CODAL Provisions of Cold Formed Steel Angle Sections under Tension Members

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Abstract: The objective of the structural design is to ensure that a structure meets its intended purpose with adequate safety and operability performance during its lifetime. The increased use of cold- formed steel members was mainly due to the ongoing research. Significant improvements in the knowledge gained through interim research have led to better design guidelines, as reflected in literature and many recent international practice codes. Allowable stress design(ASD) or the limit state approach commonly referred to as the design of the load resistance factor(LRFD) are the basis for determining the load carrying capacity of cold- formed steel members.
Keywords: Cold formed steel, LRFD, ASD, IS, AS/NZS, BS

I. INTRODUCTION

Cold steel structural members can lead to more economic design than hot rolled members due to their high weight ratio, easily manufactured and built. Moreover, the significant advantages of cold-formed sections are increased yield strength, post-buckling strength and suitable for a wide range of applications. These sections are primarily thin walled with moderate to terribly high flat widths or elements.

Such members might buckle at comparatively low compressive, shear, bending or bearing stress domestically. The cold formed steel profiles recently played a major role in the civil engineering industry. The main reason is that these profiles offer many advantages compared to hot rolled sections. Cold production process makes it possible to have virtually any desired shape and product dimension.

II. ALLOWABLE STRESS DESIGN (ASD)

The permissible stress is achieved by dividing the yield stress or the ultimate tensile strength into a safety factor. The safety factor is the ratio of the material's yield point to its working stress. In ASD, "the stresses in a structure under working or service loads do not exceed the designated permissible values "is ensured. The required strengths should not exceed the allowable design strengths permitted by the applicable design standard.

Ductile structural materials like steel can withstand strains much larger than the elastic limits. Design methods based on elastic limits fail to take advantage of such material 's ability to carry stresses above yield stresses (strain hardening). The ductile material causes stress redistribution beyond the elastic limit. The redistribution of stresses often carries additional loads. From this point of view elastic analysis is excessively conservative. The limit state design method offers alternatives to elastic design objections.

III.MATERIALS AND MECHANICAL PROPERTIES

For the design of cold formed steel members, various steels are currently listed in the AISI specification. ASTM designations for these steels, yield points, tensile strengths and elongations. From a structural standpoint, the most important properties of steel are as follows:

1) Yield point or yield strength, f_y
2) Tensile strength, f_u
3) Stress-strain relationship
4) Modulus of elasticity, tangent modulus, and shear modulus
5) Ductility

A. Yield Point, Tensile Strength, and Stress-Strain Relationship

The yield or yield strength of all five different steels varies between 215 and 250 MPa. The maximum strength of the same steels is between 260 and 350 MPa. The traction strength to yield ratios vary from 1.20 to 1.50. With regard to the stress-strain relationship, the stress-strain curve can either be the high yielding type. Fig 5.1 shows the Stress-strain curves of Tensile

B. Strength Increase From Cold Work Of Forming

The mechanical characteristics (yield point, tensile strength and ductility) of cold- formed steel sections, especially at the corners, sometimes differ substantially from those of the flat steel sheet, strip, plate or bar before formation. This is because the old- forming operation increases the point of yield and tensile strength while reducing ductility. The effects of cold work on corner mechanical properties usually depend on several parameters. The tensile strength- to- yield ratios $f_u= f_y$ and the inner bend radius- to- thickness R= t are considered to be the most important factors affecting the change in mechanical properties of cold- formed steel sections. The AISI specification for calculating the tensile yield strength of the corners and the average full section tensile yield strength for design purposes provides design equations.

C. Modulus of Elasticity, Tangent Modulus, and Shear Modulus

The strength of cold- formed steel members governed by buckling is not only dependent on the yield point. The value of E is 2.00 x 10^5 to 2.15 x 10^5MPa used in the AISI specification for the design of cold- formed structural steel components. This E value is slightly higher than the 200 GPa value used in the AISC specification for the design of hot roll formed. The tangent module is the stress- strain curve slope at any given stress

D. Ductility

According to the AISI Specification, the ratio of $f_u = f_y$ for the steels used for structural framing members should not be less than 1.08, and the total elongation should not be less than 15% for a (510 mm) gage length. If these requirements can not be fulfilled, an exception can be made for purlins and girts for which the following limits must be met. When using this material, the local elongation should not be less than 20% in a length of 10 mm across the fracture and the uniform elongation outside the fracture should not be less than 5%.

IV. LOAD CARRYING CAPACITY

The following codal provisions are used to predict the capacity of the members of the single and double angle members of the American Iron and Steel Institute, the Australian / New Zealand and British standards.

A. Indian Standards, IS: 800 - 2007

1) *Gross section yielding:* Steel members can sustain loads up to the ultimate load without failure. However, the members will elongate considerably at this load, and hence make the structure unserviceable. The design strength T_{dg} is limited to the yielding of gross cross section which is given by

$T_{dg}= f_y A_g / \gamma_{m0}$

where,

f_y = yield strength of the material in MPa, A_g = gross area of cross section in mm^2.

γ_{m0}= 1.10 partial safety factor for failure at yielding.

