

ROLE OF YIELD-TO-TENSILE STRENGTH RATIO IN THE DESIGN OF STEEL STRUCTURES

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Abstract

Avoidance of structural collapse during extreme events like earthquakes relies on the available ductility of the structure and its members and components. Over the years, the yield stress-to-tensile strength ratio (Y/T), which is a measure of the ductility of steel, has increased from 0.5 to 0.85 or more. This paper discusses; (a) Yield stress-tensile strength- Y/T ratio relations, (b) relevance of Y/T ratio in structural design and (c) treatment of Y/T ratio in the North American steel design codes. The focus of this paper is limited to ductility aspects of civil engineering structural designs, such as design of buildings and bridges. Statistical analysis of mechanical properties of steels shows that the Y/T ratio increases with increasing yield stress, and with increasing tensile strengths of steel. However, Y/T ratio is significant only in steel structural systems and members that are expected to withstand strain-hardening range stresses and strains, such as ductile elements of earthquake resistant structural systems, stress concentration situations like holes in tension members and beams, and structures designed based on plastic analysis. This paper discusses the required geometric and material conditions to prevent premature failures of steel members and structures and to ensure adequate ductility.

Introduction

Design of structures to withstand nature's fury, such earthquakes, hurricanes, tornados, snowstorm, etc., has been a challenge and at times had been a failure. Understanding the nature and the impact of such natural hazards and understanding the use and occupancy requirements of a structure are essential in order to achieve a successful structural design. As a minimum, structures are designed to be safe and serviceable during normal use and occupancy and protected from collapse during extreme events like earthquakes. Safety is assured provided the structures and their members are designed to have sufficient strength and stability such that the factored resistance is greater or equal to the effects of factored loads acting in the most critical combination. Furthermore, configuration of the structural members and their connections must ensure resistance to widespread collapse as a consequence of local failure (sufficient structural integrity). Serviceability is assured by proportioning the structural members and frames such that the serviceability parameters (deflection, vibration, etc.) are within acceptable limits for the nature of the materials supported and for the intended use and occupancy. In order to maximize the economical efficiency of structures, prevention of collapse during extreme events like earthquakes relies on the ductility of the structure. "Ductility" is the ability of the structure, its members and elements, and the structural material to undergo inelastic deformations without

loosing its strength. Structural steel is the most ductile of the modern structural engineering materials. In Canada, the structural steel must meet the requirements of CSA standard G40.20/G40.21 (CSA, 2004), whereas in U.S.A. they must meet the requirements of the American Society for Testing and Materials standard (ASTM, 2008).

Early twentieth century North American buildings and bridges used mild-carbon structural steel with a specified minimum yield stress F_y of 190 MPa - 225 MPa. Corresponding tensile strength F_u was 380 MPa – 450 MPa, giving a specified yield-to-tensile strength ratio F_y/F_u of 0.5. The yield-to-tensile ratio (Y/T) is a measure of strain hardening ability and the ductility of steel. The improvements in steel making technologies over the years, motivated by the desire to use higher strength structural steels, have made possible to produce higher strength steels (345 MPa to 690 MPa yield strength) with improved weldability, high toughness, and improved atmospheric corrosion resistance (weathering). Currently, the steel grade ASTM A992/A992M has become the dominant material specification for wide flange shapes in U.S.A. and in Canada. The ASTM A992 requirements for tensile properties include; yield point range of 345-450 MPa (50-65 ksi), minimum tensile strength of 450 MPa (65 ksi), and a maximum yield-to-tensile strength ratio of 0.85. Resulting F_y/F_u of A992 steel is between 0.77 and 0.85. In recent years A913/913M grade steels, which are high-strength low-alloy steel shapes of structural quality produced by a quenching and self-tempering process, have been introduced in the building codes. The ASTM A913 Grade 65 steel mechanical properties requirements are yield strength 450 MPa (65 ksi), and tensile strength of 550 MPa, resulting in a F_y/F_u ratio of 0.82. Steels for application in bridges are covered in ASTM A709/709M. High-performance steels (HPS) are the newest additions to the family of bridge steels. The HPS-70W [485W], which has been used the most, has ASTM requirements of minimum yield strength 485 MPa (70 ksi), and tensile strength of 585-760 MPa, resulting in a F_y/F_u ratio of 0.83 – 0.64. HPS-100W [690W], with yield strength 690 MPa (100 ksi), and tensile strength of 760-895 MPa which gives a F_y/F_u ratio of 0.91 – 0.77, is also available to reduce thickness of bridge members (Brockenbrough and Merritt, 2006). Therefore, over the years the yield-to-tensile strength ratio of structural steel has increased from 0.5 to a code permitted maximum of 0.85. Higher Y/T ratio may imply lesser strain hardening ability and lesser material ductility. Concern has been expressed over the high Y/T ratio of high strength steels ability to provide adequate strain hardening and ductility to achieve sufficient safety margins against fracture, somewhat comparable to the safety margins associated with low Y/T ratio steels. This paper discusses (a) Yield stress-Tensile strength-Y/T ratio relations, (b) role of Y/T ratio in structural design and (c) treatment of Y/T ratio in current North American steel design codes. The focus of this paper is limited to ductility aspects of civil engineering structural designs, such as ductile design of steel buildings and inelastic design of steel bridges.

