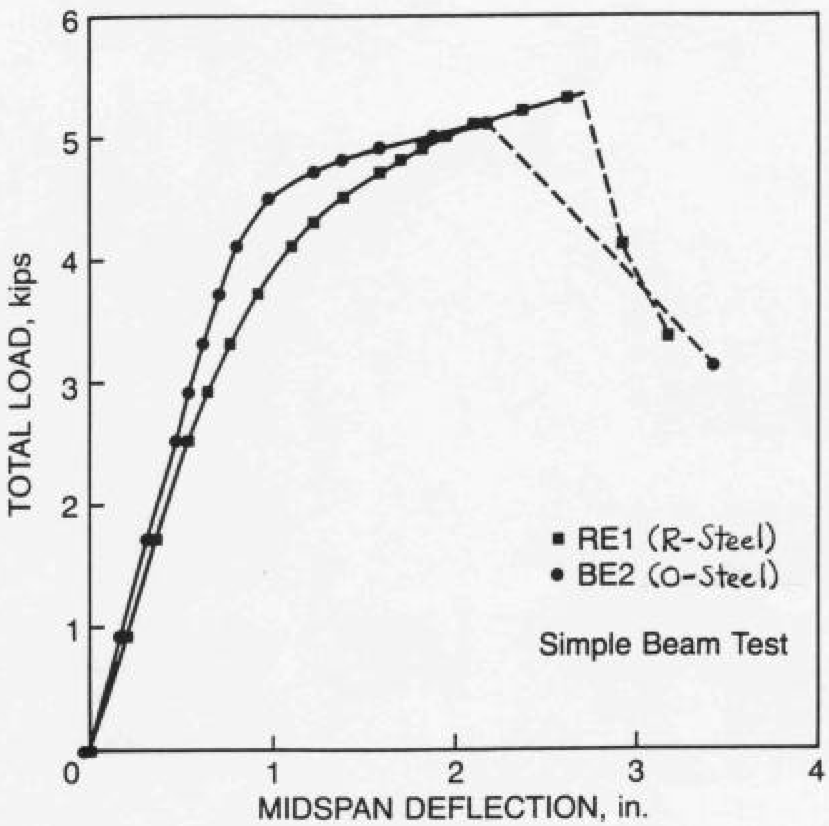


Figure 1.18. Response of LSHD steels in bending tests.

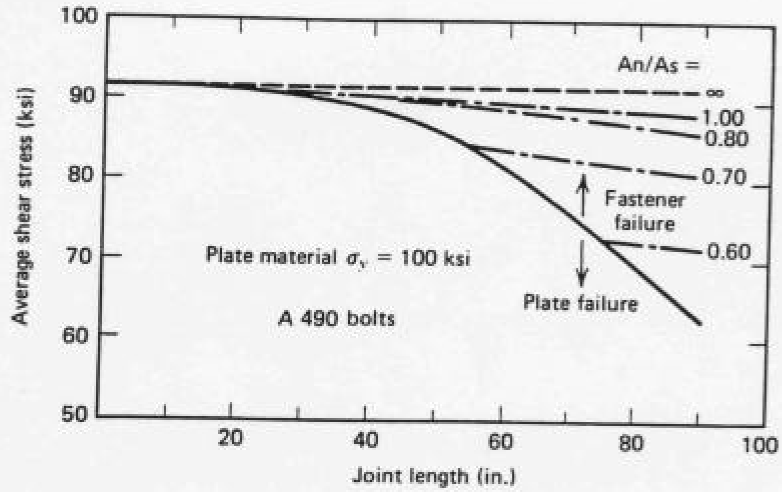


Figure 1.19. Bolt shear strength in joints of A514 steel.

