MICROALLOYED STEELS IN BUILDING CONSTRUCTION

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Abstract

Reinforced concrete buildings or steel buildings, which are subjected to numerous loads in their life time, must be designed to meet the performance objectives during construction and during the whole life of the structure, including the capacity to resist catastrophic loading events. Recent advances in steelmaking technologies have resulted in microalloyed steel products with superior properties which can easily and economically meet these multiple building performance demands. The first part of this paper summarizes the structural design concepts and the general design considerations associated with building structures. Seismic design provisions and the capacity design concepts are explained. The second part of the paper looks at the function of rebar, historical development of rebars, and acceptance criteria for reinforcing steels in North American building codes and construction. The third part of the paper reviews the historical development of structural steels, and the North American material specifications for structural steels. The stability, ductility, and the fire resistance requirements of structural steel members and the relevance of microalloyed structural steels in building construction are also discussed. The paper argues that niobium microalloying is the key to achieving superior properties in such steels.

Introduction

Buildings, from houses to hospitals, shopping malls and office towers, play an essential role in the development and maintenance of contemporary societies. Such structures, which are subjected to an array of loads, however, must be designed to satisfy the performance requirements prescribed by owners and occupiers. The modern design of buildings may need to meet multiple performance objectives defined for different loads ranging from regular loads due to use and occupancy to rare loads such as those from earthquake, fire, hurricane and blast. The majority of the building structures, including industrial buildings, are constructed either using concrete, reinforced by deformed steel bars, or using steel, utilizing structural shapes and plates. Currently the construction steel sector accounts for about 60% of the crude steel production. The reinforcement bar products represent about 35% of the construction segment, whereas the share of the structural steel plates and sections may be about 20% [1]. Improvements in steelmaking technologies have resulted in steel products, such as niobium microalloyed steels, with superior strength, ductility, weldability, toughness, fire resistance, etc. For wider acceptance and use of such microalloyed steels in building construction, however, they must be able to satisfy the building performance demands with some advantage over traditional steels. This paper focuses on the historical developments, acceptance criteria in current building codes, and the impact of microalloying of the deformed steel bars for reinforcement in concrete buildings and the hot-rolled structural steel sections in steel buildings.

Building Structural Design Concepts

The building design method used in the 1950s, known as the "Working Stress Design," also known as the "Allowable Stress Design," was elastic based in which the structural members are designed so that the actual stress caused by the service and environmental loads is limited to an allowable stress defined by the strength of material (ie. yield stress or buckling stress for steel members and compressive strength for concrete) divided by a factor of safety. At that period, the seismic risk was incorporated by considering a lateral force equal to 2-5% of the building weight [2]. Obviously, the allowable stress design method, which is still in use in some other engineering fields, is simple but it has little rational basis, and does not consider the variability associated with the loads and resistances and the true strength and deformability of materials.

Almost all countries around the world currently use the "Limit States Design" method for building design. Accordingly, the two primary "structural usefulness" limit states are: the *strength limit state*, which defines the safety of the structure, and the *serviceability limit state*, which defines the occupancy performance (ie. deflection, vibration, drift, etc.). This probability based method incorporates the variability associated with the loads through a load factor α_i (eg. dead load - 1.25, live load due to use and occupancy - 1.5, snow load - 1.5, wind load - 1.4, etc.) and to account for uncertainties in the material strength, dimensions and the workmanship a resistance factor Φ (structural steel members - 0.9, reinforcing bars - 0.85, concrete - 0.65) is used [3]. Though the variability in loads and resistance were measured, the actual values for these factors were derived to achieve an acceptable level of safety of the structure, reflected through a reliability index β. For example, in the National Building Code of Canada, NBCC-2010 [3] load factors and resistance factors for structural members are based on a target reliability index β=3.0 for a service life of 50 years, which translates to an annual probability of failure of 2.8×10^{-5}. Obviously, the environmental loads such as wind and snow loads will fluctuate during the target life of the buildings (50 years), thus the corresponding design loads are based on a matching return period of 50 years. Therefore, the fundamental strength (ultimate) limit state design criterion is stated as $\Phi R_n \geq \sum \alpha_i Q_i$, where Q_i are the various loads and associated load combinations and R_n is the nominal resistance of the structural member. The serviceability limit state criterion, however, is not yet based on a probabilistic concept, and in that respect it is similar to the allowable stress design.

In the meantime, the research community began to quantify the factors influencing the seismic loads on buildings, namely, site specific seismic hazard (reflected by accelerations and velocities for the location, and soil conditions), natural period of the structure, higher mode effects, etc. Furthermore, seismic ground motions were treated as extremely rare or accidental actions and the performance objective was to prevent collapse of the structure during such major earthquakes (thereby minimizing the loss of life and assets). Thus, the design seismic loads have a probability of exceedance of 2% in 50 years (2475-year return period). The seismic weight of the building is calculated as the sum of the actual gravity loads (ie. no load factors are applied to seismic design): 100% dead load + 50% live loads + 25% snow load to reflect the "probable" gravity loads expected to be acting when the earthquake occurs [2,3].