Stress-Strain Properties of Structural Steel

Stress-strain relations are the most commonly used mechanical properties of material in civil engineering structural design. Such properties of steel are established through tension tests in accordance with ASTM A370 testing procedures “Standard Test Methods and Definitions for Mechanical Testing of Steel Products”, (ASTM, 2008). The mechanical properties of interest in a civil engineering structural design are; yield stress (F_y), yield strain (ϵ_y), tensile strength (F_u), ultimate strain (ϵ_u), and perhaps the fracture strain (ϵ_f). Often it is convenient to use the 0.2% offset method to establish the yield stress and strain, though stress at yield plateau may be used to identify such values for steels exhibiting yield plateau. Recent experiments (Arasaratnam, 2008), on identical tensile coupons from A992 steel grade show an average yield stress and strain of 445 MPa, and 0.0022, respectively, and the tensile strength and the ultimate strain of 577 MPa, and 0.1381, respectively. The strain at fracture (ϵ_f), based on 200 mm (8”) gauge extensometer measurements was 0.2082. These values compared well with the mill test report given by the supplier of these steel beams. Figure 1 shows the idealized stress-strain properties of the steel. From the yield and ultimate stress and strain, other salient parameters such as elastic modulus (E), strain hardening modulus (E_{st}), yield ratio (Y/T) yield stress/tensile strength, elongation capacity reflected through ductility ratio (μ) ultimate strain/yield strain, etc. can be derived. Based on test results $E = 200,900$ MPa, $E_{st} = 971$ MPa, $Y/T = 0.77$, and $\mu = 63$ were obtained.

Yield Stress, Tensile Strength and Yield-to-Tensile Strength Ratio Relations

Prior to the introduction of probability based design of steel structures in the 1970’s, statistical values for mechanical properties were not generally established because the steel specifications of that era worked with specified minimum values. The studies by Galambos and Ravindra (1978) and by Kennedy and Gad Aly (1980), which established the statistical parameters for steel design for the American and Canadian steel design standards, respectively, focussed on the yield stress and not the ultimate strength. Even the steel related research studies of that era reported either incomplete or limited material property data related to the steels used in their research. Lilly and Carpenter (1940), however, provided the yield point and tensile strength of web plate, cover plates, and flange angles used in their investigation related to riveted plate girders. Two tension coupons each were considered for each of these elements, and the yield stress ranged 290-370 MPa(42-54 ksi) and the corresponding tensile strength ranged 419-505 MPa(61-73 ksi). Since then, however, various studies have reported on the statistical yield stress

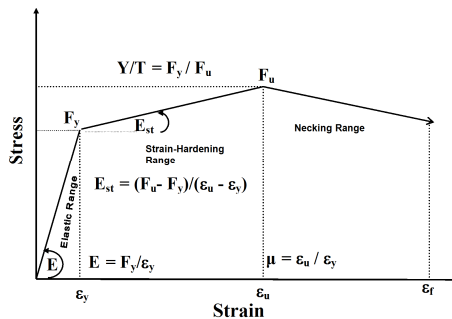


Figure 1. Idealized stress-strain properties

and the corresponding tensile strength of different steel grades;

- Yamanouchi, et. al. (1990) established the statistical features of mechanical properties of SS41 (Specified minimum $F_y = 235$ MPa, and $F_u = 403-510$ MPa) and SM50A (Specified minimum $F_y = 315$ MPa, and $F_u = 490-607$ MPa) steel grades, since these grades were widely used at that time for general steel building structures in Japan. This study was based on 4160 mill test reports of these grades of steel that were produced with blast furnaces at six major Japanese mills during the last six months of 1986. The study observed that, for the same steel grade, the measured yield stress is higher in thin plates than thick plates. Also, the scatter in yield stress is larger compared to the scatter associated with the tensile strength.

- Based on 13536 mill test reports Frank and Read (1993) reported mechanical properties of ASTM A572 Grade 50 steel. Later, Jaquess and Frank (1999) established mechanical properties of rolled sections made of A572 Grade 50 based on 59 tension tests in accordance with ASTM A370 testing procedures (ASTM, 2008).

- Sooi, et.al. (1995) reported stress-strain properties of high strength – low alloy HSLA80[550 MPa] steel, based on 33 tensile coupon tests on 9.5-25.4 mm (3/8-1”) thick plates rolled in three different mills.

- Brockenbrough (2003) conducted a statistical survey of mill test reports of plate grades typically used in building construction. This survey considered one year production of three U.S.A. steel mills ASTM A36, A572 (Grades 50,60,65), A588, A852, and A514 having thickness range of 4.8 mm (0.188”) to 101.6 mm (4”).

- Since most structural steel shapes currently used in North America are produced to a single material grade specification, ASTM A992/A992M, which meets or exceeds the A36, A572-Grade 50 (CSA G40.21 Grades 300 and 350), Bartlett, et.al. (2003) focussed on the material properties of A992. In this study, they summarized the mechanical properties of ASTM 992 as determined by 207 tests in accordance with ASTM A370. Samples for these tests were obtained from 38 heats of steel from eight different shapes provided by three producers. These results were compared with the A992 mechanical properties compiled by Dexter, et.al (2000) based on more than 20,000 mill test reports supplied by North American and European mills.

- High Performance Steel (HPS) has become increasingly common for bridge girder applications in North America. Dexter, et.al (2002) investigated the tensile properties of HPS-485W(70W) 19 mm steel plates. In order to provide a frame of reference they also considered ASTM A709-Grade 50 steel and HPS-690W(100W) 19 mm steel plates. Sause and Fahnestock (2001) reported tensile stress-strain relationships for HPS-100W steel. Their results consist of six tests from flange plates of 19 mm (3/4”) thick, and four tests from web plate of 10 mm (3/8”). Kayser, et.al. (2006) recently reported material properties of HPS-485W(70W) steel plates. Their results were based on 70 tension coupons from 22 mm (7/8”) plates and 26 coupons from 51 mm (2”) plates.

Figure 2 shows the relations between Y/T ratio and the yield stress, and the relations between Y/T ratio and the tensile strength. The data points are from the studies discussed above. Y/T ratio was established from the mechanical properties. It is evident that, in general, the Y/T ratio increases with increasing yield stress of steel grade. Y/T ratio also increases with increasing tensile strengths of steel.

