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The Design of a Single Angle Strut

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THE DESIGN OF SINGLE ANGLE STRUTS

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Synopsis

The state-of-the-art of eccentrically loaded single angle compression member design is reviewed. Current code rules and standard practices vary greatly in their treatment and there does not seem to exist a generally accepted method of design. A three dimensional computer model is developed to examine the elastic behaviour of fully welded trusses and in particular the degree of in and out of plane restraint. The particular cases when single angle web members are connected on the same sides and alternatively on the opposite sides of a Tee-chord are examined in detail. A recommended design method which uses the combined stresses interaction equation of AS 1250-1975 for buckling and bending in the plan perpendicular to the truss is proposed.

CONTENTS

1. INTRODUCTION

Angles are perhaps the most basic and widely used of all rolled structural steel sections. There is a wide range of sizes available and end connections are relatively simple.

Single angles are commonly used in light roof trusses mainly as web members and in transmission towers. In roof trusses, the single angle web members are often connected by one leg on one side of the chords and sometimes alternately on opposite sides of Tee section chords as shown in Figure 1. Despite their apparent simplicity, the analysis of eccentrically connected single angle struts and tension ties is quite complex not only because of the eccentricity but also because the principal axes are usually inclined to the frame axes.

These problems have been overcome in the design of single angle tension ties because tests have shown that the eccentric ultimate tensile capacity is not much less than the concentric ultimate capacity[1,2]. Codes[3-5] allow for eccentricity in tension ties by specifying reduced cross sectional areas so that the axial tension may be assumed to act concentrically. However, these difficulties in the design of eccentrically loaded single angle compression members have not been resolved. Existing codes[3-5] and references[6-9] vary in their treatment of single angle struts and there does not seem to be a generally accepted method of design which accounts for eccentricity and end restraint both in and out of plane.

The current British steel code BS 499:1969[4] advocates ignoring end eccentricity altogether, an approach echoed in the British "Steel Designers' Manual"[6] which is widely used in Australia. This approach was implied in Australian Standards AS CA1-1968[10] and AS 1250-1972[11], but was discontinued in AS 1250-1975[3]. The second and third editions of the Australian Institute of Steel Construction's Safe Load Tables[7,8] also reflected the current British practice by presenting tables for eccentrically connected angles calculated on the basis of concentric loading. By contrast, the fourth edition[9] published

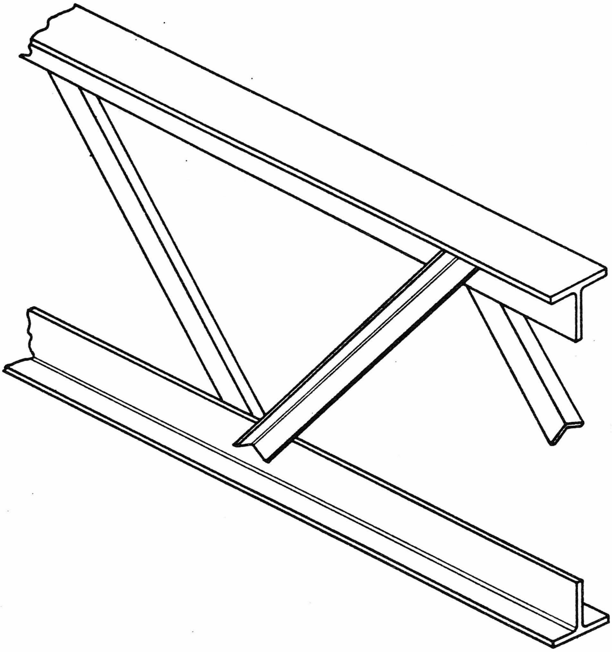


FIGURE 1a : Web members on opposite sides

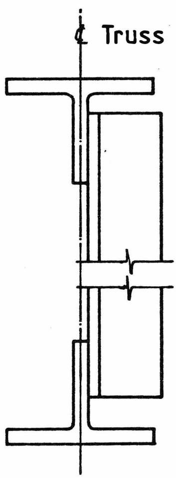


FIGURE 1b : Web members all on one side

in 1980 does not present tables for single angle struts loaded through one leg and clearly states that eccentricity with respect to the principal axes should be accounted for as a biaxial combined stresses problem.

This paper examines the state-of-the-art of eccentrically connected single angle strut design in Britain, the United States and Australia. Computer studies, using a three dimensional frame analysis program, which examines the effects of in-plane and out of plane restraint in a common type of single angle truss are reported. The particular cases where single angle web members are alternately connected on opposite sides of a Tee chord are included to demonstrate the different degrees of out of plane restraint. A simplified design method which considers buckling and eccentricity in the plane perpendicular to the plane of the truss is proposed and design examples are presented. Single angles with single bolted ends are not considered in this paper.

2. DESIGN METHODS

2.1 General

This section of the paper examines the current rules and design methods in Britain, the United States and Australia, and then investigates two recently proposed design methods. Superseded code rules and design methods in Britain and Australia are also examined not only for historical interest but also because these rules have some relevance to more recent methods.

2.2 British Practice

British design rules for single angle struts first originated in BS:449 in 1948[13]. Three tables of permissible axial compressive stress were presented - Table 7 for concentrically loaded struts, Table 8 for single angle struts with double-bolted, double-riveted or welded ends, and Table 9 for single angle struts with single-bolted or single - riveted ends. The permissible stresses were tabulated against slenderness ratio ℓ/r, where ℓ for single angle struts was specified as 0.8 times the length of the strut centre to centre of fastenings and r as the minimum radius of gyration r_v. The stresses in Table 8 were reduced compared to those in Table 7 to account for eccentricity as described in Appendix D of BS 449:1948 which stated that in preparing Table 8, eccentricity perpendicular to the plane of the gusset was included by assuming that the load is applied at the centre of thickness of the attached leg, rather than at the mid-plane of the gusset to make "an allowance for the stiffness of the connections". Tables 7 and 8 of BS 449:1948 are reproduced in Figure 2. It should be noted that the allowable stresses tabulated are based on steel with a yield stress of 15 tons/ square inch which is equivalent to about 230 MPa.

Appendix D of BS 449:1948 also referred to test results issued by the United States Bureau of Standards in 1924 for a series of tests on single angle struts with various end connections and concluded that the permissible stresses in Table 8 had a reasonable margin of safety. Nevertheless, it was felt in Britain that further experimental work should be conducted to corroborate the new rules in BS 449:1948 and subsequently Mackey and Williamson at Leeds University were commissioned to carry out a series of tests.

Mackey and Williamson[15] tested two trusses of 3 and 10 metre spans as shown in Figures 3 and 4. The trusses were constructed with double angle chords and with single angle web members which were at first single bolted and later double-bolted to one side of the gusset plates. The web members consisted of either equal angles or unequal angles with short legs outstanding. The results for the equal angle struts BH and CL of trusses G1 and G2 respectively are shown in Figure 5. Compared with the safe loads predicted by BS 449:1948, Mackey and Williamson obtained factors of safety of 2.4 and 3.0 for these failure loads and 2.7 for the unequal angle strut BM and concluded that the permissible stresses given in BS 449:1948 were conservative.

TABLE 7: Permissible working stresses in tons/sq. in. of gross section for axial loads

ℓ/r	F_a tons/ sq.in.	ℓ/r	F_a tons/ sq.in.	ℓ/r	F_a tons/ sq.in.	ℓ/r	F_a tons/ sq.in.
0	9.00	70	5.60	140	2.57	210	1.27
10	8.51	80	5.12	150	2.30	220	1.17
20	8.03	90	4.62	160	2.06	230	1.08
30	7.54	100	4.13	170	1.86	240	0.99
40	7.06	110	3.67	180	1.68	250	0.92
50	6.57	120	3.26	190	1.52	300	0.65
60	6.09	130	2.89	200	1.39	350	0.49

TABLE 8: Permissible working stresses in tons/sq.in. of gross section for discontinuous angle struts. (Double-bolted, double-riveted, or welded at ends.)

L/r	$F_{e}2$ tons/ sq.in.	L/r	$F_{e}2$ tons/ sq.in.	L/r	$F_{e}2$ tons/ sq.in.	L/r	$F_{e}2$ tons/ sq.in.	L/r	$F_{e}2$ tons/ sq.in.	L/r	$F_{e}2$ tons/ sq.in.
0	6.00	50	4.55	100	3.12	150	1.95	200	1.25	250	0.86
10	5.71	60	4.26	110	2.85	160	1.77	210	1.16	300	0.62
20	5.42	70	3.97	120	2.60	170	1.61	220	1.08	350	0.47
30	5.13	80	3.68	130	2.37	180	1.47	230	1.00		
40	4.84	90	3.40	140	2.15	190	1.35	240	0.93		

FIGURE 2 : Tables 7 and 8 of BS 449:1948

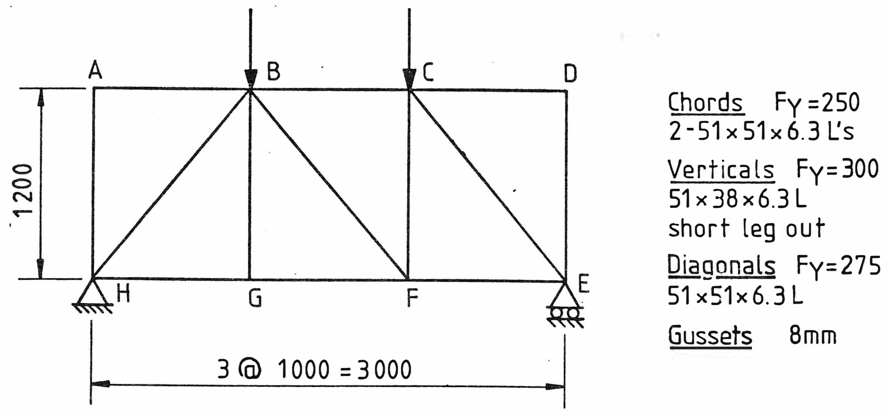


FIGURE 3 : Mackey and Williamson's truss G1

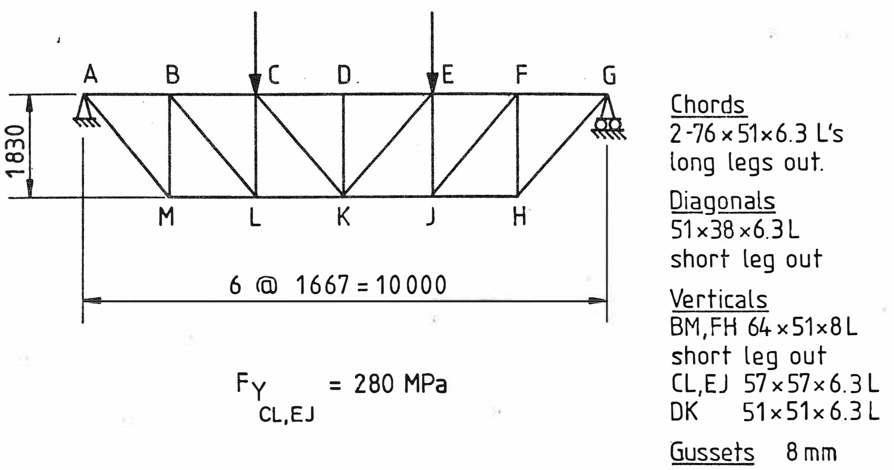


FIGURE 4 : Mackey and Williamson's truss G2

It was probably because of Mackey and Williamson's results, despite the limited range of slendernesses considered, that the British steel code was revised in 1959 with the allowable compressive stress tables for single angles being eliminated. Single angle struts with double bolted or welded ends were then to be designed as concentrically loaded struts with the slenderness ratio ℓ/r based on an effective length of 0.85 times the distance between intersection points and the minimum radius of gyration r_v. This approach was maintained in the current version of the British code BS 449:1969[4].

The safe loads of single angle struts based on the 1948 and 1969 versions of BS 449 are presented in Figure 5 as functions of modified slenderness. It can be seen that the current British code permits much higher loads for stocky struts than the 1948 version. The use of modified slenderness, which is obtained by factoring the more usual slenderness ratio ℓ/r by $\sqrt{F_Y/\pi^2 E}$, allows theoretical and experimental values of P/P_Y to be plotted almost non-dimensionally against modified slenderness. There is a slight variation in P/P_Y values with yield stress for a given modified slenderness, but the difference is small enough to be disregarded in plotting the safe load curves in Figure 5.