The seismic design provisions permit reliance on the inherent ductility of a structural system (ie. the system has the capacity to dissipate energy by undergoing inelastic deformation) to reduce the design seismic forces, which leads to smaller structural members and lower reinforcement bar quantities for the whole building system. For example, if the elastic seismic force is V_E, the design seismic force can be $V_D=V_E/(R_dR_o)$, where R_d is the ductility related factor representing the inelastic deformational capability, while R_o is the overstrength-related force modification factor, reflecting the inherent reserve strength in structural systems, Figure 1. The greater the ability of the structural system to dissipate energy, the higher is the assigned value of R_d. The NBCC-2010 code [3] has assigned a highest ductility related force reduction factor R_d of 5.0 for steel ductile moment resisting frames and to ductile steel plate shear walls (R_o for steel frames is 1.5 and for shear walls is 1.6). Similarly, the largest seismic force modification factor R_d of 4.0 has been assigned to steel reinforced concrete structural systems consisting of ductile moment resisting frames and ductile coupled walls (R_o for these systems is 1.7). Accordingly, the seismic forces can be reduced by 6-8 fold. However, implicit in the force-reduction factor approach is the assumption that the particular structural system can be designed to have a corresponding displacement ductility capacity factor of 6-8 [3].

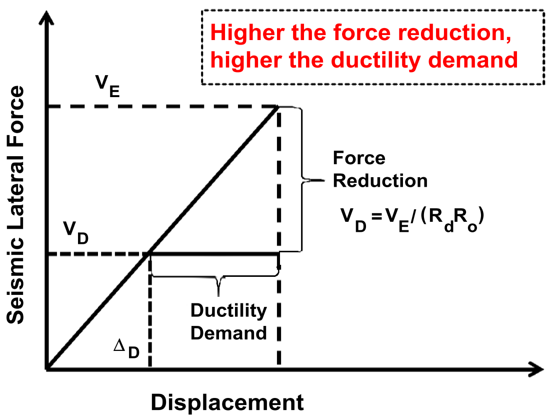


Figure 1. Force based seismic design and the “equal displacement” concept.

In recent years, however, the design for seismic resistance has been undergoing reappraisal, primarily because during recent earthquakes, the level of damage to non-structural and structural elements, economic loss due to loss of use, and cost of repair were unexpectedly high, even though the structures did not collapse. An emerging seismic design concept is known as “Performance-based Design,” which refers to the coupling of expected performance level with expected levels of seismic ground motions. As indicated earlier, the current seismic design codes are geared towards a performance level of near collapse during (extremely rare) earthquakes; the damage may be severe, but structural collapse is prevented by limiting maximum inter-story deflections, due to nonlinear seismic response, to 2.5%. This new concept attempts to bridge the gap between regular occupancy and near collapse criteria. Some of the seismic performance objectives (level of protection) a client may set are: fully operational (immediate occupancy with negligible or no damage), operational (continues in operation with minor repairable damage), life safe (life is protected, however, damage to structure is moderate to extensive and may be irreparable) and near collapse. These performance levels can be associated with different seismic hazard levels: frequent (probability of exceedance 50% in 30 years {return period = 43 years}), occasional (50% in 50 years {return period = 72 years}), rare (10% in 50 years {return period = 475 years}), etc. Future seismic design of buildings may be based on defined multiple performance objectives and associated earthquake hazard levels, and research is ongoing on the development of relationships between quantifiable performance criteria and the structural response parameters such as stress, strain and displacements.

General Design Requirements for Buildings

As a basic design requirement, the members and the joints of building structures must be designed to have sufficient structural capacity, structural stiffness, and ductility, to safely and effectively resist all effects of loads, and other influences that may reasonably be expected during fabrication, construction and the useful life of the structure. The structural system of the building, however, may be divided into “seismic-force-resisting-system (SFRS)” and gravity load resisting system. SFRS participates in the seismic resistance as well as in gravity load resistance, however, the gravity load resisting system does not resist the seismic forces, but must maintain its gravity load carrying capacity as the system undergoes the required lateral deformations during an earthquake.

The gravity load resisting system is designed in the usual manner to satisfy the strength limit state and the serviceability limit state, without any special requirements for energy dissipating capacity, though it is desirable to have a minimum level of ductility in any structure. Current design codes assure sufficient load carrying capacity (strength limit state) by ensuring the factored resistance of the members and joints is greater than the effects of the most critical combinations of factored loads acting on them. For a number of occupancy related building design situations the structural stiffness governs the final solution. Serviceability measures, such as beam deflections, inter-story and overall drift due to wind, floor vibration, depend on the stiffness of the structural members, and by extension the stiffness of the structural system.