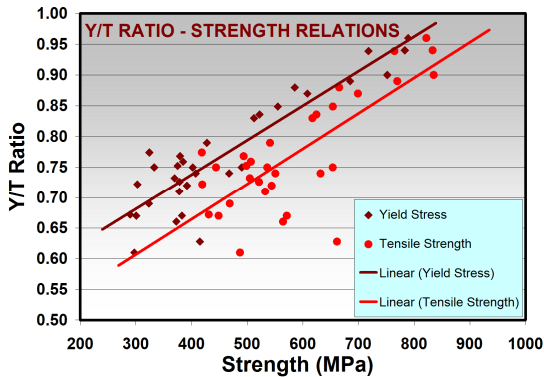


Figure 2 Y/T ratio – strength relations

Relevance of Yield-to-Tensile Strength Ratio in Ductile Structural Design

General Aspects

The initial part of idealized stress-strain relations for structural steels shown in Figure 1 is the elastic range, where steel obeys the Hooke's law and loading and unloading do not cause permanent deformations. Some design and construction practices, such as (a) working stress design which usually considers, say, $0.6F_y$ for normal loading and up to $0.8F_y$ in extreme loadings, (b) structural systems in regions of negligible seismic risk, (c) structural systems with little or no redundancy (simply supported beams), etc., result in structures that remain elastic even at extreme loadings or do not lend themselves for ductile design. Since the stresses and strains in these structures remain lower than F_y and ϵ_y , respectively, at all times, the Y/T ratio is largely irrelevant in the design and behaviour of such structures. In contrast, Y/T ratio may become relevant in steel structural systems (ductile moment resisting frames, ductile plate walls), in steel structural components (connections, link beams), and in elements of steel structural members (flanges, webs, flange holes, tension members with holes) that are expected to withstand strain-hardening range stresses and strain or even necking range strains (see Figure 1).

Steel framed system, designed to perform in a ductile manner when warranted, provides the best economical earthquake-resistant structural solution for regions of moderate and high seismicity. However, the survival of such steel structural systems in large earthquake depends directly on the ability of the system to undergo large plastic deformations. Modern seismic-force-resisting systems design is based on capacity design principles. Accordingly, specific elements or mechanisms are designed to dissipate energy during large earthquakes, and all other elements are designed to be sufficiently strong for this energy dissipation to be achieved. In this design technique, the ductile energy dissipating systems are clearly identified and detailed to constrain inelastic response to these ductile elements only. This implies that all other elements and members must be designed to remain essentially elastic for the duration of the seismic ground

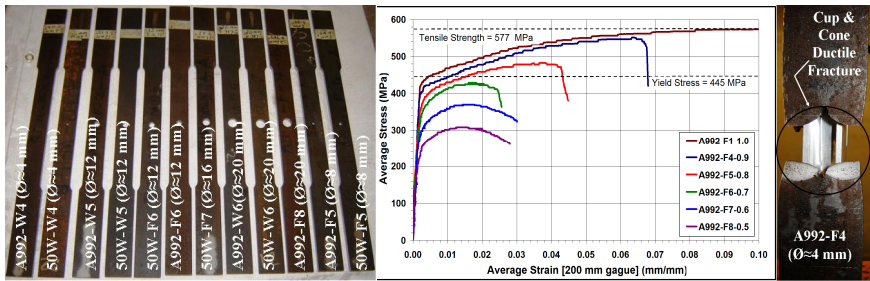


Figure 3. Tensile Response of Specimen with Single Hole

motion. In this context the Y/T ratio becomes relevant in such ductile elements only, and Y/T ratio is irrelevant for all other elements that remain elastic.

The wide flange steel sections, hollow structural steel sections, and fabricated steel plate girders are produced with flanges and webs having a variety of slenderness values. Local buckling resistance of these steel members depend on the slenderness of the flanges and the webs. The moment resistance and the plastic deformation capacity of beams are governed by the local buckling resistance of the flanges and webs. Stocky members, known as “seismically compact” sections, can attain plastic moment and can exhibit substantial inelastic rotations prior to inelastic local buckling, whereas “compact” sections, can attain plastic moment and may exhibit limited inelastic rotations prior to inelastic local buckling. Other members, known as “non-compact” sections or “slender” sections, experience elastic local buckling. Compact members must be used in the ductile design of structures. Y/T ratio may be relevant in “compact” members only, and Y/T ratio is irrelevant for “non-compact” sections and “slender” sections, which remain elastic and experience elastic local buckling failure. Flange holes, often provided for bolted connections, may cause localized concentration of stresses. The resulting strains in the vicinity of the tension flange holes may be beyond the elastic range. In this situation, Y/T ratio may determine whether the tension flange with holes is susceptible to premature flange rupture.

Tension Members: Strength and Ductility

Tension members with holes (example: bolt holes in braces and splice plates) or tapered tension members are non-uniformly stressed and are susceptible for brittle fracture type failure. The necessary condition for a tensile member with a hole to assure a ductile behaviour is to ensure that the gross-section yields before the net section fractures. That is $(A_g F_y) < (A_n F_u)$ or $(A_n/A_g) > (F_y/F_u)$, where A_n is the net area and A_g is the gross area. Kuwamura and Kato (1989) demonstrated this relations by considering steel specimen with Y/T ratio of 0.63, and having central holes with (A_n/A_g) of 0.7 and 0.5. The specimen with $A_n/A_g = 0.5$ exhibited considerably lower deformability than the specimen with $A_n/A_g = 0.7$, which is greater than the Y/T ratio. Kuwamura and Kato (1989) also considered tapered tension members made of steel grades of Y/T ratio = 0.63, 0.71, 0.77, and 0.93. They observed that the elongation at fracture ϵ_f decreases (a) with increasing Y/T ratios, and (b) in the presence of a hyperbolic taper. Sooi, et.al. (1995) conducted similar tensile tests, but using high strength low alloy steel HSLA80[550 MPa] (Y/T ratio =0.88) and straight taper (representing a tension flange of a beam member subjected to a linear moment gradient), and arrived at similar conclusions. Tension tests in accordance with ASTM A370 testing procedures (ASTM, 2008) were conducted at McMaster University on 9.4