The modified slenderness scale $(0.75L/r_v)\sqrt{F_Y/\pi^2 E}$ for the loads from Table 8 of BS 449:1948 is based on the assumption that the effective length of 0.8 times the distance centre to centre of fastenings is approximately 0.75 times L the distance between intersection points. If the modified slenderness scale $(L/r_x)\sqrt{F_Y/\pi^2 E}$ is used, then the BS 449:1948 and 1969 safe load curves apply to equal angles only. This is because the relationship between the different modified slenderness scales shown in Figure 5 is based on a ratio r_x/r_v of 1.55 which applies to equal angles.

2.3 American Practice

Apart from the tests carried out by the United States Bureau of Standards in 1924 and the tests conducted by Foehl[16] in 1948 on seven single angle struts, there appears to have

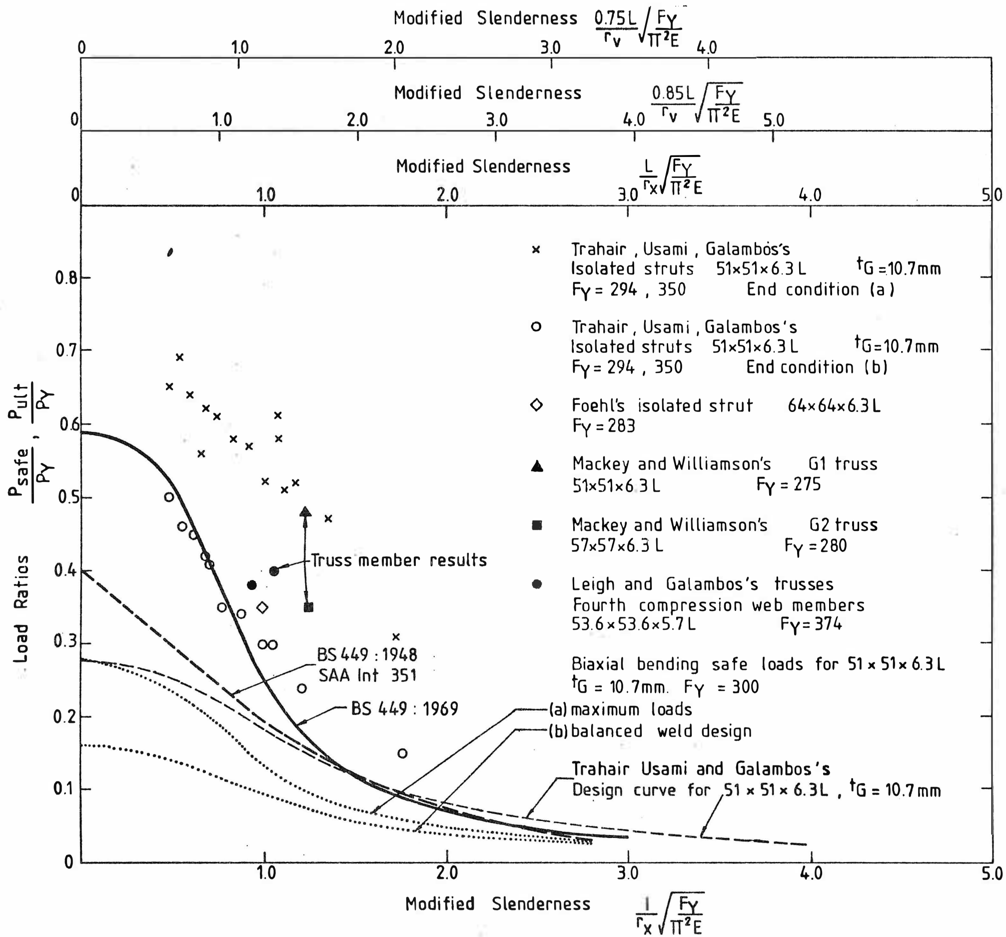


FIGURE 5 : Comparison of various design methods and test failure loads

been little experimental work done in the United States on single angle struts until recently[17,18]. Nevertheless, the American AISC Specification[5] has consistently made no concessions for the design of single angle struts thereby implying that eccentricity must be accounted for. The American Manuals of Steel Construction[20] have been more explicit over the years by recommending that bending about both principal axes be considered.

The difficulty with the biaxial bending approach is in determining where the load acts. It is common to assume that the load acts at the mid-plane of the gusset or at the mid-plane of the stem of a Tee chord, but where along these planes should the load be assumed to act. The 1955 and 1961 editions of the AISC Steel Construction Manuals[20] recommend that the applied forces should be placed at the centres of the rivets, bolts or welds. This is reasonable for angles with single fasteners at each end but does not account for any in-plane resistance to rotation which may exist at ends that are double bolted or welded.

McGuire[1] demonstrates theoretically how sensitive the stress distribution is to movement of the point of load application along the mid-plane of the gusset. For deflection of an equal angle perpendicular to the plane of the gusset, the position of the load must be such as to produce bending moments about the principal uu and vv axes in proportion to their second moments of area I_u and I_v. This will also result in constant stress along the connected leg, for which it can be shown that the distance x_P along the mid-plane of the gusset from the heel to the point of application of load P is

$$x_P = c_x + e\,\frac{I_{xy}}{I_y}$$

$$= c_x + \left(c_x + \frac{t_G}{2}\right)\frac{I_u - I_v}{2I_y} \qquad (1)$$

where the variables in this equation are defined in Figure 6. For most equal angle and gusset plate combinations, x_P is approximately equal to B/2 for deflection perpendicular to the gusset. For example, for a 64 x 64 x 8 angle and a 10 mm gusset, x_p is 32.4 mm.

By comparison, the gauge line for bolts is 35 mm from the heel and the centroid of the so-called 'balanced weld group' is on the y- axis, 18.6 mm from the heel. The marked variation in stress at the points a, b and c in Figure 6 as the point of application of load moves along the mid-plane of the gusset can be seen in Table 1. Each stress is divided by the constant stress along the connected leg when x_p equals 32.4 mm to obtain the ratios given in the table.

TABLE 1 : Relative stresses in single angle struts

Location	x_p mm	a	b	c
Centroid of balanced weld group (on y axis)	18.6	0.12	1.78	- 1.03
For deflection perpendicular to plane of gusset	32.4	1.0	1.0	- 0.83
Gauge line for bolts	35	1.14	0.87	- 0.79

McGuire[1] also examines the experimental results obtained by Gibson and Wake for a series of tests on isolated single angle tension members. He notes that although the weld group was balanced by the conventional design technique, the stiffness of welds and gussets and the rotational restraint of the jaws of the testing machine appear to have caused a shift in the point of application of load along the mid-plane of the gusset away from the y-axis. He observes that for single angles in tension connected along one leg, "the line of action of the resultant force is only partially a function of the placement of rivets, bolts or welds and appears more dependent upon the stiffness of the gussets and other members of the frame". This observation applies equally well to single angle compression members. Nevertheless, for the purpose of

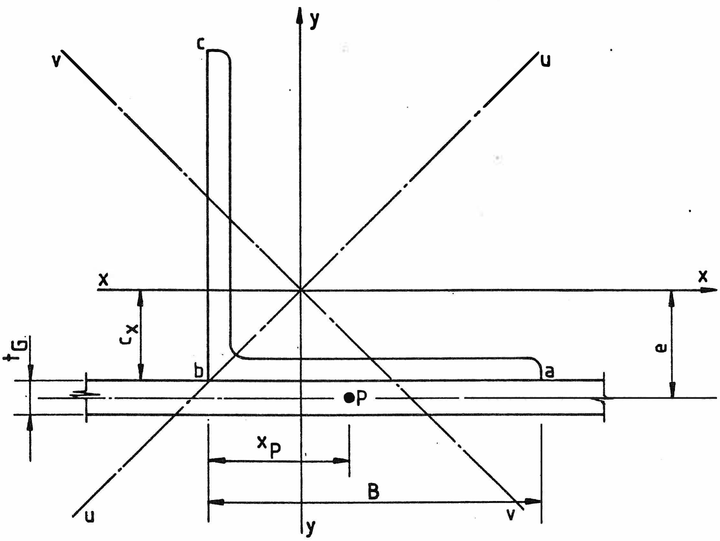


FIGURE 6 : Definition of terms

comparison, the safe loads for a 51 x 51 x 6.3 angle have been calculated on the assumption that the line of action of the axial force passes through the y axis in the mid-plane of the gusset as in the case of balanced weld design. The slenderness ratio was taken as 0.85 L/r_v and the stresses were combined as in Part D of Design Example 4. The safe loads so obtained are presented in Figure 5.

Higher safe loads will be obtained as the assumed point of application of force moves along the mid-plane of the gusset from the y axis, until the loads reach a maximum when the point of application is such as to cause the strut to deflect only in the plane perpendicular to the plane of the truss. The curve of maximum loads is also shown in Figure 5. As the load moves beyond this point, the calculated safe loads will again diminish.

2.4 Australian Practice

Australian steel codes were closely related to British standards until 1968 when AS CA1[10] was published with "substantial differences". Since then, American specifications have also exerted an influence, so that together with original Australian contributions, the present Australian Standard AS 1250-1975[3] has become a unique document. In the transition period, the rules for single angle struts in Australian codes have, in broad terms, gone the full circle from allowing for eccentricity in SAA Int. 351, to ignoring it in AS CA1-1968 and AS 1250-1972, to not ignoring it in AS 1250-1975.

The rules in the interim Australian Standard SAA Int. 351[14], which was in force from 1952 until 1968, were virtually identical to those in BS 449:1948 with eccentricity accounted for by reduced permissible axial stresses.

AS CA1-1968 and AS 1250-1972 followed the new British practice which began in BS 449:1959 of ignoring eccentricity in single angle struts with double bolted or welded ends, although the relevant Australian rules were not as explicit as the British rules. The only rules specifically concerning single angle struts in the 1968 and 1972 Australian codes were Rules 6.1.3 and 6.1.4 respectively, but these related to single angle struts with single bolted or riveted connections at each end. These rules allowed eccentricity of force to be neglected provided that the calculated axial stress did not exceed 0.80 times the maximum permissible stress for a concentrically loaded strut. If there was a 20% reduction for single-bolted ends, it was reasonable to infer that there was no reduction in permissible axial stress for double-bolted or welded ends, especially when zero reduction was explicitly allowed in the corresponding British code at the time.

The concession for single angle struts was discontinued in the 1975 edition of AS 1250, the intention being that single angle struts with eccentric end connections should be treated like any other eccentrically loaded strut. Some confusion would have been avoided had there been a footnote in the code to this effect. In any case, with this new approach as with the American approach, designers are faced with the problem of determining where the load acts.

The fourth and latest edition of the AISC's Safe Load Tables[9] has discontinued the presentation of safe load tables for eccentrically connected single angle struts, thereby reflecting the intention of AS 1250-1975 that eccentricity be accounted for. The publication of the new Safe Load Tables should certainly help dispel any doubts that eccentricity should be considered.

It is interesting to note here that the Australian Aluminium Code AS 1664-1975[21] specifies an allowable axial compressive stress for single angles equal to 0.4 times that for an equivalent centrally loaded strut in order to account for the eccentricity of connection.

2.5 Trahair, Usami and Galambos's Study

In 1969, Trahair, Usami and Galambos[17] described a theoretical and experimental study of single angle struts. Altogether, forty-six tests of isolated single struts were reported involving three different end conditions: (a) fixed except for the flexibility of the web of the Tee end brackets, (b) free to rotate out of the plane of the truss, and (c) free to rotate in the plane of the truss. They observed that for end condition (b), which gave the lowest failure loads, the predominant mode of deformation was perpendicular to the plane of the connected leg with little accompanying deflection in the plane of the connected leg and very little twisting. This led them to investigate the possibility of using the AISC[5] combined stresses equation for buckling and eccentricity in only one plane i.e. the plane perpendicular to the plane of the connected leg. As the corresponding equation in AS 1250-1975 is identical (except that AS 1250 uses a 0.60 factor in place of the American 12/23 factor), it is convenient to express the design equation using the AS 1250 notation. The design equation is as follows:

$$\frac{f_{ac}}{F_{ac}} + \frac{C_{mx} f_{bcx}}{\left(1 - \frac{f_{ac}}{0.60 F_{ocx}}\right) F_{bcx}} \leqslant 1.0 \qquad (2)$$

where

f_{ac} is the axial stress

F_{ac} is the allowable axial stress based on the full member length between intersections and on the radius of gyration r_x about the axis parallel to the plane of the truss.