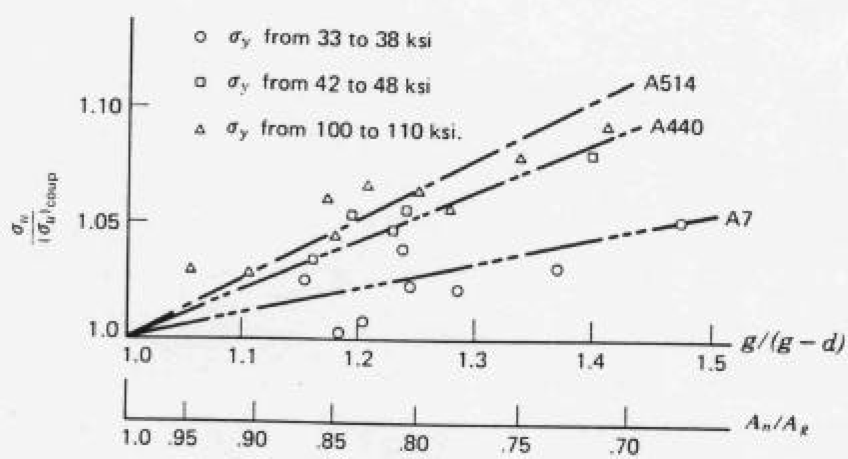


Figure 1.20. Tensile strength on net section at ultimate load for three steels.

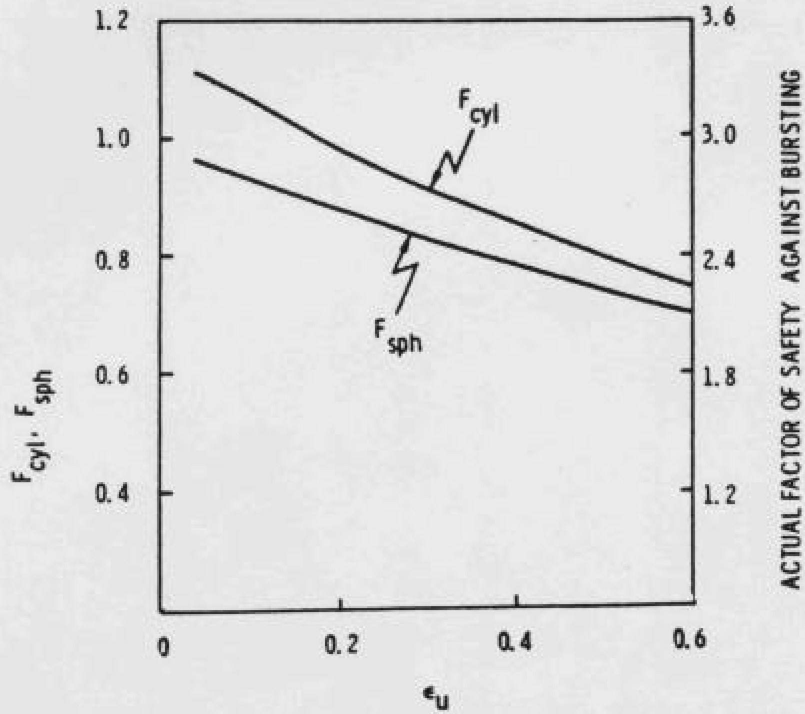


Figure 1.21. Effect of strain-hardening exponent on burst pressure.