mm steel plates (a) without hole and (b) with single hole of varying sizes. ASTM A992 [Average yield stress = 445 MPa; Y/T ratio = 0.77] and CAN/CSA G40.20/G40.21 350 W [Average yield stress = 428 MPa; Y/T ratio = 0.74] steel grades were considered for these tests. Single hole tensile specimens had $(A_n / A_g) = 0.5, 0.6, 0.7, 0.8, \text{ and } 0.9$. Figure 3 shows the specimens with holes. Figure 3 also shows the load-displacement relations associated with these specimens. Considering the A992 steel yield stress of 445 MPa ($\epsilon_y = 0.22\%$), it is evident that the specimens having $A_n/A_g > Y/T$ ratio reached ultimate loads higher than the yield load. Tensile coupons with $A_n/A_g < Y/T$ ratio however, could not reach the yield stress levels, and exhibited lesser inelastic deformations. For example, the specimen with $A_n/A_g = 0.60$, which is considerably less than the Y/T ratio of 0.77, reached an average stress of 369 MPa, which is 83% of the yield stress, while experiencing an elongation of nearly 2% (tensile ductility of about 9). Though this specimen exhibited limited ductility, it reached load levels slightly more than $(A_n \cdot F_u)$ before experiencing net section fracture. Overall, adequate strength and ductility can be achieved in tension members, provided $(A_n/A_g) > Y/T$ ratio. These observations may be applied to tension flange (with holes) of a beam in unformed bending. Dexter, et.al. (2002) conducted a statistical analysis of experimental Y/T ratios. The conclusion of this study is that adequate ductility is achievable for steel grades with $F_y < 345$ MPa (50 ksi) provided $(A_n/A_g) > (F_y/F_u)$, and is achievable for high strength steels, such as HPS-485W(70W), provided $(A_n/A_g) > 1.1 (F_y/F_u)$. The tensile ductility may be translated as rotational ductility of beams.

Bending Members: Cross-section Strength and Ductility

The ductile elements of seismic-force-resisting systems and beams designed based on plastic analysis are expected to withstand significant inelastic rotational deformations. In general, the expected inelastic rotational ductility for seismic application is about 6, whereas it must be at least 3 for beams designed based on inelastic methods of analysis. The available inelastic rotational capacity of the beams, however, may be restricted by (a) premature local buckling of the compression flanges, (b) lateral-torsional buckling of the beam, and (c) tension flange rupture. This section discusses the influence of Y/T ratio on the inelastic rotational capacity of “seismically compact” and “compact” beams.

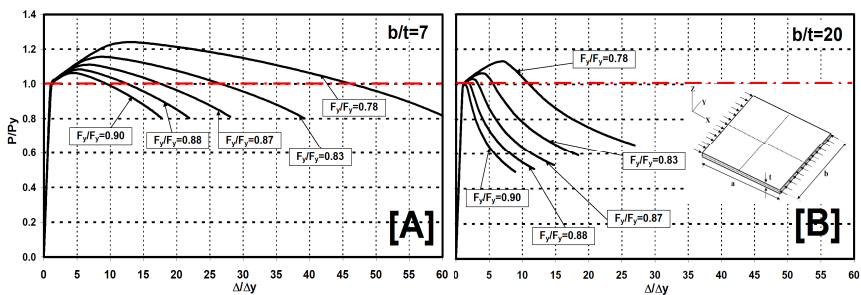


Figure 4. Effects of Y/T ratio on the Strength and Ductility of Plates [A] $b/t = 7$; [B] $b/t = 20$

Local Buckling of the Compression Flanges: The inelastic rotational capacity of a beam is determined by the inelastic deformation capacity of flanges and webs of the section, which in turn depends on (a) the slenderness of the flange and web, and (b) yield ratio of their base material. The impact of flange slenderness and the Y/T ratio may be understood by studying the inelastic behaviour of a plate subjected to uniaxial compression. Figure 4 shows the finite element method based analysis results. The analyses considered a square plate with residual

stresses and initial geometric imperfections subjected to increasing axial loads. The material was idealized as bi-linear elastic-strain hardening steel. Increasing yield stress and tensile strength variables under consideration resulted in Y/T ratio range of 0.78 to 0.90. It is clear that (a) stocky plate ($b/t = 7$) exhibit higher ductility, and (b) increasing Y/T reduces the ductility. Using a definition that adequate rotational ductility corresponds to a compressive ductility of 10, it would appear that such ductility is achievable in a plate made of steel Y/T ratio = 0.90, and having a $b/t = 7$, and in a plate made of steel Y/T ratio = 0.78, and a $b/t = 20$.

Lateral-Torsional Buckling of the Beam: Beams need to be adequately braced against lateral-torsional buckling (LTB) in order to capitalise on the inelastic rotational capacity, if any. It is also known that the local buckling and the lateral-torsional buckling phenomena are coupled in some manner. Thus, the strength and rotational capacity of a steel section not only interactively depend on the slenderness values of the flanges and webs, but, they also interact with the length slenderness. Various research studies on this subject exist (Green, et. al., 2002, Earls, 2000, Dalli and Korol, 1994), and the issue of Y/T ratio on the lateral bracing requirements is not discussed herein in view of the space limitations, except that the laterally unsupported length must be carefully controlled to prevent lateral-torsional buckling in members designed based on plastic analysis and in members designed as ductile energy dissipating element for seismic resistant system.