C_{mx} is a coefficient which depends on the distribution of bending moment along the compression member and is defined in AS 1250-1975. When the eccentricity is the same magnitude at each end of the member and is such that the strut is bent in single curvature, the value of C_{mx} is unity.

f_{bcx} is the compressive bending stress at the end of the angle equal to $Pe c_x / I_x$ (see Figure 6).

F_{bcx} is the allowable bending stress in the absence of axial load, taken as $0.60F_Y$ by Trahair et al.

F_{ocx} is the Euler buckling stress equal to $\pi^2E/(L/r_x)^2$.

Figure 5 shows how the safe loads predicted by Equation 2 compare with experimental isolated strut failure loads and with the experimental failure loads of members tested as part of a truss framework. The results for isolated members fall into two groups - the upper set for the 'fixed' end condition and the lower set for end condition (b) which allows rotation only out of the plane of the truss. The design equation (Equation 2) which is based on end condition (b), predicts the failure loads of isolated struts with that end condition with a reasonable factor of safety except for higher slenderness ratios where the factor of safety is as low as 1.5. However, the results shown in Figure 5 for actual truss members indicate much higher factors of safety which is undoubtedly due to the out of plane restraint existing in the trusses tested.

The tests conducted by Trahair, Usami and Galambos[17] also included unequal angles with either the short or long legs outstanding. They found that the application of the combined stresses equation (Equation 2) to unequal angle struts with short legs outstanding did not give an adequate factor of safety for the isolated struts tested, and they recommended against its use in such cases.

2.6 Leigh and Galambos's Method

Following Trahair, Usami and Galambos's study, the programme of research into single angle struts at Washington University continued, and in 1972, Leigh and Galambos[18,19] reported results of ten tests on full sized trusses 18.29 m in length. They confirmed that web compression members deflect and fail in a plane perpendicular to the truss, and they proposed a design method for web compression members which accounts for out of plane end moments due to the eccentricity of connection and the flexural interaction of the strut with adjacent web members.

Leigh and Galambos presented formulae expressing the web member end moments at a joint in terms of an out of plane couple about the longitudinal axis of the chord. For trusses with horizontal top and bottom chords, the out of plane couple at a joint is calculated by multiplying the vertical component of the web member forces at the joint by the relative eccentricity e_r between the two members as shown in Figure 7. This couple is then distributed to the web members in proportion to their out of plane rotational stiffnesses about an axis parallel to the chord.

They also presented a method for determining the effective length of web struts when not all the joints of a truss are laterally restrained. For their purpose, they assumed that the top chord of a truss is continuously laterally restrained by flooring or roofing and that the bottom chord is braced only at a number of discrete points. Then if the bottom end of a web strut does not coincide with a bottom chord bracing point, the bottom end is laterally restrained only by the transverse flexural stiffness of the bottom chord between bracing points. Using the theoretical model shown in Figure 8, they obtained effective lengths up to 1.67 times the lengths L of the struts they investigated.

The failure loads Leigh and Galambos obtained for the fourth compression web of their trusses J7 and J9 are shown in Figure 5 with the modified slendernesses calculated using an effective length equal to L. Truss J9 is depicted in Figure 9 and truss J7 is similar except that the member sizes are slightly different. It can be seen that the factors of safety using the current British method are approximately 1.67 and 1.27, the latter value being an inadequate factor of safety.

Comparison of the experimental failure loads with the loads predicted using Leigh and Galambos's method will not be made here. This is because their method forms the basis of the design method recommended later in this paper and the fourth compression web of truss J9 will be examined in detail in Design Example 3.

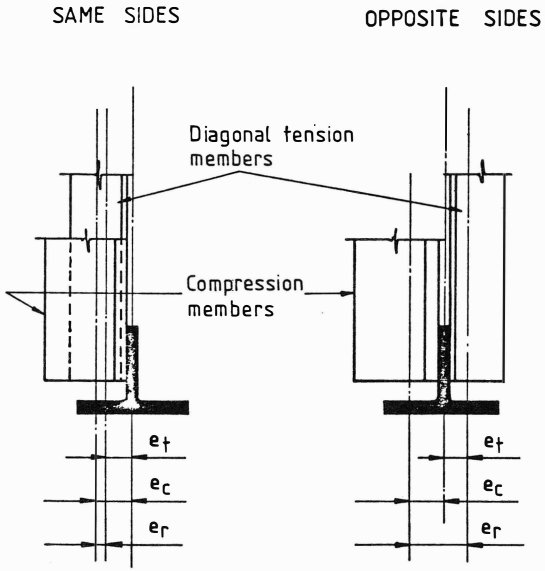


FIGURE 7 : Joint eccentricities

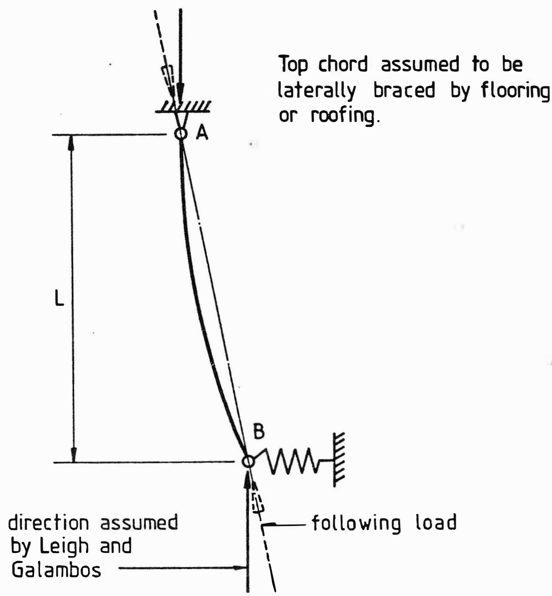
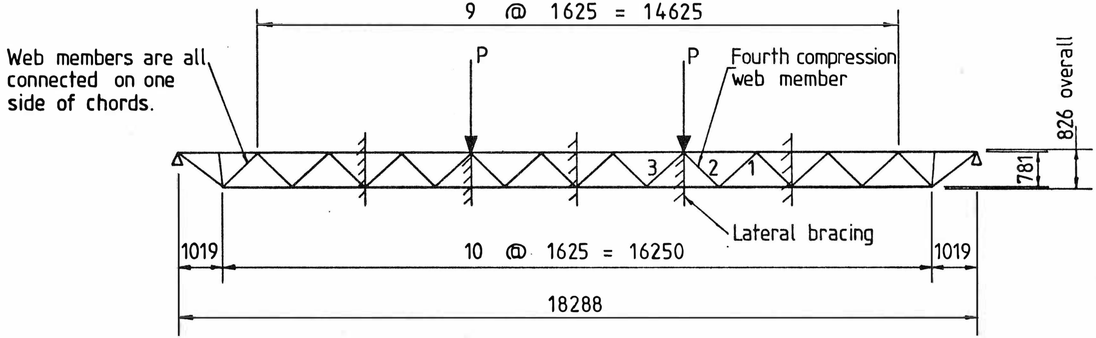


FIGURE 8 : Leigh and Galambos's model for determining effective lengths



Top chord[19]	2 - 82 × 82 × 8.6 L's stitch welded back to back with no gap to form a tee.
Bottom chord[19]	2 - 80 × 80 × 6.9 L's stitch welded back to back with no gap to form a tee.
Web member 1	35 × 35 × 4.7 L
Web member 2	54 × 54 × 5.7 L $\quad F_Y = 374$ MPa
Web member 3	27 × 27 × 3.3 L

FIGURE 9 : Leigh and Galambos's truss J9

3. COMPUTER STUDIES

3.1 Background

The most direct way of investigating the behaviour of single angle truss members is by testing full scale trusses. However, an exhaustive experimental programme is both time consuming and expensive because of the number of variables involved. Some of the variables are listed below.

a) Truss type and configuration.

b) Type of member. The web members can vary from equal angles to unequal angles with long or short legs outstanding to a mixture of these types in a given truss. The chords can vary from double angles to tees to single angles.

c) Slope of diagonals.

d) Type of joints. The joints can be with or without gusset plates and can be single-bolted, double-bolted or welded. The web members can be connected on one side of the chords, or they can be alternately connected on opposite sides.

e) Type of loading. The loading may or may not provide lateral or rotational restraint to the truss.

f) Number and type of lateral restraints.

g) Type of supports.

In an attempt to reduce the number of variables which would need to be covered experimentally, three dimensional computer models were developed by the authors using a first order three dimensional frame analysis program. Such computer models could account for the elastic behaviour of fully welded trusses encompassing most of the variables listed above. In this paper, only the computer analysis of the following variable is reported.

3.2 Behaviour of Web Members Connected on the Same Sides and on Alternate Sides

In order to compare the effects of connecting web members on the same sides and on opposite sides of chords, two computer models of the simple truss shown in Figure 10 were prepared. All of the relevant member data is shown in Figure 10 except for the properties of the short members which model the joints. The properties of the short members in the plane of the truss were calculated in an attempt to model the actual properties of the Tee web stiffened by the members attached to it. The short members perpendicular to the plane of the truss were given properties to ensure that they behaved as both flexurally and torsionally rigid members. Results for the member BJ when all the web members are attached on the same side (Case S) and when the verticals and diagonals are attached on opposite sides (Case O) are presented in Figure 11.

It can be seen that the stresses are significantly higher in the Case O web member than in the Case S member. The stresses at the ends of the member in each case are dominated by the effect of in-plane rigid frame moments which result in higher peak stresses at the ends than at the centre of the member. However, the stresses at the ends may not govern the design because they are not as critical as those at the centre of a strut as the combined stresses rules in AS 1250-1975 demonstrate. At the middle of the member, the peak stress of 149 MPa for Case O exceeds the corresponding stress of 72 MPa for Case S by 107%. The difference in behaviour is demonstrated by the mid-point deflections. In each case, the in-plane deflections relative to the ends of the member are small, being 0.5 mm for Case O and close to zero for Case S, but the out of plane deflections are 3.7 mm for Case O and 0.7 mm for Case S.

The in-plane and out of plane bending moments at the ends of the member are also shown in Figure 11. The Case O end moments about the x-axis are 3.44 kNm and 3.30 kNm, these values being more than four times the corresponding Case S values of 0.82 kNm and 0.50 kNm. Dividing the average