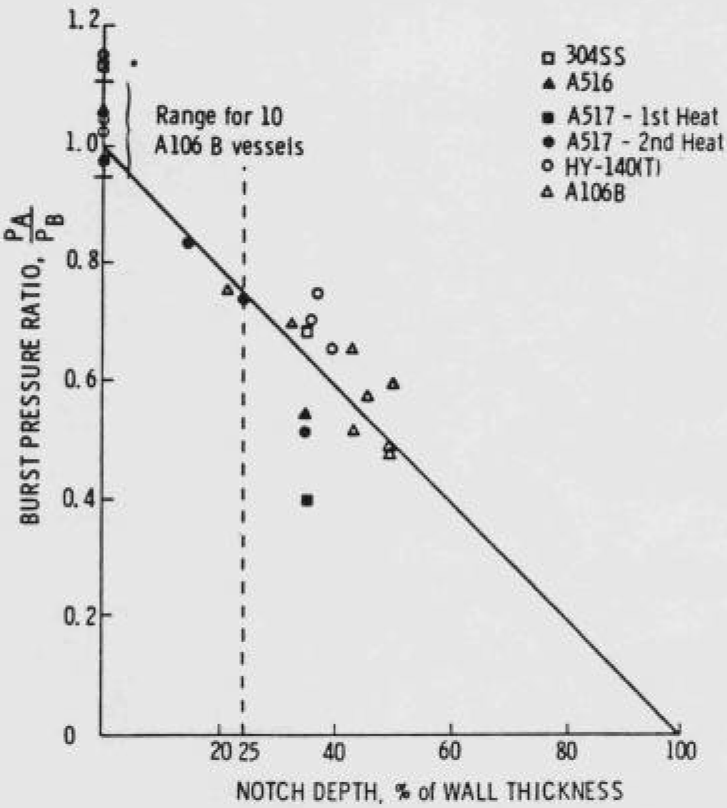
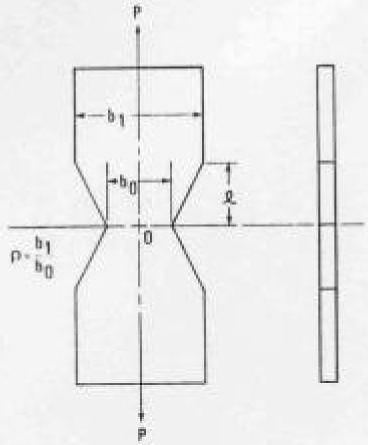
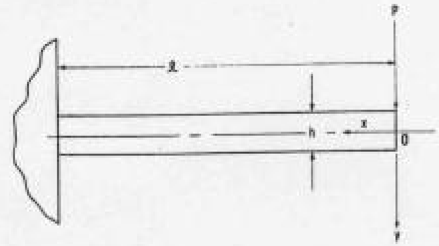


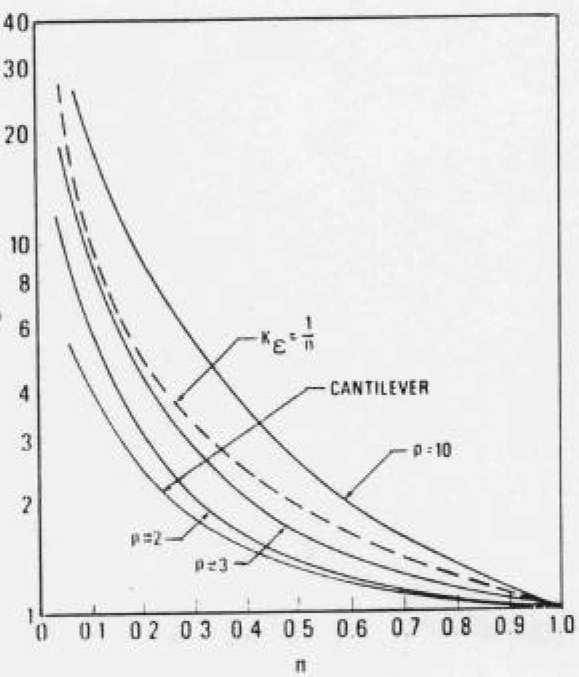
Figure 1.22. Effect of notch depth on burst pressure.



(a) Tapered bar.

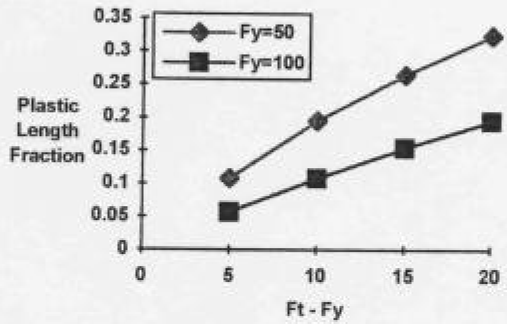


(b) Cantilever beam.