North American seismic design of both steel structural systems and steel reinforced concrete systems is based on “capacity design” principles. In capacity design; (a) specific elements or mechanisms, and their corresponding locations are chosen and then designed or detailed to dissipate energy, and (b) all other structural elements in the SFRS are then provided with sufficient reserve capacity to ensure that the selected energy-dissipating mechanisms are maintained in the selected locations without surprise formation of any additional mechanisms. In essence, first, the ductile energy dissipating element locations must be clearly identified, and then must be designed to sustain several cycles of inelastic loading with minimum strength and stiffness deterioration, whereas all other members must be designed to remain basically elastic during seismic ground motion. Essentially, a proper strength hierarchy must be provided in the SFRS to constrain inelastic response to these ductile elements. For example, the current design philosophy requires that the plastic hinges in a ductile moment resisting frame be developed at the beam ends, a short distance from the face of columns. Plastic hinges in beams are capable of tolerating larger rotations, thereby dissipating seismic energy more efficiently than hinges in columns. Besides, hinges in columns can lead to a sideways failure mechanism which is undesirable. In essence, ductile moment resisting frame design must be based on the "weak beam-strong column" idea. Formation of beam hinges at the desired location in a steel frame is possible either by strengthening the beams near the columns or by locally weakening the beams at selected locations some distance from the columns. Though the ductile plastic hinges in reinforced concrete frames are often designed to be formed at the beam ends adjacent to the face of the column, as shown in Figure 2, such plastic hinge regions can be deliberately relocated away from the column. Obviously, these plastic hinge locations in the ductile moment resisting reinforced concrete frames must incorporate appropriate reinforcement details, which are discussed in the next section, so as to achieve the necessary rotation capacity. Current seismic design standards expect a rotational ductility of about 7-9 in those ductile elements. One important point worth emphasizing to all structural and material engineers is that in seismic resistant building design sufficient ductility is required only at the predetermined energy dissipating elements ("structural fuse") and all other elements are designed as regular elastic members with a minimal level of ductility.

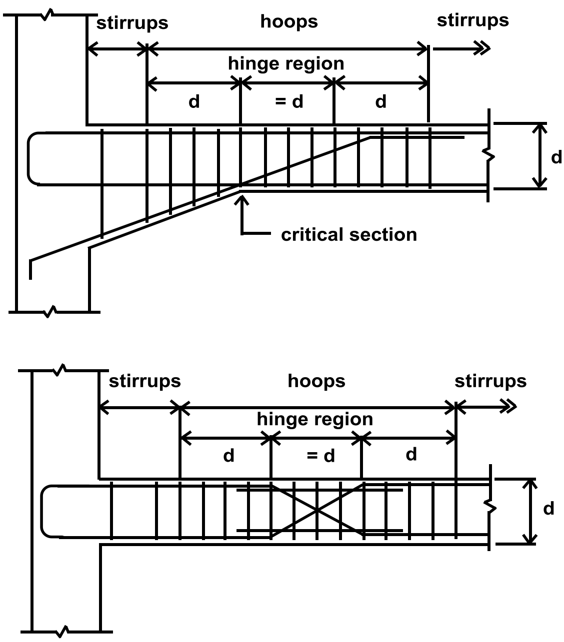


Figure 2. Examples of plastic hinges located away from column faces [4].

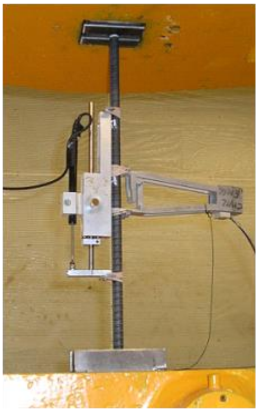
Reinforced Concrete Building Structures

Concrete has been used for construction for thousands of years, though in more primitive forms than at present. Worldwide, concrete construction is very popular, primarily because most of the constituent materials, except for cement and additives, are usually readily available at low cost locally or at a short distance from the construction site. Current buildings usually use a normal density concrete with compressive strength f'_c between 25 and 40 MPa, though some buildings have used high-strength concrete having a compressive strength in excess of 100 MPa. However, the tensile strength of concrete is so small (3-4 MPa) that it is ignored in structural design. Due to its compressive strength concrete is more suitable for members primarily subjected to compression, such as columns and arches. Further, unlike steel, concrete is a brittle material and has no ability to undergo plastic deformation without fracture. Thus, reinforcing steel bars are required to provide tensile resistance in concrete members subjected to tensile stresses, such as beams, beam-columns, etc. Use of reinforcing steel also improves the ductility capacity of such members. Reinforcing bars are used in columns in order to increase the capacity while reducing the overall size of the column. Since almost all concrete construction includes embedment of reinforcing steel, simply, concrete construction implies steel reinforced concrete construction.

The reinforcing bars for concrete, as we know them today, came about 100 years ago, and were plain, deformed or cold-twisted bars of round or square cross-sectional shapes, and structural grade bars with a minimum yield strength of 228 MPa were normally used. Currently, in Canada the primary reinforcements in concrete construction must be deformed bars only, satisfying the requirements of the standard CSA G30.18 "Carbon steel bars for concrete reinforcement" [5]. Even though this standard specifies two minimum yield strength levels, ie. 400 MPa and 500 MPa, currently, 400 MPa deformed rebar is often used in building construction. This standard [5] specifies requirements for regular bars, which are intended for general applications, and for weldable bars, which are appropriate for applications requiring enhanced weldability, enhanced ductility, restricted mechanical properties or chemical composition. The corresponding codes in the USA are A615/A615M [6] and A706/A706M [7] which are for carbon-steel bars and low-alloy bars, respectively. The strength levels of the bars in the USA range from 280 MPa to 550 MPa.