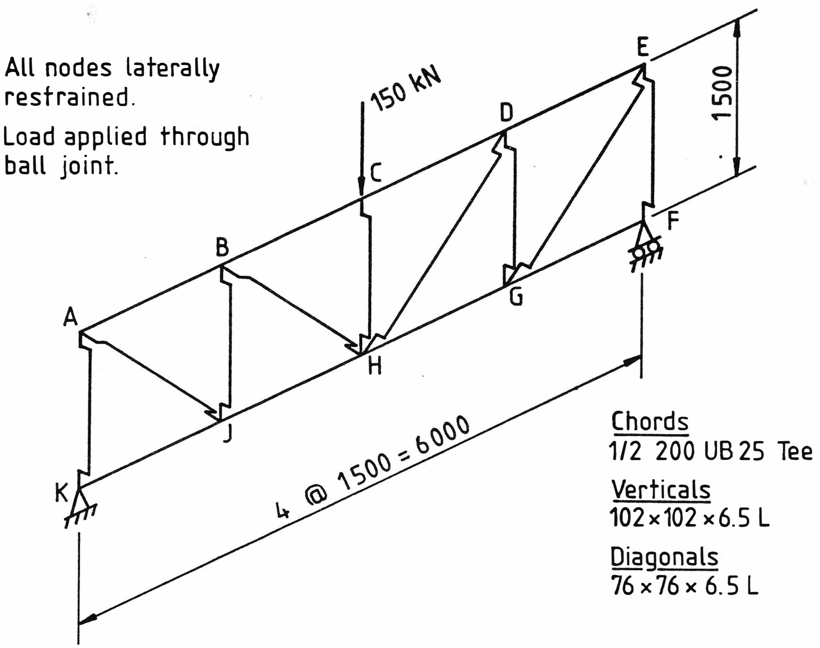


FIGURE 10 : Truss used for computer analysis

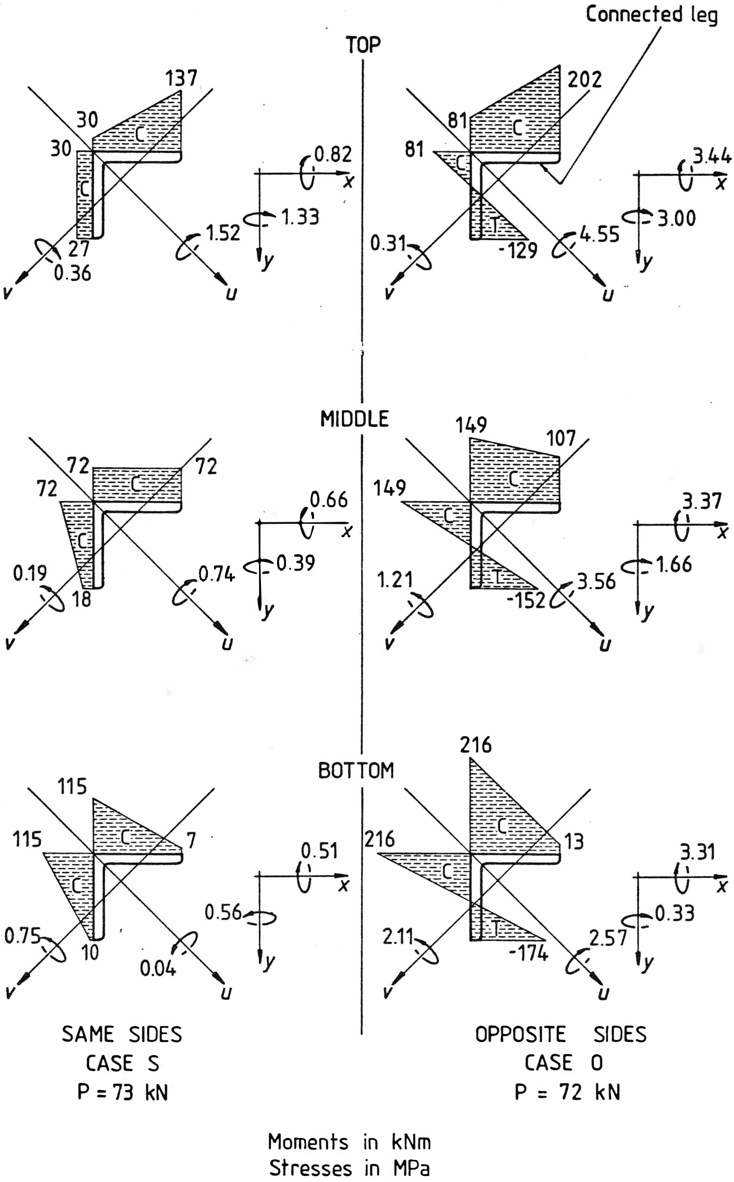


FIGURE 11 : Moments and stress distribution in member BJ of truss in Figure 10

of the two end moments in each case by the computed axial forces, the effective eccentricities for Case O and Case S are 46.8 and 9.0 mm respectively, which can be compared with the actual eccentricity of 30.2 mm used in the computer analysis.

It is evident that the effective eccentricity of 9.0 mm in Case S is much less than the 30.2 mm which would be assumed in design methods such as the one investigated by Trahair et al[17]. This is because the mutual out of plane restraint which exists between tension and compression members in this type of truss is normally ignored. In essence, the mutual restraint occurs because the tension and compression members tend to bow in opposite out of plane directions, thereby restraining each other. This does not occur when web members are connected on opposite sides, because they tend to bow in the same direction so that they are not mutually restraining.

One way of assessing the mutual restraint or lack of it between members is to determine the relative eccentricity between the tension and compression members at a joint as previously proposed by Leigh and Galambos[18] and the authors[22], and then distribute the resulting out of plane couple about the longitudinal axis of the chord in proportion to the out of plane rotational stiffnesses of the web members at the joint. This concept assumes that the ends of the web members are rigidly connected to each other at joints, and that the torsional stiffness of the web members and chords may be disregarded in distributing the out of plane couple. The latter assumption is justified by the computer results which show that relatively small torques are carried by truss members.

This approach can be used to explain why the effective eccentricity of 46.8 mm in Case O exceeds the $(c_x + t_G/2)$ eccentricity of 30.2 mm. The strut BJ is a vertical 102 x 102 x 6.5 angle restrained by 76 x 76 x 6.5 diagonal tension members so that the out of plane flexural stiffness of the strut is greater than that of the longer tension members. Consequently, the strut attracts more than its 30.2 mm share of the relative eccentricity e_r equal to 54.1 mm.

Using this approach, the difference in Mackey and Williamson's two test results shown in Figure 5 can also be explained. The P_{ult}/P_Y value of 0.48 for member BH of truss G1 is for a 51 x 51 x 6.3 diagonal strut restrained by 51 x 38 x 6.3 verticals with short legs outstanding while the P_{ult}/P_Y value of 0.25 for member CL of truss G2 is for a 57 x 57 x 6.3 vertical strut restrained by 51 x 38 x 6.3 diagonals with short legs outstanding. In other words, the higher results for the strut BH can be explained by the fact that it is flexurally less stiff perpendicular to the plane of the truss compared with its adjacent tension members than the strut CL, and is therefore more restrained out of the plane.

Each of the above cases is considered in more detail in the design examples.

4. COMPARISON OF DESIGN METHODS

In the light of the behaviour revealed by the computer studies in the previous section, it is now interesting to compare the various design methods. It is evident from Figure 5 that there is a surprisingly wide range of safe load capacities from the various design methods.

The current British method of BS 449:1969 allows much higher axial loads for stocky struts than the other methods. Some of the isolated strut failure loads obtained by Trahair, Usami and Galambos are actually lower than the British safe loads and one of the two truss member failure loads obtained by Leigh and Galambos is only 1.27 times the British safe load. It can be concluded that the current British method, which ignores eccentricity, does not give an adequate factor of safety for stocky struts and may even be unsafe if web members are connected on opposite sides of truss chords.

The earlier British method of BS 449:1948 allowed for eccentricity and bending in the plane perpendicular to the plane of the truss. While there is little difference between the 1948 and 1969 safe loads of BS 449 for slender struts, the 1948 safe loads for stocky struts are much smaller than the 1969 loads as demonstrated in Figure 5. Clearly, the effect of including eccentricity increases as the slenderness decreases.

Although the Trahair method and that of BS 449:1948 are similar in concept the safe loads obtained by the Trahair method are smaller for stocky struts and larger for slender struts. This is because the BS 449:1948 method is based on smaller eccentricities, higher slenderness ratios, different allowable axial and bending stresses, and a different combined stresses equation. The disadvantage of both methods is that they are based on fixed eccentricities and as such are not flexible enough to cater for the different situations in present day design.

The American and the current Australian approach require that eccentricity of force with respect to both principal axes be considered, with the force taken as acting in the mid-plane of the gusset at the centroid of the weld group or at the centre of gravity of fasteners. The safe loads vary according to the assumed position of the load along the mid-plane of the gusset as shown in Figure 5, but even the maximum safe loads by this method are less than those obtained by other methods. In the maximum load case, the point of application of load corresponds to the load position assumed by Trahair et al. However, the biaxial bending approach is based on a slenderness ratio of $0.85L/r_v$ whereas the Trahair method is based on the smaller ratio L/r_x, which explains the difference between the maximum biaxial bending safe load curve and the Trahair curve in Figure 5.

It can be concluded that the biaxial bending approach leads to conservative load capacities for single angle struts. Moreover, there is the problem of determining the centroid of the weld groups at each end. It is not always possible to balance the weld groups at each end and in any case, the design of end connections cannot be done until the member is sized. Even then, the centroid of the weld group may be different at each end which further complicates the design process.

The recommended design method which follows is independent of the distribution of weld and the position of fasteners (provided there are at least two fasteners at each end). In addition, the recommended method determines the out of plane eccentricity at each end of a strut according to the size, slope and eccentricity of its adjacent members and whether the web members are connected on one side or on opposite sides of the chords.

5. RECOMMENDED DESIGN METHOD

5.1 General

The design method recommended in this paper has been developed by the authors from the methods originally proposed by Trahair, Usami and Galambos[17] and Leigh and Galambos[18]. Basically, the recommended method uses the combined stresses interaction equation of AS 1250-1975[3] for buckling and bending in the plane perpendicular to the truss. Out of plane bending moments at the ends of each strut are calculated considering the eccentricity of connection and the interaction of out of plane flexure between adjacent web members.

5.2 Allowable Axial Compressive Stress F_{ac}

A pin-ended centrally loaded strut may buckle by twisting (torsional buckling), by bending (flexural buckling), or by a combined twisting and bending (flexural-torsional) mode[12], depending on the section dimensions and slendernesses of the member. Rule 6.1.1 of AS 1250-1975[3] basically assumes that members fail by a pure flexural mode, since the elastic critical stress F_{oc} is given as an Euler buckling stress. For angles whose outstand width to thickness ratios B/t are greater than $208/\sqrt{F_Y}$, the code limits the value of the allowable stress F_{ac} to 0.5 F_Y in order to guard against a torsional buckling mode.

Web compression members in a truss are neither centrally loaded nor pin-ended. Although the eccentricity of loading can be taken care of in the normal manner by considering combined axial and bending stresses, the influence of end restraints caused by the interaction of the strut with adjacent web and chord members is more difficult to deal with unless simplifying assumptions are made.

Trahair[16] presents a method of numerical solution for the flexural torsional buckling of single angle struts with elastic end restraints about any set of cross-sectional axes, but this is too cumbersome for use in design. Fortunately, it has been shown that web struts tend to buckle in a plane perpendicular to the truss, and it is therefore reasonable to consider flexural buckling alone about the x axis and to guard against torsional buckling by limiting F_{ac} to 0.5 F_Y as specified in AS 1250-1975. An effective length less than the distance L between intersection points could be justified because of the out of plane restraining influence of adjacent web members as the web strut buckles. However, because web members are not always rigidly connected to each other at joints as noted in Section 5.4.6, it is unwise to count on out of plane end restraint in determining the effective length.

In fact, an effective length greater than L may be possible, as Leigh and Galambos suggested, if the top end of a web strut is braced and the bottom end falls between bottom chord bracing points and is therefore laterally braced only by the transverse flexural stiffness of the bottom chord between these bracing points. However, any increases in effective length will not be as great as those obtained by Leigh and Galambos using the buckling model in Figure 8 because the applied loads should be "following loads" always acting in the plane AB in Figure 8 rather than in the plane parallel to the truss. This is because the direction of the applied axial load is controlled by the tension member which frames into the bottom chord joint, and the tension member must lie in the plane AB. In any case, a situation such as this is not peculiar to single angle trusses and will not be considered further in this paper.

To summarize, it is recommended that the allowable buckling stress F_{ac} be calculated from Section 6 of AS 1250-1975 using a slenderness ratio L/r_x.

5.3 Allowable Bending Stress F_{bcx}

The determination of the maximum permissible bending stress for an angle which is constrained to bend in the plane of one of its legs in the absence of axial force is fraught with difficulties. Leigh and Lay[23] consider laterally unsupported angles as beams but their safe loads are based on a deflection limit of span/180 which is not really an appropriate limit for the bending of truss members.

According to the basic principles of structural mechanics, bending moments and buckling should be considered about the principal axes. An angle bent about its minor axis or v axis (in the absence of axial compression) will not buckle in an overall flexural torsional mode. Therefore, the maximum permissible stress F_{bv} for bending about the v axis will lie between $0.60F_Y$ and $0.66F_Y$ depending on the outstand width to thickness ratio as outlined in Section 5 of AS 1250-1975. In the case of truss members, the bending about the v axis is such that the outstands are in tension and so F_{bv} can be taken as $0.66F_Y$. An angle can buckle when bent about its major axis or u axis, and expressions for the elastic critical stress F_{ob} are given for equal and unequal angles in Reference 23. For an equal angle, the elastic critical uniform moment is

$$M_{cu} = \frac{\pi}{L} \sqrt{EI_v \; GJ} \qquad (3)$$

for which it can be shown[23] that the elastic critical stress F_{ob} is given by

$$F_{ob} = \frac{\pi E}{2\sqrt{2.6}} \cdot \frac{t}{L}$$

$$= 195{,}000 \cdot \frac{t}{L} \qquad (4)$$

where 2.6 is the assumed value of Poisson's ratio. The allowable bending stress F_{bu} can be obtained by substituting F_{ob} in

Equations 5.4.3 of AS 1250-1975 in the normal manner.