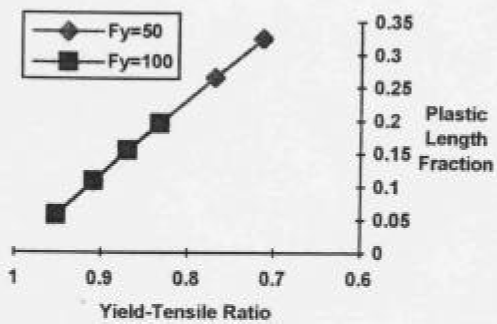


(c) Strain concentration factors.

Figure 1.23. Relation of strain-hardening modulus to strain concentration factors.

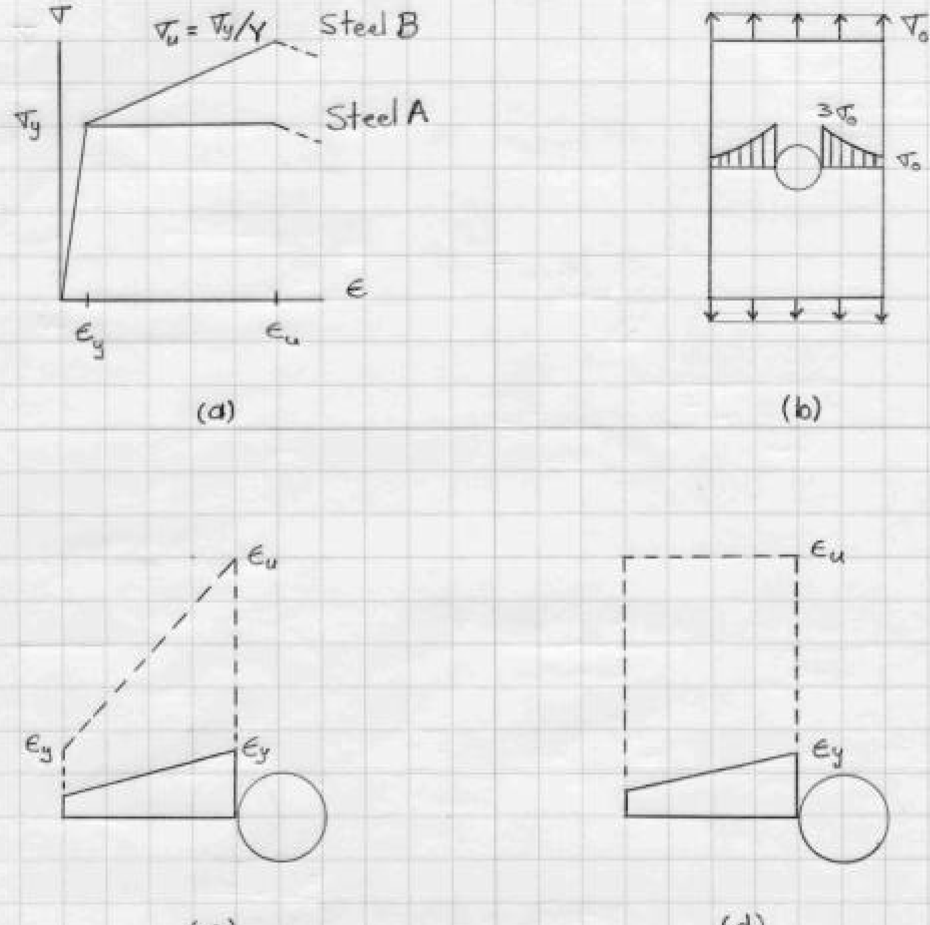


(a) Effect of Yield-Tensile Difference



(b) Effect of Yield-Tensile Ratio

Figure 1.24. Effect of yield-tensile difference and yield-tensile ratio on plastic length fraction of beam-column.