In North America, the chemical composition and mechanical properties requirements for regular and weldable steel bars for concrete reinforcement [5-7] are not as restrictive as A992 for structural steels [8] as discussed in the following section. The manufacturer has the liberty to choose the alloying elements which may include niobium. Focusing on 400 MPa grade deformed bars as widely used in Canada; <u>For regular bars</u> the only chemical composition limit is a maximum phosphorus level of 0.06% and the required tensile properties are minimum yield strength 400 MPa, minimum tensile strength 540 MPa, minimum tensile/yield ratio of 1.15 and minimum elongation over 200 mm of 7% for larger bars and 10% for smaller bars. <u>For weldable bars</u>, however, limits are specified for elements - carbon (0.30%), manganese (1.60%), phosphorus (0.035%), sulfur (0.045%) and silicon (0.50%). The carbon equivalent calculated as $CE = C + (Mn/6) + (Cu/40) + (Ni/20) + (Cr/10) - (Mo/50) - (V/10)$ must not exceed 0.55%. The required tensile properties for weldable bars are minimum yield strength 400 MPa, maximum yield strength 525 MPa, minimum tensile strength 540 MPa, minimum tensile/yield ratio of 1.15 and a minimum elongation over 200 mm of 12% for larger bars and 13% for smaller bars. The above values are slightly different from the ASTM requirements [6,7]. A difference worthy of note is that, although there is no restriction on the tensile/yield ratio for ASTM regular bars [6], the ASTM low alloy bars [7] are expected to have a minimum tensile/yield ratio of 1.25.

Recently, North American produced microalloyed reinforcing bars were tested for tensile stress-strain relationships. The test specimens were cut from 20 M (~20 mm diameter) rebars, whose chemical compositions were obtained from the corresponding mill certificates. Figure 3 shows the bar during testing and the representative non-dimensional stress-strain relationships for two of the bars; one with Nb+V and the other with V only. The test program considered three identical bars and consistent stress-strain relationships were observed. Based on the comparison above, Nb+V microalloyed steel exhibited a tensile to yield ratio of 1.68 and a strain ductility (strain at fracture/yield strain) of 45, which is significantly higher than the corresponding values exhibited by the V only reinforcing bars, Figure 3.



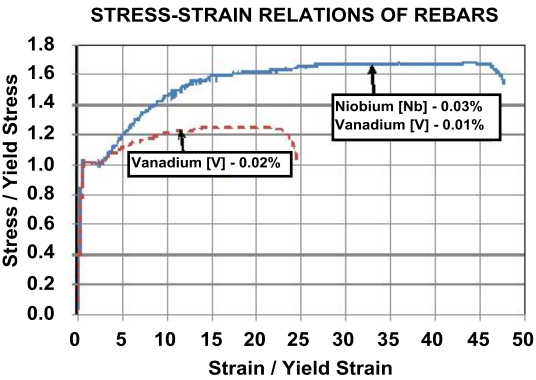


Figure 3. Tensile stress-strain relations for microalloyed reinforcing bars.

The design capacity of a reinforced concrete member is typically based on the following assumptions:

- Concrete in compression reaches its strength of f'_c, with a maximum usable compressive strain of 0.0035;
- Concrete will not carry any tensile stress;
- At the ultimate tensile limit state, primary reinforcing bars reach their yield strength, f_y;
- The factored axial resistance of concentrically loaded reinforced concrete column, since it consists of concrete and steel rebars, is taken as $Pr_{max}=0.80\ [\Phi_c(0.8\ f'_c)\ (A_g-A_{st}) + \Phi_s\ A_{st}\ f_y]$, where A_g and A_{st} are the gross cross-sectional areas of concrete and the bars, respectively;
- More than 4% of longitudinal reinforcements ($A_{st}>0.04A_g$) may result in construction difficulties in placing and compacting concrete.

Obviously, then, high strength rebars, such as Nb-bearing microalloyed steel rebars, can increase the column capacity and/or reduce member sizes and ease construction. Note that all concrete columns must have reasonably spaced ties in order to provide confinement, which means providing lateral support to the rebars and so prevent them from buckling, and to create minimal ductility.

Flexural members, such as beams and girders, are designed to be “under-reinforced” in order to generate reasonable ductility. “Under-reinforced” implies that the amount of primary reinforcement is such that these bars will yield well before the ultimate state of the member is reached. Under-reinforcement criteria depend on the strengths of the steel bars and of the concrete, however, approximately 1.5% content of a 400 MPa steel in a 35 MPa concrete beam would ensure steel yielding well before the ultimate limit state. The moment resistance of tension reinforced beams is approximately $Mr \approx \Phi_s\ A_{st}\ f_y\ (0.9\ d)$, where d is the distance from the extreme compression fiber to the centroid of longitudinal tension reinforcement. High strength rebars may be used to increase the moment resistance, however, because the area of steel must also be limited to maintain the under-reinforced condition there may not be any significant increase in the overall moment resistance. Alternatively the moment resistance of members can be increased by providing somewhat equal amounts of steel both in the compression side and tension side (doubly reinforced beams).

Regularly spaced steel stirrups are usually needed in reinforcing beams to provide the necessary shear resistance. Serviceability limit states, such as deflection, need to be satisfied. Since reinforced concrete beams crack in tension at relatively low loads, the flexural stiffness based on a cracked section, for the portion of the beam that is cracked, must be considered in calculating immediate and long term deflections. Long term deflections in doubly reinforced beams are considerably lower than in the singly reinforced beams. The strength of rebars has no influence on deflection.