To attain the maximum possible value of $0.66F_Y$ for F_{bu}, the elastic critical stress F_{ob} must be at least three times the yield stress, a fact which can be deduced from Equation 5.4.3 (2) of AS 1250. Substituting $3F_Y$ into Equation 4 above, it follows that if the ratio L/t is less than $65000/F_Y$, then F_{bu} can be taken as $0.66F_Y$.

Having determined the allowable bending stresses F_{bu} and F_{bv}, the calculated bending stresses f_{bu} and f_{bv} (in the absence of axial force) can be combined using the following equation

$$\frac{f_{bu}}{F_{bu}} + \frac{f_{bv}}{0.66F_Y} \leqslant 1 \qquad (5)$$

where f_{bu} and f_{bv} may be tensile or compressive.

For an angle beam which is constrained to deflect perpendicular to its x axis for example, the bending stresses can be determined from My/I_x without considering the principal axes. It is therefore convenient to compare the resulting maximum bending stress, tensile or compressive, against an allowable value F_{bx}. Since F_{bu} is always less than or equal to $0.66F_Y$ in Equation 5, it is conservative to take F_{bx} equal to F_{bu}.

In the flexure of equal angle truss members, the tensile stress in the outstand exceeds the compressive stress in the connected leg. The tensile outstand stress is reduced by the compressive stress f_{ac} due to the presence of the axial force, but just how these tensile and compressive stresses should be combined is not clear from AS 1250. In the absence of more precise information, it will be assumed that the nett tensile stress in the outstand due to bending and axial compression should be limited to $0.66F_Y$. The maximum allowable compressive bending stress in the connected leg will be denoted F_{bcx} rather than F_{bx}.

5.4 Member End Moments

5.4.1 General

Single angle trusses with web members connected by the double-bolting or welding of one leg are three dimensional rigid frames with members bent about both principal axes, and twisted. Because truss experiments have shown that single angle web compression members deflect predominantly in a plane perpendicular to the truss[15,18], it is convenient, if unorthodox, to consider out of plane and in plane behaviour rather than principal axis bending. Out of plane behaviour could be classified as that due to out of plane eccentricity and in plane behaviour due to rigid frame moments in the absence of out of plane eccentricity.

In reality, for a single angle strut to deflect in a plane perpendicular to the truss, there must be both an out of plane moment (about the x-axis) and an in plane moment (about the y-axis). This is evident from Table 1 in which the 32.4 mm value of x_P to cause out of plane deflection only, is equivalent to a 13.8 mm eccentricity about the y-axis and a 23.6 mm eccentricity about the x-axis. Without the presence of both moments, the bending stress equation $f_{bcx} = Pec_x/I_x$ would not be valid.

Similarly, if it were possible to connect the web members without eccentricity but with the principal axes still inclined to the frame axes, the presence of 'in plane' rigid frame action would cause both in plane and out of plane moments.

The secondary stresses due to in plane rigid frame action are traditionally disregarded in triangulated truss analysis. This stems from pre-computer days when it was virtually impossible to determine the rigid frame moments manually and so trusses were regarded as pin-jointed. Nowadays, it is just as easy to use a rigid frame computer program to analyse a truss as it is to use a pin-jointed truss program. Generally, the in plane moments obtained are not significant. Moreover, the moments are such as to bend the web members in double curvature in the plane of the truss, which is not as severe a combined stress situation as bending in single curvature.

In short, it is assumed in the recommended design method that in plane rigid frame effects will not be critical in the design of single angle struts, and are therefore disregarded.

5.4.2 Axes

For web members and chords, the z axis is the longitudinal axis with the x axis being a cross-sectional axis in the plane of the truss and the y axis completing an orthogonal right handed system as shown in Figure 12a. The out of plane moments can be considered with respect to the x and z chord axes, with moments and rotations obeying the right hand screw rule.

5.4.3 Out of plane couples M_{Tz} and M_{Bz}

The eccentric connection of the single angle web members results in a couple of forces about the x and z chord axes. For the strut shown in Figure 12b, the out of plane couple M_{Tz} about the longitudinal axis of the top chord can be expressed as

$$M_{Tz} = - \left(P_2 e_2 \sin \gamma_{2T} + P_3 e_3 \sin \gamma_{3T}\right) \qquad (6)$$

where P_2 and P_3 are the axial forces in members 2 and 3 with compression positive. Members are numbered so that 2 and 3 intersect at the top chord;

e_2 and e_3 are the transverse eccentricities of members 2 and 3 measured in the y direction from the longitudinal centroidal axis of the chord;

and γ_{2T} and γ_{3T} are the angles between the centroidal axes of the chord and members 2 and 3 measured anticlockwise from the chord z axis. These angles are always positive.

If there are no chord shear forces or external loads applied at the joint, then the components of the member forces

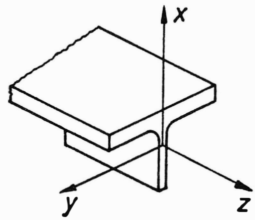


FIGURE 12a : Chord axes

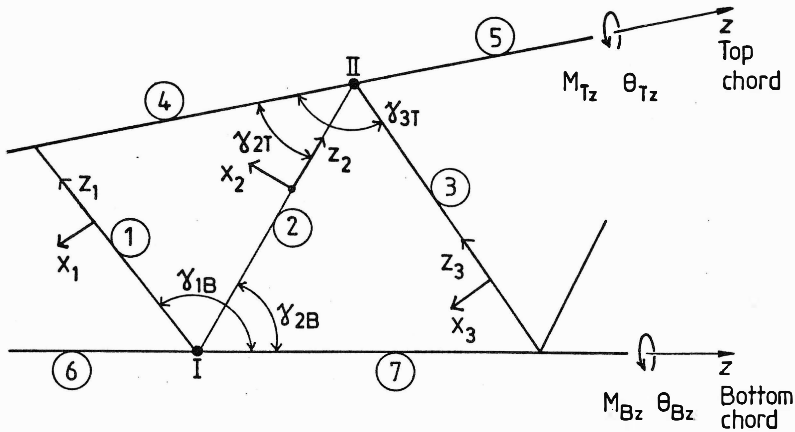


FIGURE 12b : Definition of truss geometry normal to the chord must be equal and opposite, in which case the expression for M_{Tz} can be more simply written as

$$M_{Tz} = - P_2 \sin \gamma_{2T} (e_2 - e_3) \qquad (7)$$

In other cases where there are other than two web members at a joint or where there are external loads applied to the top chord, the moment M_{Tz} can be determined in a similar manner.

The corresponding expression for the out of plane couple M_{Bz} at the bottom chord joint is given by

$$M_{Bz} = P_2 e_2 \sin \gamma_{2B} + P_1 e_1 \sin \gamma_{1B} \qquad (8)$$

which in the absence of external loads applied to the chord can be more simply expressed as

$$M_{Bz} = P_2 \sin \gamma_{2B} (e_2 - e_1) \qquad (9)$$

It should be noted that the eccentricities e_1, e_2 and e_3 have the same sign when the web members are all connected on the same side of the chords.

5.4.4 Out of plane couples M_{Tx} and M_{Bx}

The eccentricity of web member forces also gives rise to out of plane couples M_{Tx} and M_{Bx} about the chord x axes. For the top chord in Figure 12b the couple M_{Tx} can be expressed as

$$M_{Tx} = P_2 e_2 \cos \gamma_{2T} + P_3 e_3 \cos \gamma_{3T} \qquad (10)$$

while for the bottom chord

$$M_{Bx} = - (P_2 e_2 \cos \gamma_{2B} + P_1 e_1 \cos \gamma_{1B}) \qquad (11)$$

5.4.5 Distribution of couples

Assuming that web members are rigidly connected to each other at joints, the out of plane couples M_{Tz} and M_{Tx} can be distributed amongst the chord and web members framing into the joint as end moments and torques. Fortunately, as confirmed by the three dimensional computer analysis, the torsional stiffness of open sections such as angles, channels and tees is extremely low relative to their flexural stiffnesses and any torques developed may be ignored. The out of plane member end moments can then be determined by the solution of the following simultaneous equations expressing M_{Tz} and M_{Tx} in terms of the rotations θ_{Tx} and θ_{Tz} at joint II.

$$M_{Tx} = A_T\theta_{Tx} + B_T\theta_{Tz} \qquad (12)$$

$$M_{Tz} = C_T\theta_{Tx} + D_T\theta_{Tz} \qquad (13)$$

where

θ_{Tx} is the rotation of joint II about the chord x axis

θ_{Tz} is the rotation of joint II about the chord z axis

$$A_T = E\left(\frac{\alpha_4 I_4}{L_4} + \frac{\alpha_5 I_5}{L_5} + \frac{\alpha_2 I_2}{L_2}\cos^2\gamma_{2T} + \frac{\alpha_3 I_3}{L_3}\cos^2\gamma_{3T}\right) \qquad (14)$$

$$B_T = E\left(-\frac{\alpha_2 I_2}{L_2}\sin\gamma_{2T}\cos\gamma_{2T} - \frac{\alpha_3 I_3}{L_3}\sin\gamma_{3T}\cos\gamma_{3T}\right) \qquad (15)$$

$$C_T = B_T \qquad (16)$$

$$D_T = E\left(\frac{\alpha_2 I_2}{L_2}\sin^2\gamma_{2T} + \frac{\alpha_3 I_3}{L_3}\sin^2\gamma_{3T}\right) \qquad (17)$$

I_2 to I_5 are the second moments of area about the chord and web member x axes

L_2 to L_5 are the member lengths between intersection points

E is Young's modulus

and

α_2 to α_5 are stiffness coefficients.

The value of the stiffness coefficients α_2 and α_3 for the web members depends on the conditions at the bottom of the members. If the bottom end is effectively fixed against out of plane rotation, α_2 and α_3 would equal 4. If the conditions at the bottom end are identical to those at the top, then α_2 and α_3 would equal 2.

The value of the coefficients α_4 and α_5 for the chord members depends on the spacing of lateral chord restraints such as purlins and on the nature of the out of plane couples M_{Tx} at the adjacent top chord joints. If for example, there are no lateral restraints, but there are a number of identical joints at regular intervals with identical couples M_{Tx}, then the chord will deflect laterally as shown in Figure 13.

In this case, M_4 and M_5 can be expressed as follows

$$M_4 = \frac{3EI_4}{\frac{L_4}{2}} . \theta_{Tx} \qquad (18)$$

$$M_5 = \frac{3EI_5}{\frac{L_5}{2}} . \theta_{Tx} \qquad (19)$$

and therefore

$$\alpha_4 = \alpha_5 = 6 \qquad (20)$$

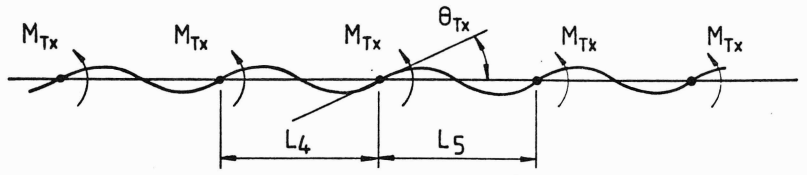


FIGURE 13 : Plan on top chord

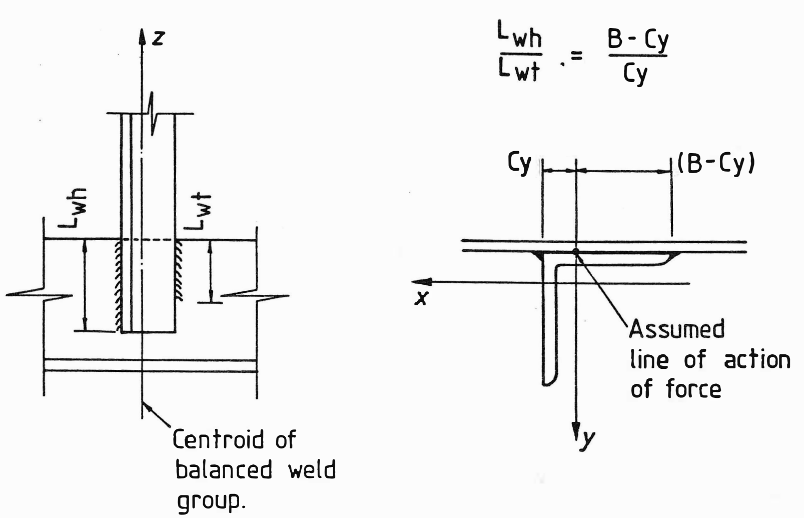


FIGURE 14 : Balanced weld design

Solving Equations 12 and 13 for θ_{Tx} and θ_{Tz}, the end moment at the top of strut 2 about its x axis can be expressed as

$$M_{2T} = \frac{2EI_2}{L_2} \cdot \frac{\left(C_T \sin\gamma_{2T} + D_T \cos\gamma_{2T}\right) M_{Tx} - \left(B_T \cos\gamma_{2T} + A_T \sin\gamma_{2T}\right) M_{Tz}}{A_T\, D_T - B_T\, C_T} \qquad (21)$$

In most cases, the transverse stiffness of the chords will be much greater than that of the web members and consequently, the coefficient A_T will be much greater than the coefficients B_T, C_T and D_T. In these cases, the moment M_{2T} can be approximated by

$$M_{2T} \simeq - \frac{2EI_2}{L_2} \cdot \frac{\sin\gamma_{2T}\, M_{Tz}}{D_T} \qquad (22)$$

i.e.

$$M_{2T} \simeq - \frac{I_2 L_3 \sin\gamma_{2T}}{I_2 L_3 \sin^2\gamma_{2T} + I_3 L_2 \sin^2\gamma_{3T}} \cdot M_{Tz} \qquad (23)$$

The moment at the bottom of strut 2 can be similarly expressed as

$$M_{2B} \simeq - \frac{I_2 L_1 \sin\gamma_{2B}}{I_2 L_1 \sin^2\gamma_{2B} + I_1 L_2 \sin^2\gamma_{1B}} \cdot M_{Bz} \qquad (24)$$

Similar expressions can be easily derived when there are more than two web members framing into a joint as demonstrated in Design Example 2.