Tension Flange Rupture: When the tension flange of a beam has fastener holes, its strength and ductility may be controlled due to tension flange fracture. Beam tests were conducted at McMaster University on W 200 x 42 [W 8 x 28] structural beam of A992 steel grade [Measured yield strength 409 MPa, Tensile strength 531 MPa, Y/T ratio = 0.77]. Figure 5 shows the test setup which simulates fully braced beams. Since the beams were closely braced against lateral-torsional buckling, they are expected reach the plastic moment resistance $M_p = 176$ kN.m. The theoretical yield moment resistance $M_y = 160$ kN.m. The test beams were about 3 m (10') long and were subjected to four-point loads. The beams were simply supported at the ends with a pin support and a roller support. Each test beam was aligned, braced, instrumented and was subjected to increasing displacement controlled loads, until failure. The load, vertical displacements and the rotations were monitored. Based on the spacing between supports and loads, the observed parameters were converted to moments and rotations. Figure 5 shows the non-dimensional moment versus non-dimensional rotation characteristics. The rotations were non-dimensionalized with respect to θ_p , which is the plastic rotation corresponding to M_p . The beam bending cases considered are: [a] tension flange with no hole, and [b] two tension flange

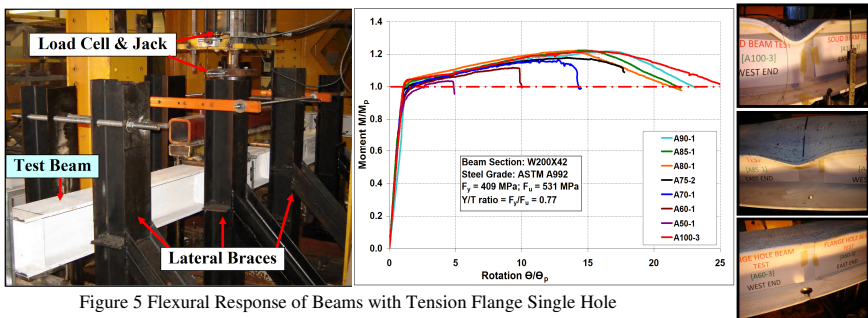


Figure 5 Flexural Response of Beams with Tension Flange Single Hole

holes of varying sizes. The holes were at the mid-span tension flange, and the hole sizes under consideration resulted in $(A_n/A_g) = 0.5, 0.6, 0.7, 0.75, 0.8, 0.85, \text{ and } 0.9$. The specimen designation A-100 indicates a solid beam with no flange hole and, say, specimen designation A-75 indicates that the flange had a net flange area of 75% and 25% of flange area amounted to the hole. From Figure 5, the beam having no flange holes sustained moments higher than the theoretical plastic moment. The experimental ultimate moment was 214 kN.m, which is 22% more than the plastic moment based on mill certificate yield strength of the beam material. Beam rotational ductility can be established from this plot. Using the rotation associated with theoretical moment resistance as reference, the ultimate moment related ductility was $\theta_n/\theta_p^+ = 15.2$, and the total ductility corresponding to the descending strength $\theta_p^-/\theta_p^+ = 25.2$. Obviously, the rotational ductility values are very much different from the strain ductility values associated with the material. As shown in the Figure 5, the solid beam eventually failed due to local buckling of the compression flanges. As the tension flange hole sizes increase the ultimate moment, the ultimate ductility, and the total ductility decrease. Also, the failure mode switches from local buckling of compression flange to tension flange fracture. This change is evident when $(A_n/A_g) > (F_y/F_u) = Y/T$ ratio. Nevertheless, the beam having a large hole, such as the beam with $A_n/A_g = 0.5$, exhibited a total ductility of more than 5, prior to failure due to tension flange net section fracture at the flange hole location.

Treatment of Y/T Ratio in Current North American Steel Design Codes

In Canada, the standard “Limit States Design of Steel Structures- CAN/CSA-S16-01” (CSA, 2005) provides rule and requirements for the design, fabrication and erection of steel structures. The design is based on limit states philosophy. The structural steel shall meet the requirements of CSA standard G40.20/G40.21 (CSA, 2004) or ASTM Standard A992/A992M (ASTM, 2008). Permitted “Structural Quality Steel” includes eight strength levels [ranges from 260 MPa to 700 MPa] and seven steel types; Type W- Weldable steel, Type WT - Weldable Notch-Tough Steel, Type R - Atmospheric Corrosion-Resistant Steel, Type A - Atmospheric Corrosion-Resistant Weldable Steel, Type AT - Atmospheric Corrosion-Resistant Weldable Notch-Tough Steel, Type Q - Quenched and Tempered Low-Alloy Steel Plate, Type QT - Quenched and Tempered Low-Alloy Notch-Tough Steel Plate. Type 350A and 350AT are normally used in bridge construction, whereas 350W grade is often used in building construction. Since, Canada no longer produces 350W grade, in lieu, ASTM A992 and A572 grade 50 are widely used in Canada. In USA, ASTM A992 is the most commonly referenced specification for W-shapes. Consequently, International steel producers roll wide flange beams and other standard structural I - beams to satisfy multiple North American Standards. Any one of the steels discussed above can be used for civil structures designed based on elastic analysis, and for structures in low seismic regions.