The resistance of a reinforced concrete member subjected to a bending moment and an axial load (beam-column) is not so easy to establish, since it may be subject to tensile stresses and cracking, depending on the relative magnitude of the axial force. However, it can be understood that the axial load capacity of a beam-column is reduced in the presence of a bending moment and *vice versa*.

In a seismic design based on the capacity design principle, a proper strength hierarchy must be provided to confine inelastic action to the energy dissipating elements (plastic hinges). Here we elaborate this point using a design of a ductile moment resisting reinforced concrete frame in accordance with the weak-beam-strong-column philosophy. The design sequence is as shown in Figure 4 [1]. The beams at plastic hinge locations are proportioned first, such that their dependable flexural strength is as close as possible to moments due to seismic load combinations. The following reinforcing details are necessary at the plastic hinge in order to ensure adequate ductility and stable hysteretic response. Due to load reversals, the beam at a plastic hinge must be doubly reinforced. Since the top and bottom steel bars are expected to yield, the bars must be fully anchored on either side of the plastic hinge. When bars are in compression they must be prevented from premature buckling. To this end, each longitudinal bar at the plastic hinge must be restrained against buckling by closely spaced hoop reinforcements (ties). The maximum spacing of the hoops is quite crucial at the plastic hinge locations and proper detailing of the plastic hinge is a significant requirement in seismic design [2]. Since shear failure is brittle, the shear strength at all sections along the beams is designed to be higher than the shear corresponding to probable moments at the beam plastic hinge, which are calculated as theoretical moment resistances (using resistance factors of 1.0) when the steel stress is $1.25f_y$, where 1.25 is a factor for possible overstrength of rebars [3]. The weak beam/strong column hierarchy must be achieved at the beam column intersection. For a joint having columns above and below and beams on either side, considering the joint moment equilibrium,

$$M_{nc1}+M_{nc2} \geq M_{pb1}+M_{pb2}, \tag{1}$$

where M_{pb1} and M_{pb2} are the probable beam moments, and M_{nc1} and M_{nc2} are the necessary factored moment resistances of the columns [4]. Column ties are detailed considering the shear arising in columns due to the probable moments in beams, confinement requirements, and stability of compression reinforcements [5]. Because beam-column joints are poor energy dissipaters, it is preferable that joints remain in the elastic range. Due to seismic actions, joints experience high shear, which can be resisted through joint concrete and joint shear reinforcements. Readers interested in additional details related to these design and detailing requirements may consult a concrete design code [4] or other books [9] on the seismic design of concrete buildings.

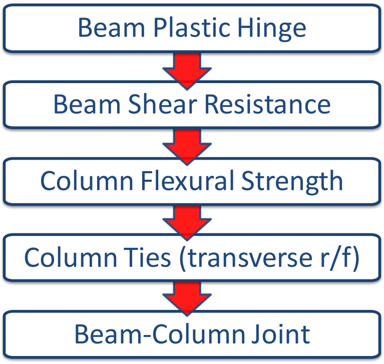


Figure 4. Capacity design sequence of ductile moment resisting reinforced concrete frame.

Reinforced concrete building design, especially seismic design of such structures, is still evolving based on research studies all around the world. Uncertainties and variability associated with the structural configuration, seismic loads, concrete material properties and bar detailing present a design challenge. The desirable characteristics of reinforcing steel for seismic plastic hinges, in order to reduce the over-strength factor (current value of 1.25), are (a) low variability of actual yield strength from specified yield strength, and (b) a long yield plateau followed by gradual strain hardening [9]. Niobium microalloyed rebar has the potential to satisfy these requirements.

Table I. Chemical Composition and Mechanical Properties of Structural Steels

Item	CSA G40.21 – 350W[10]	A572 Grade 50[11]	ASTM A992[8]
Chemical Composition wt.%			
Carbon (C)	0.23 max	0.23 max	0.23 max
Chromium (Cr)	Not specified	Not specified	0.35 max
Copper (Cu)	Not specified	Not specified	0.60 max
Manganese (Mn)	0.5 – 1.50	1.35 max	0.5 – 1.60
Molybdenum (Mo)	Not specified	Not specified	0.15 max
Nickel (Ni)	Not specified	Not specified	0.45 max
Phosphorus (P)	0.04 max	0.04 max	0.035 max
Silicon (Si)	0.40 max	0.15 – 0.40	0.40 max
Sulfur (S)	0.05 max	0.05 max	0.045 max
Vanadium (V)	0.10 max	0.01 – 0.15	0.15 max
Vanadium (V) + Niobium (Nb)	0.10 max	0.02 – 0.15	0.15 max
Niobium (Nb)	0.10 max	0.005 – 0.05	0.05 max
Carbon Equivalent			0.45 max
Mechanical Properties			
Yield Point	345 MPa min	345 MPa min	345 – 450 MPa
Tensile Strength	450 – 650 MPa	450 MPa min	450 MPa min
Yield-to-Tensile Ratio	Not specified	Not specified	0.85 max
Elongation in 200 mm	19% min	18% min	18% min
Elongation in 50 mm		21% min	21% min