2) *Net Section Rupture:* When the tension member with a hole is loaded statically, the point adjacent to the hole reaches the yield stress f_y. On further loading, the stress in other fibers away from the hole progressively reaches the yield stress f_y. Deformations of the member continue with increasing load until final rupture of the member occurs when the entire net cross section of the member reaches the ultimate stress f_u. The rupture strength of an angle connected through one leg is affected by shear lag. The design strength T_{dn}, as governed by rupture at net section is given by:

$T_{dn}= 0.9\, f_u\, A_{nc} / \gamma_{m1} + \beta\, A_{go}\, f_y / \gamma_{m0}$

where,

$\beta = 1.4 – 0.076\, (w/t)\, (f_y / f_u)\, (b_s / L_c) \leq f_u\, \gamma_{m0} / f_y\, \gamma_{m1} \geq 0.7$

where,

w = outstand leg width b_s = shear lag width

L_c = Length of the end connection, i.e., distance between the outermost bolts in the end joint measured along the load direction or length of the weld along the load direction.

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3) *Block shear failure:* Block shear failure is considered as a potential failure mode at the ends of an axially loaded tension member. In this failure mode, the failure of the member occurs along a path involving tension on one plane and shear on a perpendicular plane along the fasteners. The block shear strength T_{db}, of connection shall be taken as the smaller of:

$T_{db} = (A_{vg} f_y / (\sqrt{3}\, \gamma_{m0}) + f_u A_{tn} / \gamma_{m1})$ or

$T_{db} = (A_{vn} f_u / (\sqrt{3}\, \gamma_{m1}) + f_y A_{tg} / \gamma_{m0})$

where,

A_{vg}, A_{vn} = minimum gross and net area in shear along a line of transmitted force, respectively.

A_{tg}, A_{tn} = minimum gross and net area in tension from the bolt hole to the toe of the angle, end bolt line, perpendicular to the line of force.

f_u, f_y = ultimate and yield stress of the material respectively.

B. Australian and New Zealand standards, AS/NZS: 4600 - 2005

The nominal section capacity of a member in tension shall be taken as the lesser of

$N_t = A_g f_y$ (1), $N_t = A_n 0.85 K_t A_n f_u$ (2)

where Ag= gross cross sectional area of the member, mm^2

f_y= yield stress of the material, N/mm^2

K_t= correction factor for distribution of forces = 0.85

A_n= net area of the cross-section, obtained by deducting from the gross area of the cross section, the sectional area of all penetrations and holes, including fastener holes mm^2

f_u= tensile strength used in the design, N/mm^2

C. American Iron And Steel Institute, Aisi: 2001 (Manual)

The tensile capacity P_n, of a member should be determined from

$P_n = A_n A_e f_u$

Where f_u= tensile strength of the connected part of a member, N/mm^2

$A_e = U A_n$ and $U = 1.0 - 0.36 X / L < 0.9$ and $U > 0.5$

A_n= effective net sectional area of the member, mm^2

X = distance from shear plane to centroid of the cross section, mm

L= length of the end connection i.e. distance between the outermost bolts in the joint along the length direction mm

British Standard, BS: 5950 - 1998 (Part 5)

The tensile capacity Pt, of a member

$P_t = A_e * p_y$

Single angles

$A_e = a_1 (3a_1+4a_2) / (3a_1+a_2)$

Double angles

$A_e = a_1 (5a_1+6a_2) / (5a_1+a_2)$

For double angles connected to the same side of gusset plate the effective area can be determined as that of single angles. A_e = effective area of the section.

a_1 = the net sectional area of the connected leg.

a_2 = the gross sectional area of the unconnected leg.

V. BOLTED CONNECTION

Different types of bolted connections exist. They can be categorized based on the type of loading and splicing. It subjects the bolts to strengths that tend to shear the shank. The connection hanger puts the bolts in tension. In most bolted connection, the bolts are subjected to shear. Bolts can fail in shear or in tension.

It can calculate the shear strength or the tensile strength of a bolt. Simple connection if the force line acting on the connection passes through the center of gravity of the connection, then each bolt can be assumed to withstand the same load share. The strength of the simple connection is equal to the sum of strengths in the connection of the individual bolts. Fig 5.2 shows the bolted connection under tensile

VI. DESIGN CRITERIA FOR BOLTED CONNECTIONS

Design strength should be more than design load. The center of gravity of bolts should coincide with the centre of gravity of the connected members. The length of connection should be kept as small as possible. In the determination of the allowable load the following points should be considered. such as minimum spacing and edge distance in line of stress, tensile capacity of steel sheets, Bearing capacity between bolts and steel sheets and Shear capacity of bolts.

1) *Pitch of the Bolts (p):* It is the centre-to-centre spacing of the bolts in a row, measured along the direction of load.
2) *Gauge Distance (g):* It is the distance between the two consecutive bolts of adjacent rows and is measured at right angles to the direction of load.
3) *Edge Distance (e):* It is the distance of bolt hole from the adjacent edge of the plate.

The clear distance between bolts which are arranged in rows parallel to the directions of force, also the distance from center of any bolt to that end or other boundary of the connecting member towards which the pressure of the bolt is directed shall not be less than 1.5d nor less than

$P/(0.6f_y)$

where

d=diameter of bolt

P= force transmitted to bolt

f_y = yield point

VII. CONCLUSIONS

Codal provisions, review and compare the provisions of various codes of practice in this chapter. Comparison of the provisions of the British, American, Australian and New Zealand standards is carried out along with experimental results. The ultimate limit state is checked for strength consideration and the serviceability limit state is checked for actual service condition. The load acting and the resistance of the structure to load are variables that are considered in the Limit State Design.

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