5.4.6 Minimum end moments

The foregoing distribution of out of plane couples is based on the assumption that the web members are rigidly connected to each other at joints. This is a reasonable assumption when web members are connected on opposite sides so that the ends of the members overlap and are therefore connected directly back to back (except for the gusset plate or Tee chord stem in between). When web members are all connected on one side, there is usually some flexibility in the joint because the ends do not overlap and are thus connected to a gusset plate or to the thin stem of a Tee chord. Unfortunately, there is insufficient information available on the experimental behaviour of single angle truss joints, and so it is necessary to set a conservative minimum out of plane end moment to allow for joint flexibility.

The permissible axial stresses for single angles in BS 449:1948 were derived on the assumption that the load was applied through the mid-plane of the connected leg which corresponds to an eccentricity of $(c_x - t/2)$. Mackey and Williamson[15] found for their truss G2 that the out of plane end moments for double bolted members connected to 8 mm gusset plates were less than $P(c_x - t/2)$. It is therefore recommended that minimum end moments based on an eccentricity of $(c_x - t/2)$ be used in the design of single angle struts, whether connected on the same sides or on opposite sides. Hopefully, future experimental work will lead to smaller minimum moments.

5.5 Design of Welded Connections

Welded connections for single angle struts are often designed by the balanced weld method[6,25] in which more weld is placed along the heel than along the toe to ensure that the centroid of the weld group lies on the y axis as shown in Figure 14. This approach is based on the assumption that the line of action of the axial force is in the plane of the welds and passes through the y axis, any in-plane or out of plane moments transmitted by the weld group being ignored. In this case, the total length of weld is determined by dividing the axial force by the capacity of the weld per unit length. If the weld group is not balanced, the in-plane moment of force with respect to the centroid of the weld group can be included in the design.

However, as mentioned earlier, the line of action of the axial force under elastic conditions is virtually independent of the placement of welds. In fact, the line of action is nearer the mid-point of the connected leg for both equal and unequal angles, regardless of the weld distribution.

It would therefore seem more logical, if out of plane moments carried by the weld group are not considered, to balance the welds about the mid-point of the connected leg.

In practice, the weld group will have to transmit out of plane moments. For any eccentricially connected member, the couple due to eccentricity at each end must be carried either by the member or by the supports, or be shared between both, depending on the relative stiffnesses and strengths of the member and its supports. In the design of weld groups for angles, out of plane moments are traditionally disregarded[6,24,25] even when the angle is connected to a rigid support so that all of the moment is taken by the support and the weld group, and none by the member. For example, the weld groups for double angles connected back to back to a gusset plate are not designed for out of plane moments[24], apparently without adverse effects, even though the weld group for each angle must carry the full moment due to the eccentricity of connection of each angle. Whether disregarded or not, the out of plane moments which do exist will be carried more by the heel weld than the toe weld, and so there is some argument for balancing welds in the conventional mamner.

In conclusion, without a detailed experimental investigation of truss joints, there is no justification at this stage for including out of plane moments in the design of welded single angle truss joints, or for departing from the conventional procedure of balanced weld design.

5.6 Summary of Recommended Design Method

(1) Determine axial forces in web members and stress f_{ac} in web strut.

(2) Determine effective length ℓ and the slenderness ratio ℓ/r_x. The effective length will equal L unless there are some external restraints.

(3) Determine F_{ac} and F_{ocx}.

(4) Calculate M_{Tz} and M_{Bz}. Use Equations 7 and 9 if there are two web members framing into the joint and there are no externally applied forces. Similar equations can be derived for other cases.

(5) Calculate M_{2T} and M_{2B}. Use Equations 23 and 24 if there are two web members framing into the joint. Ensure that M_{2T} and M_{2B} are not less than the minimum moment $P_2 (c_x - t/2)$.

(6) Calculate the maximum compressive stress f_{bcx} using the numerically larger of the two moments M_{2T} and M_{2B}.

$$f_{bcx} = \frac{M_2 \; c_x}{I_x}$$

If in the unlikely situation, the end moments cause the outstand to be more highly stressed in compression than the heel, then

$$f_{bcx} = \frac{M_2 \; (B - c_x)}{I_x}$$

(7) Calculate C_{mx} from $C_{mx} = 0.6 - 0.4\beta$, where β is the ratio of the smaller to the larger moment. Normally, the end moments will be of opposite sign so that the member will be bent in single curvature and β will be negative.

(8) Determine F_{bcx}. If $L/t < 65000F_Y$, $F_{bcx} = 0.66F_Y$.

(9) Substitute the above values into the combined stresses equation of Clause 8.2.1(a) of AS 1250-1975.

(10) Check the combined stress ratio at supports using Clause 8.2.1(b).

(11) Check tensile outstand stress is less than $0.66F_Y$.

(12) Select new member sizes if the combined stress ratios are too small or greater than unity.

Step 4 may be eliminated and Step 5 considerably simplified for a one sided truss by conservatively assuming equal and opposite end moments of magnitude $P_2 c_x$. Safe Load Tables for single angle struts in one sided trusses could be prepared on this basis. These safe loads would be slightly larger than those obtained by the method of Trahair, Usami and Galambos, the difference being due to smaller assumed eccentricities in the former method.

If the end connections are to be welded, the conventional method of balanced weld design should be used wherever possible.

6. DESIGN EXAMPLES

6.1 Design Example 1

A. Using the recommended design method, check the member CL of Mackey and Williamson's truss G2 shown in Figure 4 for two symmetrically placed truss loads of 32.5 kN each.

B. Calculate the allowable member capacity using this method and determine the factor of safety compared with the experimental failure load of the member.

C. Compare the allowable capacity and the factor of safety with those of the current British method.

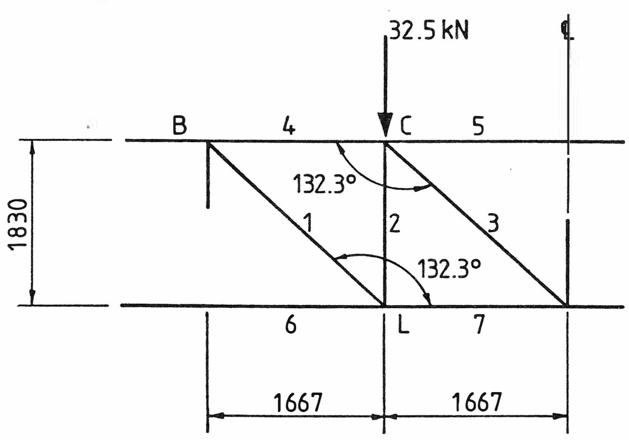


FIGURE 15 : Design Example 1

NOTES:
1. Loads applied symmetrically about truss centreline.
2. Loads applied through ball joints.
3. Double bolted joints.
4. Gusset plates 8 mm thick.
5. Lateral restraints at joints A, B, D, F and G.

Members 1 and 3:

51 x 38 x 6.3 ∠
Short leg out

$I_1 = I_3 = 6.59 \times 10^4\ mm^4$
$e_1 = e_3 = 10.3 + 8/2 = 14.3$
$L_1 = L_3 = 2475$

Member 2:

57 x 57 x 6.3 ∠
$F_Y = 280$ MPa

$I_2 = 20.6 \times 10^4\ mm^4$
$A = 682\ mm^2$
$c_{x2} = 16.4$
$e_2 = 16.4 + 8/2 = 20.4$
$r_{x2} = 17.4$
$L_2 = 1830$

PART A:

1. $P_1 = -\frac{32.5}{\sin(132.3)} = -43.96$ kN (tension)

$P_2 = 32.5$ kN

$P_3 = 0$

$f_{ac} = \frac{32.5 \times 10^3}{682} = 48$ MPa

2. $\ell = 1830$

$\frac{\ell}{r_x} = \frac{1830}{17.4} = 105$

3. $F_{ac} = 77$ MPa
$F_{ocx} = 179$ MPa

4. $M_{Tz} = -32.5 \times .0204 = -0.663$ kNm
$M_{Bz} = 32.5 \times (.0204 - .0143) = 0.198$ kNm

5. $$M_{2T} = \left[\frac{20.6 \times 10^4 \times 2475}{20.6 \times 10^4 \times 2475 + 6.59 \times 10^4 \times 1830 \times \sin^2(132.3)}\right](-0.663)$$

$$= 0.886 \times 0.663$$

$$= 0.587 \text{ kNm}$$

$$M_{2B} = -0.886 \times 0.198$$

$$= -0.175 \text{ kNm}$$

Minimum moment $= 32.5 \times (.0164 - .0063/2)$

$= 0.431$ kMn

$\therefore M_{2T} = 0.587$ as before

and $M_{2B} = 0.431$ kNm

6. $$f_{bcx} = \frac{0.587 \times 10^6 \times 16.4}{20.6 \times 10^4} = 47 \text{ MPa}$$

7. $$C_{mx} = 0.6 + 0.4 \times \frac{.431}{.587} = 0.89$$

8. $$\frac{L}{t} = \frac{1830}{6.3} = 290 > \frac{65000}{280} = 232$$

$$\therefore F_{ob} = \frac{195000}{290} = 672 \text{ MPa}$$

$$\therefore F_{bcx} = F_{bu} = \left(.95 - .50\sqrt{\frac{280}{672}}\right) 280 = 176 \text{ MPa}$$

9. Combined stresses Clause 8.2.1(a)

$$\frac{48}{77} + \frac{.89 \times 47}{\left(1 - \frac{48}{.6 \times 179}\right) \times 176}$$

$= 0.62 + 0.44 = 1.06 > 1.0$ No good

10. Combined stresses Clause 8.2.1(b)

$$\frac{48}{.6 \times 280} + \frac{47}{172}$$

$= 0.29 + 0.27 = 0.56 < 1.0$ O.K.

11. Outstand stress $= 48 - \dfrac{0.587 \times 10^6 \times (57 - 16.4)}{20.6 \times 10^4}$

PART B:

Max. safe load $\simeq \frac{32.5}{1.06} = 30.7$ kN

Experimental failure load = 66.4 kN

Factor of safety $= \frac{66.4}{30.7} = 2.16$

PART C: British method

$$\frac{\ell}{r_v} = \frac{0.85 \times 1830}{11.2} = 139$$

F_{ac} = 47 MPa

P_{safe} = 47 x .682

= 32.1 kN

Factor of safety $= \frac{66.4}{32.1} = 2.07$

6.2 Design Example 2

A. Using the recommended design method, check the member BH of Mackey and Williamson's truss G1 as shown in Figure 3 for truss loads of 23 kN at the third points.