Steel used for structures designed based on plastic analysis, however, must be steel with Y/T ratio $= F_y/F_u \leq 0.85$ (CSA, 2005-Clause: 8.6). At normal circumstances all the steels discussed above, except 700Q and 700QT steels, would satisfy this Y/T ratio limit. In order to prevent premature local buckling of elements: the flange slenderness of I-section $b/t \leq 145/(\sqrt{F_y})$, and the web slenderness of I-section $h/w \leq 1100/(\sqrt{F_y})$ (Clause: 11). In order to prevent premature lateral-torsional buckling of the beam, the length slenderness between braced hinge location to the nearest adjacent point on the frame similarly braced must be $(L_{cr}/r_y) \leq (25000 + 15000 \kappa)/F_y$, where, κ is the ratio of the smaller factored moment to the larger factored moment at opposite

ends of the unbraced length (positive for double curvature and negative for single curvature) (Clause: 13.7). Note that accordingly, higher strength steel beams are more prone to lateral-torsional buckling, and as such, would require closely spaced braces. Steel used in the energy dissipating elements associated with earthquake resistant steel frames must use steel with Y/T ratio = $F_y / F_u \leq 0.85$. Further, F_y shall not exceed 350 MPa. In columns in which the only expected inelastic behaviour is at the column base F_y shall not exceed 480 MPa (CSA, 2005- Clause: 27.1.5.1). Beams in ductile moment resisting frames, link beams in ductile eccentrically braced frames, and beams in ductile plate walls must satisfy flange slenderness of $b/t \leq 145/(\sqrt{F_y})$, and the web slenderness of $h/w \leq 1100/(\sqrt{F_y})$. The length slenderness of these beams must be $(L_{cr}/r_y) \leq (17250 + 15500 \kappa)/F_y$ (CSA, 2005- Clause: 27).

The beams having fastener openings can be designed on the basis of the properties of the gross section, provided the fastener holes do not exceed 15% of the gross flange area (CSA, 2005- Clause: 14.1). When fastener holes exceed that 15% exemption, then the excess shall be deducted and modified gross section properties must be established. Tension members, including the braces in tension-only bracings systems of a moderately ductile concentrically braced frame, are designed to resist the least of $A_g \cdot F_y$ and $0.85 \cdot A_n \cdot F_u$. Stated differently, gross section can be used to establish the tensile resistance provided $(A_n/A_g) > 1.176 \cdot (F_y/F_u)$. If this condition is not satisfied, then the tensile resistance must be limited to $\{0.85 (A_n/A_g)/(F_y/F_u)\} \cdot (A_g \cdot F_y)$.

The national standard CAN/CSA-S6-06 "Canadian Highway Bridge Design Code" (CSA, 2006) is used in the design, evaluation, and structural rehabilitation design of fixed and moveable highway bridges in Canada. Though this code gives the fracture toughness requirements for steel used in bridges, it permits the use any structural steel conforming to CSA standard G40.20/G40.21 (CSA, 2004). However, weathering steel members must be Type A - Atmospheric Corrosion-Resistant Weldable Steel, and fracture critical members and primary tension members must be Type AT, Type WT or Type QT steels. The Y/T ratio is mostly ignored in this code, since it promotes treatment of materials as elastic and encourages elastic analysis of bridges, though small deflection theory and large deflection theory must be used for analysis as appropriate. However, since the steel design clauses are duplicated from "Limits States Design of Steel Structures- CAN/CSA-S16-01" (CSA, 2005) the Y/T ratio arises in the tension member design only. Continuous girders with limited inelastic rotational capacity can be provided by limiting the flange slenderness and web slenderness [i.e. $b/t \leq 145/(\sqrt{F_y})$, $h/w \leq 1100/(\sqrt{F_y})$] to the limits associated with the structures designed based on plastic analysis.

The "Specification for Structural Steel Buildings", ANSI/AISC 360-05 (AISC, 2005a) provides the design criteria for steel framed buildings and other structures in the U.S.A. Design, fabrication and erection of structures for low-seismic areas can be solely based on the above specification. However, steel framed buildings and other structures located in high-seismic areas must comply, in addition to the above specification, with the requirements of Seismic Provisions for Structural Steel Buildings, ANSI/AISC 341-05 (AISC, 2005b). AISC 360-05 (AISC, 2005a) permits the use of hot-rolled structural shapes and other structural steel material conforming to a relatively long list of ASTM specifications (ASTM, 2008), however, ASTM A992/A992M is the most commonly referenced specification for W-shapes. Steel used in the seismic load resisting system members, in which inelastic behaviour is expected, should not exceed 345 MPa (50 ksi). Similarly, in structures designed based on an inelastic analysis the F_y of members undergoing plastic hinging should not exceed 450 MPa (65 ksi) (AISC-360-Appendix 1- Clause: 1.2). The

slenderness limits for flanges and webs of members subject to plastic hinging corresponds to “compact” sections; the flange slenderness of I-section $b/t \leq 0.38(\sqrt{E/F_y}) = 170/(\sqrt{F_y})$, and the web slenderness of I-section $h/w \leq 3.76(\sqrt{E/F_y}) = 1682/(\sqrt{F_y})$ (AISC-360-Table B4.1). In order to prevent premature lateral-torsional buckling of the beam, the length slenderness between braced hinge location to the nearest adjacent point on the frame similarly braced must be $(L_p/d/r_y) \leq (0.12 + 0.076 (M_1/M_2))[E/F_y] = (24000 + 15200 (M_1/M_2))/F_y$, where, M_1 is the smaller factored moment and M_2 is the larger moment at the ends of the unbraced length (M_1/M_2 is positive for double curvature and negative for single curvature) (AISC-360-Appendix: 1 Clause: 1.7).

Tension members are designed to the lower value considering limit states of tensile yielding in the gross section ($A_g \cdot F_y$) and tensile rupture in the net section ($A_n \cdot F_u$). However, the associated resistance factors given in AISC-360(AISC, 2005a) are different. Accounting for this difference and modifying it to match the form given in the Canadian code (CSA, 2005), gross section can be used to establish the tensile resistance provided $(A_n/A_g) > 1.2 \cdot (F_y/F_u)$. If this condition is not satisfied, then the tensile resistance must be limited to $\{0.833 (A_n/A_g)/(F_y/F_u)\} \cdot (A_g \cdot F_y)$. According to AISC-360-Clause: F13 (AISC, 2005a), for the beams having a tension flange hole, the possibility of tensile rupture of tensile flange is considered as follows; (a) For steels with $F_y/F_u < 0.8$, tensile rupture does not apply if $(A_n/A_g) > (F_y/F_u)$ and for steels with $F_y/F_u > 0.8$, tensile rupture does not apply if $(A_n/A_g) > 1.1(F_y/F_u)$. Then the moment resistance can be established based on the properties of the gross section. Accordingly, for example, 20% exemption is given for steel with Y/T ratio = 0.8, and the exemption amount decreases with increasing Y/T ratios. (b) If the above conditions are not satisfied, then flexural strength is limited by the tensile rupture of the tension flange which is established for a “compact” section as $\{(A_n/A_g) \cdot (S/Z) / (F_y/F_u)\} M_p$, where M_p is the plastic moment resistance of the beam, and S and Z are the elastic and plastic section modulus of the section, respectively.