About 100 years ago, the plate and structural sections used in steel building construction had a yield strength of 190-210 MPa and a tensile strength of 380-450 MPa. About 50 years ago, the first high strength low alloy (HSLA) steels with a yield strength of about 350 MPa and a tensile strength of about 480 MPa came into use for building construction. In Canada, the chemical and mechanical properties of structural steels for general construction and engineering purposes is covered by the standard CSA G40 21 "Structural Quality Steel" [10], which permits eight yield strength levels ranging from 260 MPa to 700 MPa. However, currently, in Canada, often Type 350W steel grade is specified for building construction. In the USA, ASTM A992 [8] is the most commonly referenced specification for steel shapes. Since Canada no longer produces 350W grade, in lieu, ASTM A992 [8] and A572 grade 50 [11] are accepted. Consequently, international steel producers roll wide flange beams and other standard structural I - beams to satisfy multiple North American Standards. Table I shows the chemical composition and the mechanical properties as per standards G40.21-350W [10], A572 grade-50 [11], and A992 [8].

From the structural performance standpoint the advantage of ASTM A992 structural steel [8] is the tighter material definition; Yield Point = 345 – 450 MPa (a maximum yield strength of 480 MPa is permitted for specimens from the web), Minimum Tensile Strength = 450 MPa, Maximum Yield-to-Tensile Ratio = 0.85, Minimum Elongation over 200 mm = 18% and a maximum Carbon Equivalent of 0.45% (0.47% for shapes with flange thickness over 50 mm) (Carbon Equivalent = C + (Mn)/6 + (Cr+Mo+V)/5 + (Ni+Cu)/15).

The specified upper limit on yield strength reduces the possibility of the weld metal becoming under-matched, which in turn reduces the potential for brittle fracture. The specified yield range ensures better seismic performance of steel structures based on "capacity design" principles. Minimum elongation and maximum yield-to-tensile ratio specifications also ensure adequate ductility. The limit of carbon equivalent ensures adequate weldability. ASTM A992 [8] permits a maximum of 0.05% of niobium. The seismic design requirements in Canada [12] include that the yield strength of steel used in the energy-dissipating elements must not exceed 350 MPa and yield-to-tensile ratio must be less than 0.85 (T/Y $\geq$1.17).

Figure 5 shows a tension coupon during testing and the tensile stress-strain results for three identical specimens. The test specimens were cut from the flange of a A992 steel section, whose mill certificate indicated microalloying elements of 0.021% niobium and 0.002% vanadium. Consistent stress-strain relationships were observed. Based on these tests the following material properties were derived for this steel; Yield Point = 445 MPa, Tensile Strength = 577 MPa, Yield-to-Tensile Ratio = 0.77, Elongation over 200 mm = 20.8%, Modulus of Elasticity E = 203 GPa; strain ductility at ultimate load and at fracture are 63 and 95, respectively.

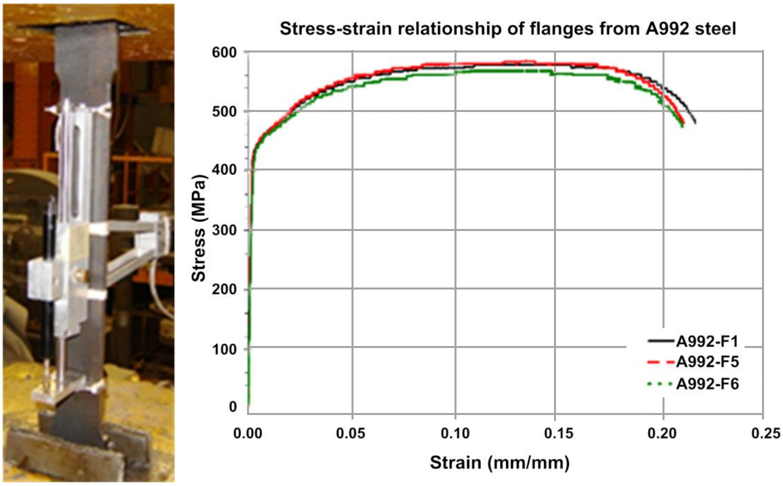
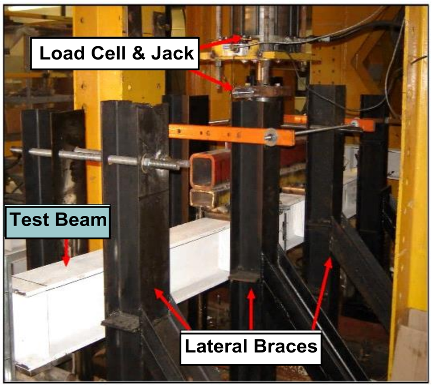


Figure 5. Tension test results.

Member resistances used for strength limit state are typically based on the yield stress of the steel and to a small extent on the ultimate tensile strength. In simple terms, assuming buckling does not govern failure, the resistance of a steel member is the product of its geometric sectional property and the strength of the material ($A.F_y$, $S.F_y$, $Z.F_y$, etc.). Obviously, an increase in yield strength translates into a reduction in sectional size. Through improvements in steelmaking technologies the industry is already producing high strength steels, thus, adequate strength can be supplied relatively easily. It is well known that microalloyed steels have better combinations of strength, toughness and ductility than carbon-manganese steels. Over the last two decades the production of niobium microalloyed structural steel has increased substantially. Niobium is often added at a level of 0.025-0.035% to promote strong grain refinement, enhance hardenability and increase the yield strength through precipitation. A yield strength of 600 MPa or higher is easily achieved in Nb-bearing steels [9]. Thus, the strength of Nb-bearing steels is not an issue for building construction.