Steel B / Steel A:

$$\text{Ratio of Inelastic Deflections} = \frac{\epsilon_u}{(\epsilon_u + \epsilon_y)/2} \cong 2$$

$$\text{Ratio of Maximum Loads} = \frac{\sigma_u}{(\sigma_u + \sigma_y)/2} = \frac{2}{1+Y}$$

Figure 2.1 Effect of yield-tensile ratio on local yielding. (a) Stress-strain curves. (b) Tension strap with bolt hole. (c) Strain at maximum load for steel A. (d) Strain at maximum load for steel B.

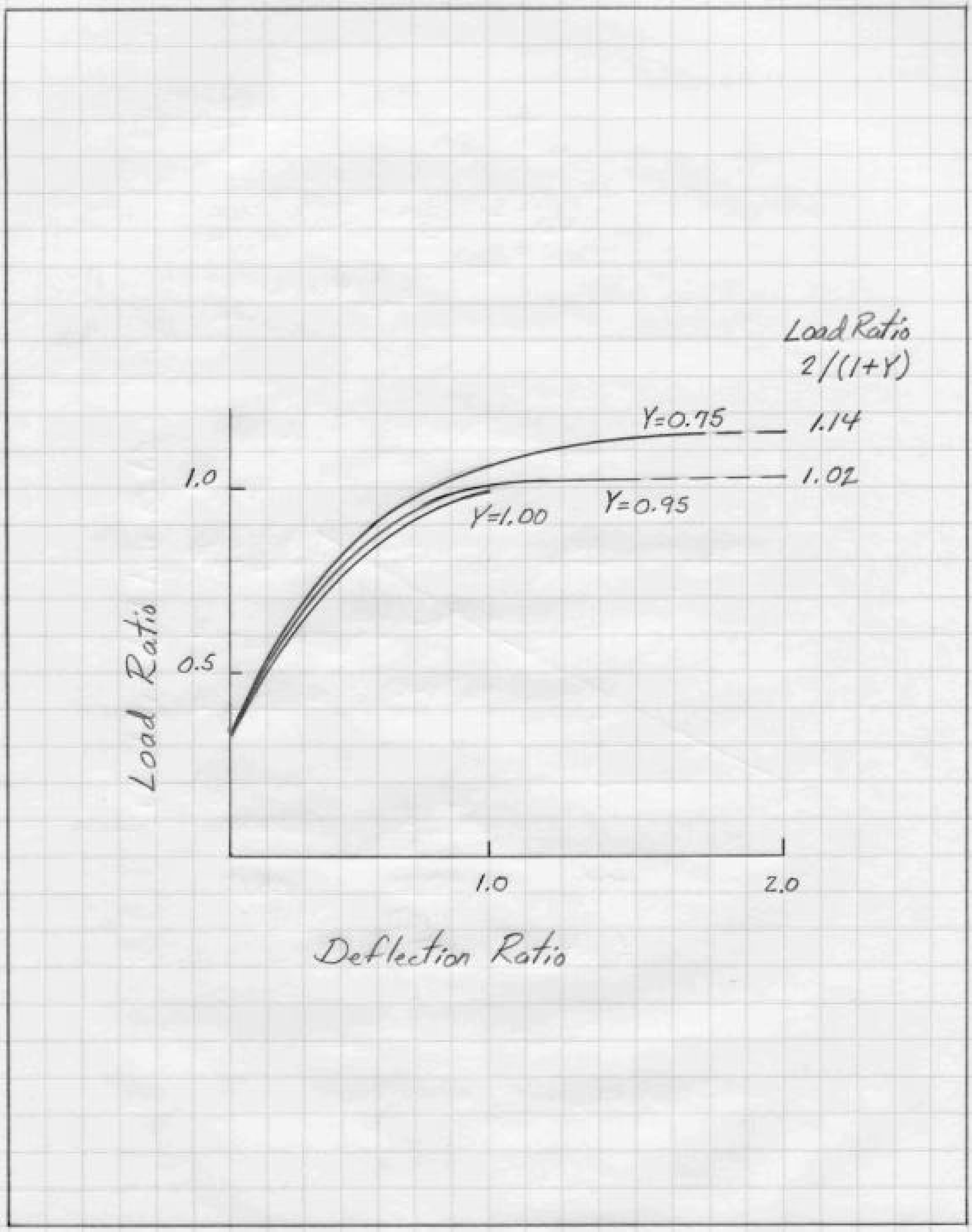
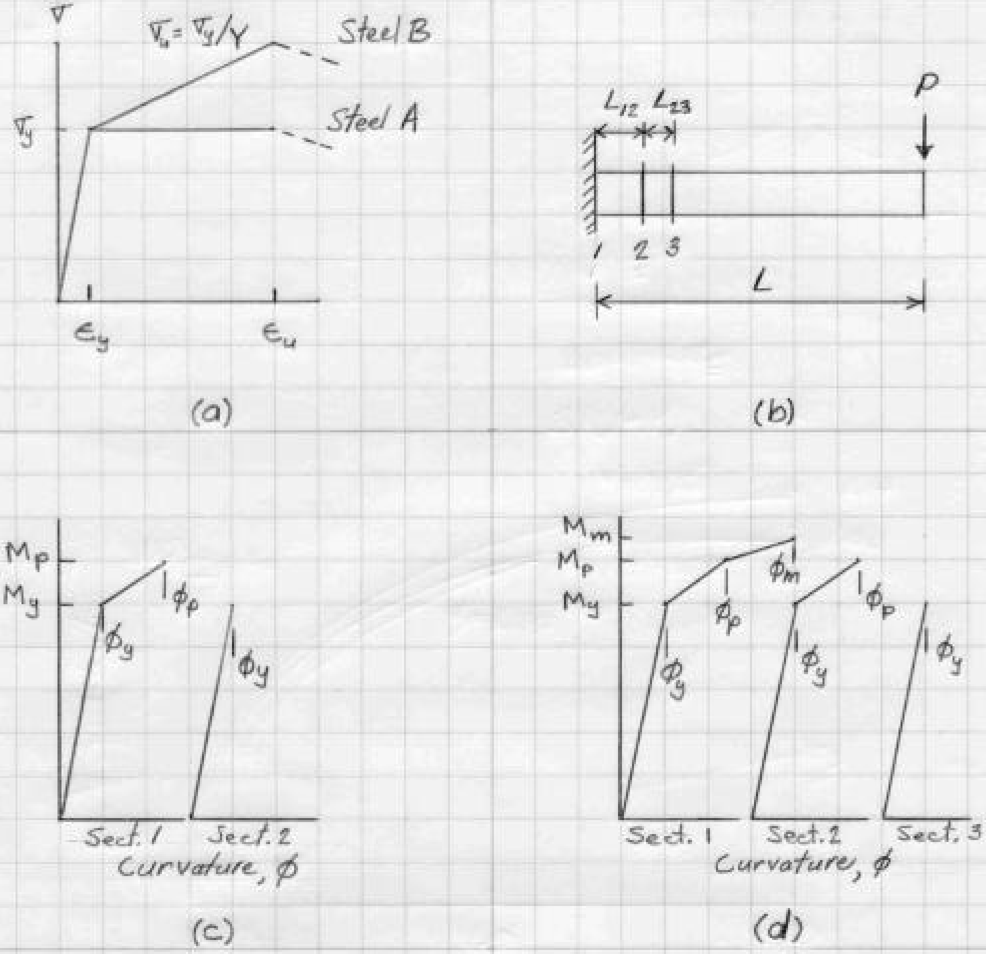