Even in high seismic risk zones, the steel seismic load resisting system can be designed and detailed to be (a) ductile, (b) moderately ductile, and (c) limited ductile. In AISC-341 (AISC, 2005b), the corresponding moment resisting frames are referred to as Special Moment Frames (SMF), Intermediate Moment Frames (IMF) and Ordinary Moment Frames (OMF). The SMF are expected to withstand significant inelastic deformations when subjected to the forces resulting from the motions of the design earthquake. Since, Y/T ratio may play a major role in such systems, and in view of the space limitations, here, we discuss the special seismic load resisting systems only. In general, the members associated with these special systems, including the beams and columns of SMF should be “seismically compact” [must satisfy flange slenderness $b/t \leq 0.30(\sqrt{E/F_y}) = 134/(\sqrt{F_y})$, and web slenderness $h/w \leq 2.45(\sqrt{E/F_y}) = 1096/(\sqrt{F_y})$] (AISC-341-Table I-8-1). In addition, both beam flanges must be laterally braced with a maximum spacing of $(L_b/r_y) \leq (0.086)[E/F_y]=17200/F_y$. These slenderness limits are somewhat relaxed for beams associated with moderately ductile and ordinary lateral-force-resisting-systems.

AASHTO LRFD Bridge Design Specifications (AASHTO, 2007) is widely used in the design, evaluation, and structural rehabilitation of highway bridges in U.S.A. Though, steels for application in bridges are covered in ASTM A709/709M, AASHTO specification includes mandatory notch toughness and weldability requirements. The latest AASHTO specification encourages the use of high performance steels (HPS) over conventional bridge steels due to their enhanced properties, and it lists seven grades of steels with minimum yield stress range from 250

– 690 MPa, and corresponding tensile strength range from 400 – 760 MPa. The specified Y/T ratios for these steel grades range between 0.63 and 0.91. For most part, the AASHTO steel design clauses seem to be a mixture of the “Canadian Highway Bridge Design Code” (CSA, 2006), “Specification for Structural Steel Buildings”, ANSI/AISC 360-05 (AISC, 2005a), and perhaps other codes. For example, in the design of tension members, the gross section can be used to establish the tensile resistance provided $(A_n/A_g) > 1.188.(F_y/F_u)$. If this condition is not satisfied, then the tensile resistance must be limited to $\{0.842 (A_n/A_g)/(F_y/F_u)\} \cdot (A_g.F_y)$. Continuous “compact” girders with limited inelastic rotational capacity can be provided in negative flexure (girder over piers) by limiting the flange slenderness and web slenderness to $b/t \leq 0.38(\sqrt{[E/F_y]}) = 170/(\sqrt{F_y})$, and $h/w \leq 3.76(\sqrt{[E/F_y]}) = 1682/(\sqrt{F_y})$, respectively, and by providing compression flange bracing at $(L_b/r_y) \leq (0.12 + 0.076 (M_1/M_2))[E/F_y] = (24000 + 15200 (M_1/M_2))/F_y$. Overall, ductility and Y/T ratio have limited applications in bridge structures.

Concluding Remarks

Civil engineering structures, such as buildings and bridges, are designed to be safe and serviceable during normal use and occupancy and safe from collapse during extreme events like earthquakes. In order to maximize the economical efficiency of structures, prevention of collapse during extreme events like earthquakes relies on the ductility of the structure. In nearly all buildings designed today, survival in large earthquakes depends directly on the ability of the primary structural framing to dissipate energy hysteretically while undergoing large inelastic deformation. To allow for energy dissipation during a large earthquake and/or to allow for redistribution of bending moments in plastically designed members, first the plastic hinges in the member must be able to inelastically rotate without losing the bending capacity of the section. Recent years the specified yield strengths and specified yield-to-tensile strength ratio (Y/T ratio) of structural steel has increased. In general, the Y/T ratio increases with increasing yield stress of steel grade. Y/T ratio also increases with increasing tensile strengths of steel. Y/T ratio is largely irrelevant in the design and behaviour structures that remain elastic even at extreme loadings or do not lend themselves for ductile design. Y/T ratio may become relevant in steel structural systems that are expected to withstand strain-hardening range stresses. Y/T ratio may determine whether a tension member or the tension flange containing fastener holes fail in gross section yielding (ductile) or fail due to net section fracture (brittle). The Y/T ratio may dictate the slenderness limits to prevent local buckling of flanges and webs, and may dictate the lateral brace spacing for the compression flanges of the beams in a ductile lateral-force-resisting-system. In the current North American steel design codes, Y/T is explicitly referenced in the Canadian building design code only, where it states that the steel used in the energy dissipating elements associated with earthquake resistant steel frames, and steel used for structures designed based on plastic analysis must be with Y/T ratio = $F_y/F_u \leq 0.85$. However, in these codes, Y/T ratio indirectly governs the design of tension members and beams, and also Y/T ratio has been implied by limiting the maximum yield strength of the steel that may be used in seismic load resisting system members. Though, it is convenient to consider Y/T ratio as a measure of the ability to withstand plastic loading the real inelastic deformation capacity of steel elements, members, and frames will depend on many other properties as well.