B. Calculate the allowable member capacity using this method and determine the factor of safety compared with the experimental failure load of the member.

C. Compare the allowable capacity and the factor of safety with those of the current British method.

NOTES:
1. Assume gusset plates 8 mm thick.
2. Double bolted joints.
3. Complete rotational and translational freedom at both support points and one loading point.

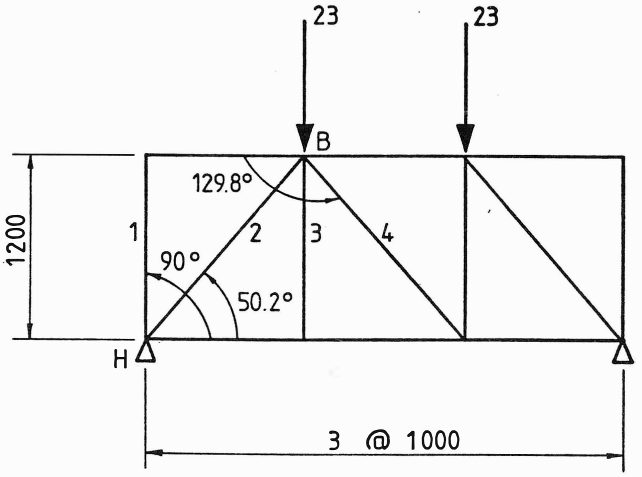


FIGURE 16 : Design Example 2

Members 1 and 3: $I_1 = I_3 = 6.59 \times 10^4 \text{ mm}^4$

51 x 38 x 6.3 L $e_1 = e_3 = 10.3 + 8/2 = 14.3$

Short leg out $L_1 = L_3 = 1200$

Members 2 and 4: $I_2 = I_4 = 0.143 \times 10^6 \text{ mm}^4$

51 x 51 x 6.3 L $c_{x2} = c_{x4} = 14.9$

$F_Y = 275$ MPa $e_2 = e_4 = 14.9 + 4 = 18.9$

$r_{x2} = 15.3$

$L_2 = 1562$

$A = 608 \text{ mm}^2$

<u>PART A:</u>

1. $P_1 = P_3 = P_4 = 0$

$$P_2 = \frac{23}{\sin(50.2)} = 29.9 \text{ kN}$$

$$f_{ac} = \frac{29.9 \times 10^3}{608} = 49 \text{ MPa}$$

2. $\ell = L = 1562$ (no rotational restraints applied by load or support)

$$\frac{\ell}{r_x} = \frac{1562}{15.3} = 102$$

3. $F_{ac} = 80$ MPa

$F_{oc} = 190$ MPa

4. $M_{Tz} = -\ 23.0 \times .0189 = -\ .435$ kNm

$M_{Bz} = 0.435$ kNm

5. $$M_{2T} = -\frac{\frac{I_2}{L_2}\sin\gamma_{2T}}{\frac{I_2}{L_2}\sin^2\gamma_{2T} + \frac{I_3}{L_3}\sin^2\gamma_{3T} + \frac{I_4}{L_4}\sin^2\gamma_{4T}} \cdot M_{Tz}$$

$$= -\left[\frac{\frac{0.143 \times 10^6}{1562}\sin(50.2)}{\left(\frac{0.143 \times 10^6}{1562}\sin^2(50.2)\right) \times 2 + \frac{6.59 \times 10^4}{1200}}\right](-\ 0.435)$$

$$= 0.432 \times 0.435$$

$$= 0.188 \text{ kNm}$$

$$M_{2B} = -\left[\frac{0.143 \times 10^6 \times 1200 \times \sin(50.2)}{0.143 \times 10^6 \times 1200 \times \sin^2(50.2) + 6.59 \times 10^4 \times 1562}\right] \times 0.473$$

$$= -\ 0.646 \times 0.435$$

$$= -\ 0.281 \text{ kNm}$$

Check minimum end moment

$$= 29.9 \times \left(.0149 - \frac{.0063}{2}\right) = 0.351 \text{ kNm}$$

$\therefore\ M_{2T} = 0.351$ kNm

and $M_{2B} = -\ 0.351$ kNm

6. $$f_{bcx} = \frac{0.351 \times 10^6 \times 14.9}{0.143 \times 10^6} = 37 \text{ MPa}$$

7. $C_{mx} = 1.0$

8. $$\frac{L}{t} = \frac{1562}{6.3} = 248 > \frac{65000}{275} = 236$$

$$\therefore\ F_{ob} = \frac{195000}{248} = 786 \text{ MPa}$$

$$\therefore\ F_{bcx} = F_{bu} = \left(.95 - .50\sqrt{\frac{275}{786}}\right) 275 = 180 \text{ MPa}$$

9. Combined stresses Clause 8.2.1(a)

$$\frac{49}{80} + \frac{37}{\left(1 - \frac{49}{.6 \times 190}\right) \times 180}$$

$$= 0.61 + 0.36 = 0.97 < 1.0 \quad \text{O.K.}$$

10. Combined stresses Clause 8.2.1(b)

$$\frac{49}{.6 \times 275} + \frac{37}{180}$$

$$= 0.30 + 0.20 = 0.50 \quad \text{O.K.}$$

11. Outstand stress $= 49 - \dfrac{.351 \times 10^6 \times (51 - 14.9)}{.143 \times 10^6}$

$$= -40 \text{ MPa} \quad \text{O.K.}$$

PART B:

Max. safe load $\simeq \dfrac{29.9}{.97} = 30.8$ kN

Experimental failure load = 80.26 kN

Factor of safety $= \dfrac{80.26}{30.8} = 2.61$

PART C: British method

$$\frac{\ell}{r_v} = \frac{.85 \times 1562}{9.89} = 134$$

F_{ac} = 50 MPa

P_{safe} = 50 x .608

= 30.4 kN

Factor of safety $= \dfrac{80.26}{30.4} = 2.64$

6.3 Design Example 3

A. Using the recommended design method, check the fourth compression web member of Leigh and Galambos's truss J9 shown in Figure 9 for the case of two loads of 33 kN applied symmetrically about the truss centreline.

B. Calculate the allowable member capacity using this method and determine the factor of safety compared with the experimental failure load of the member.

C. Compare the allowable capacity and the factor of safety with those of the current British method.

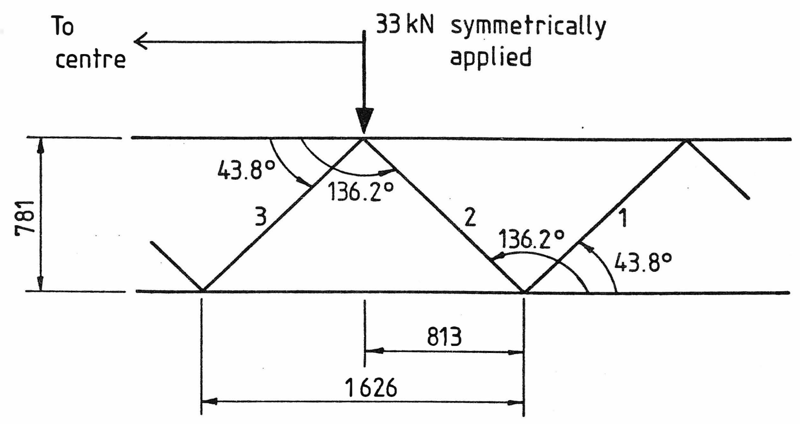


FIGURE 17 : Design Example 3

NOTES:
1. Loads applied through ball joint.
2. Joints welded.
3. Chords composed of two angles stitch welded back to back to form Tee.
4. Web members numbered so that 2 and 3 meet at top chord.

Member 1:

34.7 x 34.7 x 4.7 L

$I_1 = 3.32 \times 10^4$ mm^4

$c_{x1} = 10.4$

$e_1 = 10.4 + 6.9 = 17.3$

$L_1 = 1127$

Member 2:

53.6 x 53.6 x 5.7 $\angle$

F_Y = 374 MPa

$I_2 = 1.55 \times 10^5$ mm^4

$A = 578$ mm^2

$c_{x_2} = 15.5$

$e_{2Top} = 15.5 + 8.6 = 24.1$

$e_{2Btm} = 15.5 + 6.9 = 22.4$

$r_{x_2} = 16.4$

$L_2 = 1127$

Member 3:

27.4 x 27.4 x 3.3 $\angle$

$I_3 = 1.16 \times 10^4$ mm^4

$c_{x_3} = 8.1$

$e_3 = 8.1 + 8.6 = 16.7$

$L_3 = 1127$

PART A:

1. $$P_1 = -\frac{33}{\sin(43.8)} = -47.7 \text{ kN}$$

$$P_2 = 47.7 \text{ kN}$$

$$P_3 = 0$$

$$f_{ac} = \frac{47.7 \times 10^3}{578} = 83 \text{ MPa}$$

2. $$\ell = L = 1127$$

$$\frac{\ell}{r_x} = \frac{1127}{16.4} = 69$$

3. F_{ac} = 160 MPa $\quad F_{oc}$ = 415 MPa

4. $$M_{Tz} = -33 \times .0241 = -0.795 \text{ kNm}$$

$$M_{Bz} = 33 \times (.0241 - .0173) = 0.224 \text{ kNm}$$

5. $$M_{2T} = -\left[\frac{1.55\text{x}10^5\text{x}1127\text{x}\sin(136.2)}{1.55\text{x}10^5\text{x}1127\text{x}\sin^2(136.2)+1.16\text{x}10^4\text{x}1127\text{x}\sin^2(43.8)}\right](-.795)$$

$$= 1.344 \times 0.795$$

$$= 1.07 \text{ kNm}$$

$$M_{2B} = - \left[\frac{1.55 \times 10^5 \times \sin (136.2)}{1.55 \times 10^5 \times \sin^2 (136.2) + 3.32 \times 10^4 \times \sin^2 (43.8)} \right] \times 0.224$$

$$= - 1.190 \times .224$$

$$= - 0.267 \text{ kNm}$$

Check minimum moment $= 47.7 \times (.0155 - .0057/2)$

$= 0.595$ kNm

$\therefore M_{2T} = 1.07$ kNm as before

and $M_{2B} = - 0.595$ kNm

6. $f_{bcx} = \dfrac{1.07 \times 10^6 \times 15.5}{1.55 \times 10^5} = 107$ MPa

7. $C_{mx} = 0.6 + 0.4 \times \dfrac{.595}{1.07} = 0.82$

8. $\dfrac{L}{t} = \dfrac{1127}{5.7} = 198 > \dfrac{65000}{374} = 174$

$\therefore F_{ob} = \dfrac{195000}{198} = 985$ MPa

$\therefore F_{bcx} = F_{bu} = \left(.95 - .50\sqrt{\dfrac{374}{985}}\right) \times 374 = 240$ MPa

9. Combined stresses Clause 8.2.1(a)

$$\frac{83}{160} + \frac{0.82 \times 107}{\left(1 - \dfrac{83}{.6 \times 415}\right) \times 240} = 0.52 + 0.55 = 1.07 > 1.0$$

10. Combined stresses Clause 8.2.1(b) clearly O.K.

11. Outstand stress $= 83 - 263 = - 180 < .66 \times 374 = 247$ O.K.

PART B:

Max. safe load $\doteqdot \dfrac{47.7}{1.07} = 44.6$ kN

Experimental failure load $= \dfrac{12.8 \text{ kip} \times 4.45}{\sin (43.8)}$

$= 82.3$ kN

Factor of safety $= \dfrac{82.3}{44.6} = 1.85$

PART C: British Method

$$\frac{\ell}{r_v} \div \frac{.85 \times 1127}{\left(\frac{16.4}{1.55}\right)} = 91$$

F_{ac} = 112 MPa

P_{safe} = 112 x .578

= 64.7 kN

Factor of safety $= \frac{82.3}{64.7} = 1.27$

DESIGN EXAMPLE 4

A Using the recommended design method, determine the allowable capacity of the web member BJ of the truss shown in Figure 10. The truss has all web members connected on the same side and is acted on by a central concentrated load.

B Repeat Part A but with web members connected alternately on opposite sides.

C Calculate the allowable capacity to the current British method.