Serviceability limit state control depends on a structure's stiffness. Though the actual stiffness of a structural system depends on the member arrangement and the connections between the members, in simple terms, the stiffness of a steel member is the product of the modulus of elasticity of steel and the geometric sectional property (E.A, E.I, etc.). Here, E is the elastic modulus. Similarly, the resistance against local buckling and overall buckling largely depends on the member stiffness rather than the member strength. Since the E value is around 200,000 MPa for all steels, unfortunately, the advantage of high strength often cannot be readily utilized in such situations. Concrete encased or concrete filled steel sections, buckling-restrained braces, and similar innovative construction techniques can increase the sectional stiffness and enable the higher strength of microalloyed steels to be harnessed in building construction. The building construction industry would immensely benefit if the stiffness (modulus of elasticity) of steel members could be increased. Steel research publications are silent on this mechanical property, and the structural engineers assume the universal value of E = 200,000 MPa for steels, including Nb-bearing steels. Even a 10-20% increase in the modulus of elasticity of steels would have far reaching benefits in applicability and use of such steels in building construction.

Steel section strength and rotational ductility are often more pertinent to structural design than the underlying material properties. A structural section's characteristics depend on the material properties, as well the cross-sectional geometry. Figure 6 shows the test set-up and the moment-rotation relationship for a W 8 x 28 [W 200 x 42] (8" wide at 28 lb/ft) structural beam made of A992 steel. The test set-up simulated a fully braced beam, and the beam under consideration was expected to reach its plastic moment capacity. The test beam was about 3 m long and was subjected to four-point loading. The beam was simply supported at the ends with a pin and roller supports. The test beam was aligned, braced, instrumented and was subjected to increasing displacement controlled loads, until failure. The beam eventually failed due to local buckling of the compression flanges. Lateral-torsional movements were also observed at high rotations. The load, vertical displacements and the rotations were monitored. Based on the distances, the observed parameters were converted to moments and rotations. From Figure 6, the moment carried by the beam is higher than the theoretical plastic moment. The experimental ultimate moment was 214 kN.m, which is 18% more than the plastic moment based on the mill certificate yield strength of the beam material. Beam rotational ductility can also be established from this plot. Using the rotation associated with theoretical moment resistance as reference, the ultimate moment related ductility was θ_m/θ_{p+} = 12.5, and the ductility corresponding to the descending strength θ_{p-}/θ_{p+} = 16.6. Obviously, the rotational ductility values are very much different from the strain ductility values. However, the rotational ductility exhibited by the beam is many times more than the ductility values required by the steel design standards [12,13].



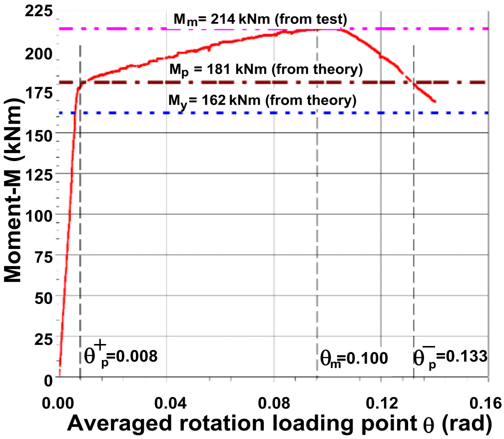


Figure 6. Beam test results.

In seismic resistant building design significant ductility is required of the predetermined energy dissipating elements. The Canadian steel design code permits the use of CSA G40.20/G40.21 or ASTM A992/A992M steels with yield strength less than 350 MPa for such beams. Furthermore, the yield strength of SFRF columns expected to have an inelastic hinge at the base may be as high as 480 MPa. The beams must be compact sections (capable of achieving plastic moment capacity, high ductility and hysteretic energy dissipation capacity prior to local buckling) and must be adequately laterally braced for plastic hinge. The capacity design technique used in seismic design expects accurate prediction of capacity of the ductile element ("structural fuse"). However, the actual yield stress of as-supplied steel can be considerably higher than the minimum specified yield stress. The design of columns must incorporate the probable moment resistance at the ductile elements (beam ends) in its design and ensure that the columns remain elastic. Figure 7 shows the reactions at a joint in a ductile steel moment resisting frame, where the probable yield stress factor R_y is taken as 1.1.

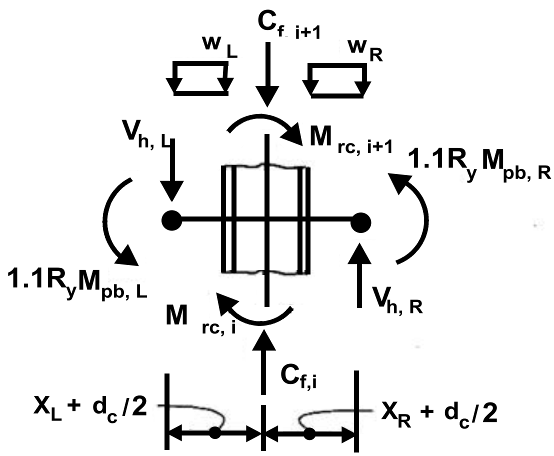


Figure 7. Ductile moment resisting frame joint – free body diagram [12].