References

- [1] AASHTO (2007), AASHTO LRFD Bridge Design Specifications (SI Units), American Association of State Highway and Transportation Officials, Washington, D.C., USA.
- [2] AISC (2005a), Specification for Structural Steel Buildings, ANSI/AISC 360-05, American Institute of Steel Construction, Inc., Chicago, Illinois, U.S.A.
- [3] AISC (2005b), Seismic Provisions for Structural Steel Buildings, ANSI/AISC 341-05, American Institute of Steel Construction, Inc., Chicago, Illinois, U.S.A.
- [4] Arasaratnam, P. (2008), "Effects of Flange Holes on Flexural Behavior of Steel Beams", Ph.D. Thesis, McMaster University, Hamilton, Ontario, Canada, p.xxv, p. 350.
- [5] ASTM (2008), Annual Book of ASTM Standards, Section: 1- Iron and Steel Products, [A370-07-Standard Test Methods and Definitions for Mechanical Testing of Steel Products, A36-Standard Specification for Carbon Structural Steel, A572/A 572M -Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel, A709/A 709M-Standard Specification for Structural Steel for Bridges, A913/A913M- Standard Specification for High-Strength Low-Alloy Steel Shapes of Structural Quality Produced by Quenching and Self-Tempering Process (QST), A992/A 992M -Specification for Structural Steel Shapes] ASTM International, PA, U.S.A.
- [6] Bartlett, F.M., Dexter, R.J., Graeser, M.D., Jelinek, J.J., Schmidt, B.J., and Galambos, T.V. (2003), Updating Standard Shape Material Properties Database for Design and Reliability, Engineering Journal, First Quarter, pp.2-14.
- [7] Brockenbrough, R.L. and Merritt, F.S. (2006), Structural Steel Designer's Handbook, 4th Edition, McGraw-Hill, New York, U.S.A.
- [8] Brockenbrough, R.L. (2003), MTR Survey of Plate Material Used in Structural Fabrication, Engineering Journal, AISC, First Quarter, pp. 42-49.
- [9] CSA (2004), General Requirements for Rolled or Welded Structural Quality Steel/ Structural Quality Steel CAN/CSA G40.20-04/G40.21-04, Canadian Standards Association, Mississauga, Ontario, Canada.
- [10] CSA (2005), Limit States Design of Steel Structures, CAN/CSA-S16-01, Canadian Standards Association, Mississauga, Ontario, CANADA.
- [11] CSA (2006), Canadian Highway Bridge Design Code, CAN/CSA-S6-06, Canadian Standards Association, Mississauga, Ontario, CANADA.
- [12] Dalli, M.L. and Korol, R.M. (1994), Local Buckling Rules for Rotation Capacity, Engineering Journal, AISC, Second Quarter, pp. 41-47.
- [13] Dexter, R.J., Graeser, M., Sarri, W.K., Pascoe, C., Gardner, C.A., and Galambos, T.V. (2000), Structural Shape Material Property Survey, University of Minnesota, MN, U.S.A.
- [14] Dexter, R.J., Altstadt, A., and Gardner, C.A., (2002), Strength and Ductility of HPS70W Tension Members and Tension Flanges With Holes, Research Report, University of Minnesota, Minneapolis, MN, USA.
- [15] Frank, K.H. and Read, D.R. (1993) Statistical Analysis of Tensile Data for Wide-Flange Structural Shapes, University of Texas at Austin, Austin, TX, U.S.A.
- [16] Galambos, T.V. and Ravindra, M.K., (1978), Properties of Steel for Use in LRFD, Journal of the Structural Division, ASCE, Vol. 104, ST9, September, pp. 1459-1468.
- [17] Green, P.S., Sause, R., and Ricles, J.M. (2002), Strength and Ductility of HPS Flexural Members, Journal of Constructional Steel Research, Vol.58, pp.907-941.

- [18] Earls, C.J. (2000), Influence of Material Effects on Structural Ductility of Compact I-Shaped Beams, *Journal of Structural Engineering*, ASCE, Vol. 126, No.11, pp.1268-1278.
- [19] Jaquess, T.K. and Frank, K.H. (1999), Charecterization of the Material Properties of Rolled Sections, Technical Report for SAC Joint Venture, University of Texas, Austin, TX, U.S.A.
- [20] Kayser, C.R., Swanson, J.A. and Linzell, D.G., (2006), Characterization of Material Properties of HPS-485W(70W) TMCP for Bridge Girder Applications, *Journal of Bridge Engineering*, Vol. 11, No. 1, pp.99-108.
- [21] Kennedy, D.L.J. and Gad Aly, M. (1980), Limit States Design of Steel Structures-Performance Factors, *Canadian Journal of Civil Engineering*, Vol. 7, pp.45-77.
- [22] Kuwamura, H. and Kato, B., (1989) Inelastic Behaviour of High Strength Steel Members with Low Yield Ratio, *Pacific Structural Steel Conference*, Australian Institute of Steel Construction, Australia, pp. 429-437.
- [23] Lilly, S.B., and Carpenter, S. T., (1940), Effective Moment of Inertia of a Riveted Plate Girder, *Transactions, American Society of Civil Engineers*, Paper No. 2089, pp.1462-1517.
- [24] Sause, R. and Fahnestock, L.A., (2001), Strength and Ductility of HPS-100W I-Girders in Negative Flexure, *Journal of Bridge Engineering*, ASCE, Vol.6, No.5, pp.316-323.
- [25] Sooi, T.K., Green, P.S., Sause, R. and Ricles, J.M., (1995), Stress-Strain Properties of High-Performance Steel and the Implication for Civil-Structure Design, *Proceedings of the International Symposium on High Performance Steels for Structural Applications*, Cleveland, Ohio, USA, pp.35-43.
- [26] Yamanouchi, H., Kato, B., and Aoki, H. (1990) Statistical Features of Mechanical Properties of Current Japanese Steels, *Materials and Structures*, Vol. 23, pp.305-315.