Figure 2.2. Dimensionless load-deflection plots for tension strap with yield-tensile ratio of 0.75, 0.95, and 1.00.



Rotation:

$$\theta_A = \frac{1}{2}\phi_p L \left(1 + \frac{\phi_y}{\phi_p}\right)\left(1 - \frac{M_y}{M_p}\right)$$

$$\theta_B = \frac{1}{2}\phi_p L \left[\left(1 + \frac{\phi_m}{\phi_p}\right)\left(1 - \frac{M_p}{M_m}\right) + \left(1 + \frac{\phi_y}{\phi_p}\right)\left(\frac{M_p}{M_m} - \frac{M_y}{M_m}\right)\right]$$

Figure 2.3. Effect of yield-tensile ratio on moment-rotation behavior of cantilever beam. (a) Stress-strain curves. (b) Cantilever beam with end load. (c) Moment-curvature plots for sections 1 and 2 for steel A. (d) Moment-curvature plots for sections 1, 2 and 3 for steel B.

Steel A:

$$M_1 = P_{max} L = M_p \quad \therefore \; P_{max} = M_p / L$$

$$M_2 = P_{max}(L - L_{12}) = M_y \quad \therefore \; \frac{M_p}{L}(L - L_{12}) = M_y$$

$$\text{or} \quad L_{12} = L\left(1 - \frac{M_y}{M_p}\right)$$

Rotation.

$$\theta_A = \frac{(\phi_p + \phi_y)}{2} L_{12} = (\phi_p + \phi_y)\frac{L}{2}\left(1 - \frac{M_y}{M_p}\right)$$

$$\theta_A = \frac{1}{2}\phi_p L \left(1 + \frac{\phi_y}{\phi_p}\right)\left(1 - \frac{M_y}{M_p}\right)$$

Steel B:

$$M_1 = P_{max} L = M_m \quad \therefore \; P_{max} = M_m / L$$

$$M_2 = P_{max}(L - L_{12}) = M_p \quad \therefore \; \frac{M_m}{L}(L - L_{12}) = M_p \quad \text{or} \; L_{12} = L\left(1 - \frac{M_p}{M_m}\right)$$

$$M_3 = P_{max}(L - L_{12} - L_{23}) = M_y \quad \therefore \; \frac{M_m}{L}(L - L_{12} - L_{23}) = M_y \quad \text{or} \; L_{23} = L\left(\frac{M_p}{M_m} - \frac{M_y}{M_m}\right)$$

Rotation.

$$\theta_B = \frac{(\phi_m + \phi_p)}{2} L_{12} + \frac{(\phi_p + \phi_y)}{2} L_{23}$$

$$\theta_B = \frac{1}{2}\phi_p L \left[\left(1 + \frac{\phi_m}{\phi_p}\right)\left(1 - \frac{M_p}{M_m}\right) + \left(1 + \frac{\phi_y}{\phi_p}\right)\left(\frac{M_p}{M_m} - \frac{M_y}{M_m}\right)\right]$$

Figure 2.4. Derivation of relationships for moment-rotation behavior of cantilever beam.

Steel A:

Assume $\frac{\phi_y}{\phi_p} = \frac{1}{2}$ and $\frac{M_y}{M_p} = \frac{1}{1.12}$

Then $\theta_A = \frac{1}{2}\phi_p L\left(1+\frac{1}{2}\right)\left(1-\frac{1}{1.12}\right)$

$= 0.0804\,\phi_p L$

Steel B:

Assume also that $\frac{M_p}{M_m} = Y$ and $\frac{\phi_p}{\phi_m} = Y$

Then $\theta_B = \frac{1}{2}\phi_p L\left[\left(1+\frac{1}{Y}\right)(1-Y) + \left(1+\frac{1}{2}\right)\left(Y - Y\frac{1}{1.12}\right)\right]$

$= \frac{1}{2}\phi_p L\left[\frac{1}{Y}(1-Y^2) + \frac{3Y}{2}(0.107)\right]$

$= \frac{1}{2}\phi_p L\left[\frac{1}{Y}(1-Y^2) + 0.161\,Y\right]$

Y	$\theta_B/\phi_p L$	θ_B/θ_A
0.75	0.352	4.38
0.85	0.232	2.88
0.95	0.128	1.59
1.00	0.080	1.00

Figure 2.5. Simplification of relationships for moment-rotation behavior of cantilever beam.