As indicated earlier, significant ductility is required only for the predetermined energy dissipating elements, and all other elements are designed as regular elastic members, which presents an opportunity to achieve better economy through "mix-and-match" of different grade steels for different elements of a building frame. Hitherto, building projects rarely go beyond the use of a single steel grade for the entire project. This is done for reasons of construction site management, where different grades of steels could cause confusion, with the potential for wrong member placement. Improved construction management practices, such as the use of RFID tags for material tracking, at fabrication and construction sites are expected to facilitate such optimized construction.

Steels subjected to repetitive loads and steel exposed to freezing temperatures require superior toughness. The minimum mean daily temperature in northern parts of Canada is about -50 °C, and in currently populated areas may be between -10 °C and -40 °C. Primary tension members and fracture critical steel members subjected to dynamic and impact loading made of 350WT or AT steel are expected to absorb at least 27 J of energy during Charpy V-notch tests conducted at a specified temperature depending on the minimum service temperature. Nb-bearing steels exhibit improved toughness, and improved weldability particularly when produced with low carbon content (<0.10%C). Figure 8 shows the Charpy V-notch absorbed energy for Nb-bearing low-carbon steels and higher carbon steels to demonstrate this point [13].

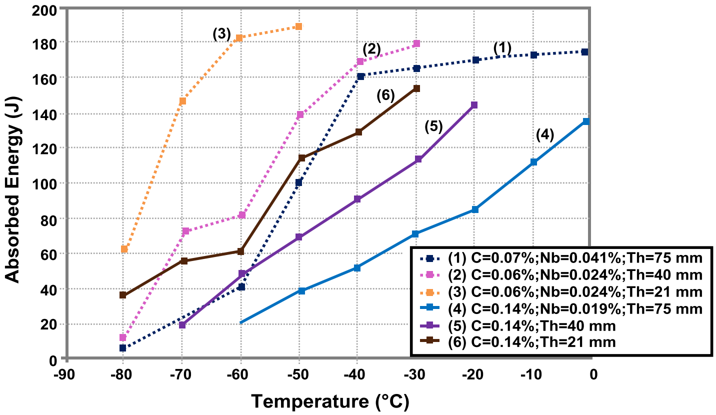


Figure 8. Charpy V-notch toughness of Nb-bearing low carbon steels [14].

Recently, North American steel design codes have introduced provisions for structural design for fire conditions. The owner decides on the design basis for fire. The deterioration of strength and stiffness of steel, having a yield strength less than 450 MPa, can be included in the design using code prescribed reduction factors. Accordingly, the yield stength of such steels at 600 °C is about 50% of the yield strength at 20 °C. As shown in Figure 9, Nb-Mo alloyed steels exhibit superior elevated temperature properties compared to other construction steels.

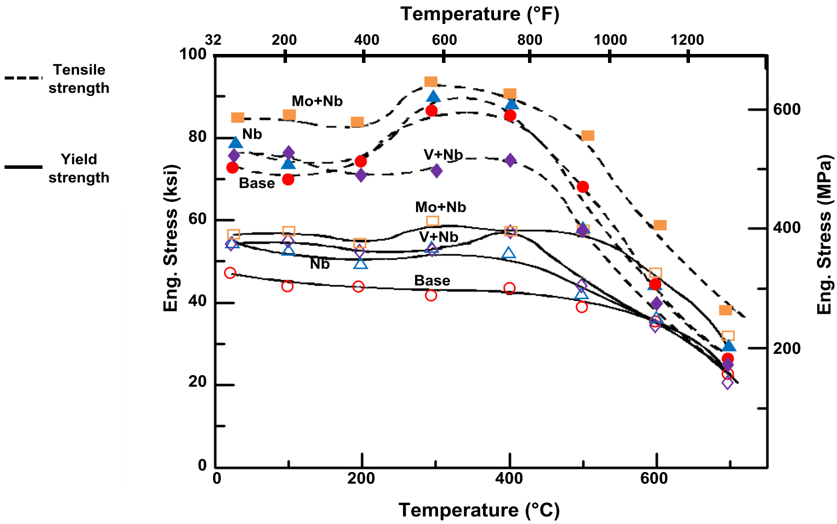


Figure 9. Elevated temperature properties of Nb-bearing structural steels [14].

Conclusions

This paper briefly discussed the structural design and constructional considerations associated with reinforced concrete building structures and steel building structures. The paper also looked at the current acceptance criteria for microalloyed steels for North American building codes and construction. Niobium microalloyed structural steel and reinforcing steel bars have been shown to exhibit superior strength, ductility, toughness and weldability than carbon-manganese steels. Improvements in construction management practices may lead to optimum use of microalloyed steels in building construction. Value added properties of Nb-bearing steels and the resulting material use and weight reductions clearly outweigh any cost premiums a builder may encounter in using such steels.

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