AUSTRALIAN STEEL INSTITUTE

AS 4100 DS06

Steel Structures – Single angle compression members

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SINGLE ANGLE COMPRESSION MEMBERS

Preface

Standards Australia has recently amended Clause 8.4.6 (Ref. 1) of AS 4100—1990 (Ref. 2). This Limit States Data Sheet provides a commentary on this amended Clause.

Introduction

The behaviour of a single angle compression member is complicated by the details of its connection to the rest of the structure, and by the behaviour of the members connected to it. Connections are usually made to one leg of the angle so that the angle is loaded eccentrically in a plane that lies between its principal planes. Different types of end connection provide different types of restraint both in and out of the plane of the connected leg. The restraining actions between the angle and its adjacent members also depend on the relative stiffnesses of the adjacent members and the angle, and theoretically vary within the range from full restraint to the angle to full restraint to the adjacent members (in which case the angle is negatively restrained).

Many different methods have been proposed for designing angles, but few of these are supported by data from well-conducted tests.

Because of this, the principal method given in AS 4100 for designing a single angle is to use the appropriate beam-column methods of Clause 8.3 (for section capacity), Clause 8.4.2 (for in-plane capacity), Clause 8.4.4 (for out-of-plane capacity), and Clause 8.4.5 (for biaxial bending capacity). These clauses allow the designer to make appropriate assumptions about the eccentricity and the effective length.

For example, a single angle that is welded to substantial main members that are lightly loaded may act as if almost fixed at both ends, in which case the effective end eccentricities will be small and the effective length will be close to half the angle length.

At the opposite extreme, a main member such as a tee-section chord may rely on the single angle to restrain it against flexural or twist rotations, in which case the angle will have an effective length greater than the actual length and a significant end eccentricity.

Another single angle may be connected in such a way that end rotations are almost unrestrained, in which case the end eccentricities will be significant, and the effective length will be close to the angle length.

The designer will therefore need to assess the end connections and the behaviour of the adjacent members in order to make appropriate assumptions for the eccentricities and the effective length.

Design using Clauses 8.3 and 8.4.5

Clause 8.4.6 requires the designer to use either of two methods, the first of these being given in Clauses 8.3 and 8.4.5. In this method, no guidance is given to the designer on the appropriate eccentricities or effective length to be used, and so it is the designer's responsibility to make assumptions that are appropriate to the structure under consideration.

Design using Clause 8.4.6

The remainder of Clause 8.4.6 provides and alternative method of design which gives guidance on the end eccentricities and effective length. This method is derived from a conversion to limit states design of the working stress design method developed in Reference 3 for eccentrically loaded double bolted or welded single angles in trusses.

The working stress method was based on observations of both test data and theoretical analyses of welded angles in trusses which indicated that the angles deflect mainly in planes perpendicular to the trusses, and with very little twisting. For this to be so, the angles have to have connections that can provide significant restraints against end rotations in the planes of the trusses. Because of this, the application of the method is restricted to single angles with at least double bolted or welded end connections, and cannot be used for single bolted angles.

The assumption that the eccentrically connected angle deflects only in the plane perpendicular to the truss allows simplifications to be made with respect to its buckling behaviour under axial load, and to its bending behaviour under end moments, Its compression strength under axial load N^* alone is based on the use of the nominal compression capacity N_{ch} for buckling perpendicular to the plane of the truss.

The bending capacity of the angle in the absence of axial load is also approximated by considering deflections only in the plane perpendicular to the truss. In this case, the design moment M_h^* out of the plane of the truss must be accompanied by an in-plane moment M_n^* supplied by the in-plane restraint which prevents rotation in the plane of the truss. Thus, the nominal moment capacity (M_{bx}) of the angle is assumed to be reached when the out-of-plane moment M_h^* reaches the out-of-plane component $M_{bx}\cos\alpha$ of the nominal moment capacity M_{bx}, in which α is the angle between the plane of M_h^* and the plane of M_{bx}.

The design capacity of the angle under combined compression N^* and bending M_h^* is formulated as a linear interaction equation between the compression ratio $N^* / \phi N_{ch}$ and the moment ratio $M_h^* / \phi M_{bx}\cos\alpha$.

For an equal leg angle whose length-thickness ratio l/t is less than the stated limit, lateral buckling effects may be ignored, and the member moment capacity M_{bx} may be taken as the section capacity M_{sx}. For other equal leg angles, a simple approximation is given for the elastic buckling moment M_o which can then be used in Clause 5.6.1.1 to obtain the moment member capacity M_{bx}.

Simple approximations are given for the end eccentricity e to be used in determining the design bending moment M_h^*. These are based on the conditions at the connection of the angle to the compression chord of the truss, and the assumption that the compression chord does not restrain the angle against rotations out of the plane of the truss. When the chords do provide restraints, then smaller values of the eccentricities may be used, provided these are justified by a rational elastic analysis.

The value of the factor α_m to be used in determining M_{bx} may be assumed conservatively to be given by α_m = 1.0. Alternatively, it may be argued that the tension chord will provide some restraint at the other end of the angle, so that a higher value of α_m may be used. If this restraint halves the end moment so that β_m = -0.5, then α_m = 1.3.

REFERENCES

1 Standards Australia, Amendment No. 3 to *AS 4100 steel structures*, Sydney, 1995.

2 Standards Australia, *AS 4100 Steel structures*, Sydney, 1990.

3 Woolcock, ST and Kitipornchai, S, 'Design of Single Angle Web Struts in Trusses', *Journal of Structural Engineering*, ASCE, Vol. 112, No. 6, June 1986, pp 1327-45.