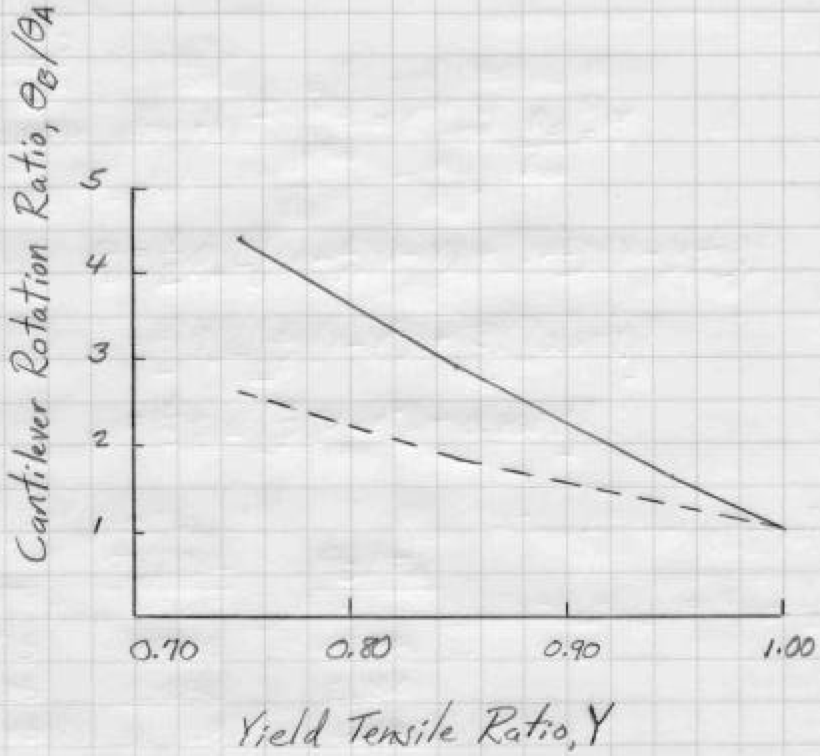
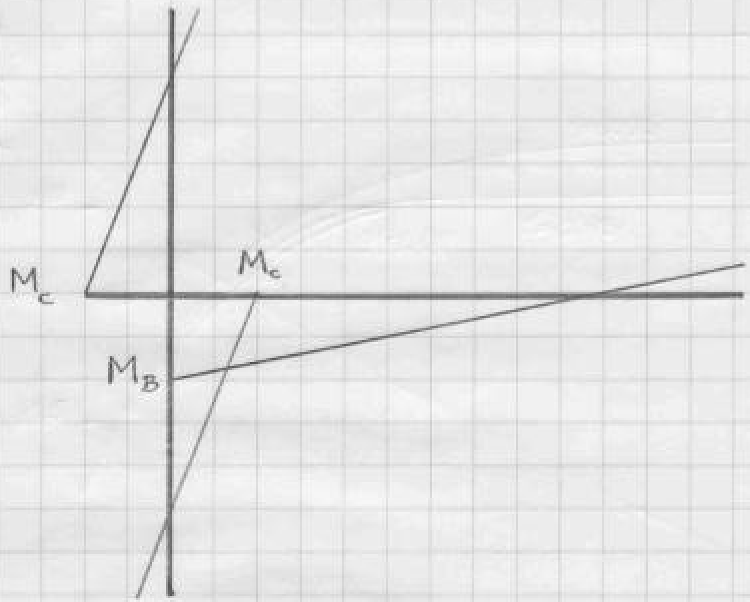


Figure 2.6. Illustration of effect of yield-tensile ratio on moment-rotation behavior of cantilever beam.



$$\frac{\sum Z_c \left(F_{yc} - P_{uc}/A_g\right)}{\sum Z_b F_{yb}} \geq 1.0$$

A_g = Gross column area
F_{yb} = Spec. min. yield strength of beam
F_{yc} = " " " " " column
P_{uc} = Required axial strength of column
Z_b = Plastic section modulus of beam
Z_c = " " " " column

Figure 2.7. Elastic moment diagram at beam-to-column intersection for moment frame subjected to seismic loading.