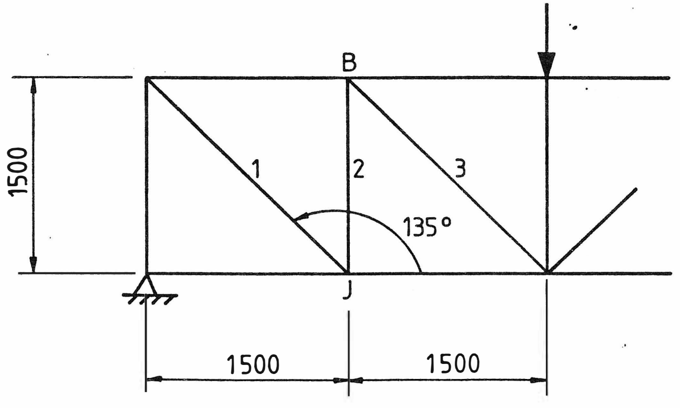


FIGURE 18 : Design Example 4

D For the allowable capacity from Part B, determine the biaxial combined stress ratios if the force is assumed to act in the mid plane of the web of the Tee chord through the y axis.

Members 1 and 3: $I_1 = I_3 = 0.496 \times 10^6\ mm^4$

76 x 76 x 6.5 $\angle$ $c_{x_1} = c_{x_3} = 21.0$

same sides $e_1 = e_3 = 21.0 + \frac{5.8}{2} = 23.9$

opposite sides $e_1 = e_3 = -23.9$

$L_1 = L_3 = 2121$

Member 2: $I_2 = 1.24 \times 10^6\ mm^4$

102 x 102 x 6.5 $\angle$ $A = 1260\ mm^2$

$F_Y = 260$

$c_{x_2} = 27.3$

$e_2 = 27.3 + 5.8/2 = 30.2$

$r_{x_2} = 31.5$ $L_2 = 1500$

PART A:

1. Try central concentrated truss load of 210 kN

$$P_1 = -105\sqrt{2} = -148\ kN$$

$$P_2 = 105\ kN$$

$$P_3 = -148\ kN$$

$$f_{ac} = \frac{105 \times 10^3}{1260} = 83\ MPa$$

2. $\ell = L = 1500$

$$\frac{\ell}{r_x} = \frac{1500}{31.5} = 47.6$$

3. $F_{ac} = 142\ MPa$; $F_{oc} = 879\ MPa$

4. $M_{Tz} = -105 \times (.0302 - .0239) = -0.662\ kNm$

$M_{Bz} = 0.662\ kNm$

5. $$M_{2T} = -\left[\frac{1.24 \times 10^6 \times 2121}{1.24 \times 10^6 \times 2121 + 0.496 \times 10^6 \times 1500 \times .50}\right](-0.662)$$

$= 0.876 \times .662$

$= 0.579$ kNm

$M_{2B} = -\ 0.579$ kNm

Minimum moment $= 105 \times (.0273 - .0065/2)$

$= 2.53$ kNm

$\therefore M_{2T} = 2.53$ kNm

and $M_{2B} = -\ 2.53$ kNm

6. $f_{bcx} = \dfrac{2.53 \times 10^6 \times 27.3}{1.24 \times 10^6} = 56$ MPa

7. $C_{mx} = 1.0$

8. $\dfrac{L}{t} = \dfrac{1500}{6.5} = 230 < \dfrac{65000}{250} = 260$

$\therefore F_{bcx} = 0.66 \times 260 = 172$ MPa

9. Combined stresses Clause 8.2.1(a)

$$\frac{83}{130} + \frac{56}{\left[1 - \dfrac{83}{.6 \times 879}\right] \times 172}$$

$= 0.64 + 0.39 = 1.03 < 1.0$ but O.K.

10. Combined stresses Clause 8.2.1(b) clearly not critical

11. Outstand stress $= 83 - 152 = -\ 69$ MPa O.K.

$\therefore$ Max. safe load $\doteqdot \dfrac{105}{1.03} = 102$ kN (same sides)

PART B:

1. Try central concentrated truss load of 165 kN

$P_1 = -\ 82.5 \sqrt{2} = -\ 117$ kN

$P_2 = 82.5$ kN

$P_3 = -\ 117$ kN

$$f_{ac} = \frac{82.5 \times 10^3}{1260} = 65 \text{ MPa}$$

2. & 3. $F_{ac} = 130$ MPa; $F_{oc} = 879$ MPa

4. $M_{Tz} = - 82.5x(.0302 + .0239) = - 4.46$ kNm

$M_{Bz} = 4.46$ kNm

5. $M_{2T} = 0.876 \times 4.46$

$= 3.91$ kNm

$M_{2B} = - 3.91$ kNm

Minimum moment $= 82.5 \times (.0273 - .0065/2)$

$= 1.98$ kNm

6. $$f_{bcx} = \frac{3.91 \times 10^6 \times 27.3}{1.24 \times 10^6} = 86 \text{ MPa}$$

7. $C_{mx} = 1.0$

8. $F_{bcx} = 172$ MPa

9. Combined stresses Rule 8.2.1(a)

$$\frac{65}{130} + \frac{86}{\left(1 - \frac{65}{.6 \times 879}\right) 172}$$

$= 0.50 + 0.57 = 1.07 > 1.0$

10. Combined stresses Rule 8.2.1(b)

Clearly not critical

11. Outstand stress $= 65 - 235$

$= - 170 < .66 \times 260 = 172$ MPa O.K.

Max. safe load $\doteqdot \frac{82.5}{1.07} = 77.1$ kN (opposite sides)

PART C: British method

$$\frac{\ell}{r_v} = \frac{0.85 \times 1500}{20.1} = 63.4$$

$$F_{ac} = 129 \text{ MPa}$$

$$P_{safe} = 163 \text{ kN}$$ (same sides or opposite sides)

PART D:

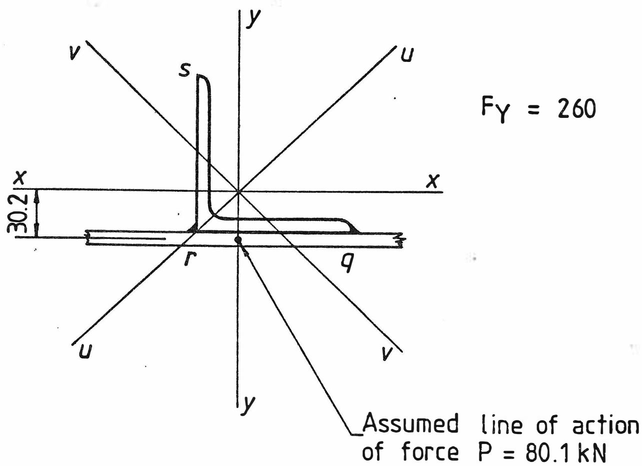


FIGURE 19 : Design Example 4, Part D

$$f_{ac} = \frac{80.1 \times 10^3}{1260} = 64 \text{ MPa}$$

$$\frac{\ell}{r_{min}} = \frac{\ell}{r_v} = \frac{.85 \times 1500}{20.1} = 63$$

$$\therefore F_{ac} = 129 \text{ MPa}; \quad F_{ocv} = 500 \text{ MPa}$$

$$\frac{\ell}{r_u} = \frac{.85 \times 1500}{39.6} = 32 \rightarrow F_{ocu} = 1960 \text{ MPa}$$

$$M_u = M_v = 0.0302 \cos(45) \times 80.1$$
$$= 1.71 \text{ kNm}$$

$$\therefore f_{bsu} = -\frac{1.71 \times 10^6}{27.5 \times 10^3} = -62 \text{ MPa} \quad \text{(tension - ive)}$$

$$f_{bqu} = 62 \text{ MPa}$$

$$f_{bsv} = f_{bqv} = -\frac{1.71 \times 10^6}{0.51 \times 10^6}\left[102 \times \cos(45) - \frac{27.3}{\cos(45)}\right]$$
$$= -112 \text{ MPa}$$

$$f_{brv} = \frac{1.71 \times 10^6}{0.51 \times 10^6} \times \frac{27.3}{\cos(45)} = 129 \text{ MPa}$$

Combined stresses Rule 8.2.1(a)

$$F_{bcu} = F_{bcv} = 0.66 \times 260$$
$$= 172 \text{ MPa}$$

Point r (Heel)

$$\frac{64}{129} + \frac{1.0 \times 129}{\left(1 - \frac{64}{.6 \times 500}\right) 172}$$

$$= 0.50 + 0.95 = 1.45 > 1.0 \text{ No good}$$

Point q

f_q = 64 - 112 + 62 = 14 MPa clearly not critical

Point s (Outstand)

f_s = 64 - 112 - 62 = - 110 MPa clearly not critical

The combined stress ratio at the heel is significantly greater than unity. The allowable capacity of the strut using this method would be approximately 55 kN (≑ 80.1/1.45).

7. CONCLUSIONS

(1) Surprisingly little experimental work on single angle trusses has been reported. From the information which is available, the predominant mode of deformation of single angle web compression members is in a plane perpendicular to the plane of the truss.

(2) The current British method, which ignores eccentricity, does not appear to give an adequate factor of safety for stocky struts and may even be unsafe when web members are connected on opposite sides of chords.

(3) The American approach and the current Australian approach consider eccentricity with respect to both principal axes which does not generally reflect the observed behaviour of web compression members described in Conclusion (1) above. The safe loads obtained by these approaches are shown to be unnecessarily conservative as a result.

(4) The conventional method of balancing welds does not account for the fact that the principal axes of the member are inclined to the frame axes. When principal axes are considered, the line of action of the axial force is nearer to the mid-point of the connected leg than to the y axis. However, there is some advantage in the balanced weld distribution for carrying out of plane moments because the heel weld is more effective than the toe weld for these moments.

(5) Single angle struts in trusses whose web members are connected alternately on opposite sides have less theoretical capacity than the same struts in a one sided truss as is evident from Design Example 4. Connecting web members on opposite sides is suitable only for welded joints, but it does simplify joint detailing. This advantage is offset by the need to turn the truss over during fabrication.

When this is considered with the reduced member capacities, there does not seem to be any economical justification for designing a truss with web members on opposite sides in preference to a one sided truss.

(6) The recommended design method is more complex than past Australian methods, but is certainly no more difficult than the biaxial bending approach required by the current Australian code, and considerably less conservative.

(7) Further experimental work is required to confirm the proposed design method, particularly for unequal angles.

8. ACKNOWLEDGEMENT

The authors gratefully acknowledge the assistance given by Mr J.M. Leigh, Senior Research Officer, B.H.P. Research Laboratory, Melbourne.

APPENDIX A - NOMENCLATURE

Symbol	Meaning
A_T, B_T, C_T, D_T	stiffness functions (see Equations 9 and 10)
B	length of legs of angle
c_x	perpendicular distance from the centroid to the contact face of the gusset
C_{mx}	coefficient which allows for distribution of bending moment in compression member
e	eccentricity of centroid of web member with mid-plane of gusset plate
e_c, e_t	eccentricities (see Figure 7)
e_r	nett eccentricity (see Figure 7)
E	Young's modulus of elasticity
f_{ac}	average axial compressive stress
f_{bcx}	maximum compressive bending stress
f_{bu}, fbv	bending stresses about u and v axes
F_{ac}	maximum permissible axial compressive stress
F_{bc}	maximum permissible compressive bending stress
F_{bu}, F_{bv}, F_{bx}	maximum permissible bending stresses about u, v and x axes
F_{oc}	elastic buckling stress of a compression member
F_{ob}	elastic buckling stress of a beam
F_Y	yield stress
I_x, I_y, I_{xy}	second moments and product moments of area about the x- and y-axes
I_u, I_v	second moments of area about the major and minor principal axes
J	St. Venant torsional constant
ℓ	effective length
L	length of compression member between intersection points
M_{cu}	elastic critical moment about u axis
M_{Tx}, M_{Bx}	out of plane couples at the top and bottom chords about the chord x-axis
M_{Tz}, M_{Bz}	out of plane couples at the top and bottom chords about the chord longitudinal axis

Symbol	Meaning
M_{2T}, M_{2B}	end moments at the top and bottom of strut 2
M_u, M_v	bending moments about the major and minor principal axes
P	axial load
P_{ult}	ultimate load capacity of the strut
P_Y	squash load
r, r_x, r_v	radii of gyration
t	thickness of leg of angle
t_G	thickness of gusset plate
x_p	distance along the mid-plane of the gusset from the heel to the point of application of load (see Figure 6)
α	stiffness coefficient
β	end moment ratio
γ	angle at the intersection of the web member and the chord
θ_{Tx}, θ_{Tz}	rotation at joint II about the chord x- and z-axes (see Figure 12b)

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