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Krishna Kishore Sankisa
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COMPRESSIVE RESISTANCE AND 
BLOCK SHEAR STRENGTH OF ANGLES

by

KRISHNA KISHORE SANKISA

A Thesis
submitted to the
Faculty of Graduate Studies and Research
through the Department of
Civil and Environmental Engineering
in Partial Fulfillment of the Requirements for
the Degree of Master of Applied Science at
the University of Windsor

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ABSTRACT

This investigation deals with two distinct topics: compressive resistance of 60° angles and block shear strength of 90° angles.

Angles have been used as leg, diagonal and horizontal members in lattice towers for a long time. Lattice towers with triangular base (as compared to rectangular base towers) are more economical but the leg members have to be bent inwards to 60° (either by cold forming or by schifferlerizing) to enable the connections between the bracing members and leg members without the necessity to use bent gusset plates. The results of 19 cold-formed 60° angles and 19 hot-rolled schifferlerized angles tested under concentric axial compression are presented. For the finite element analysis, a commercially available software "ABAQUS" has been used for non-linear solution using a Newtonian approach with eight node shell elements. The FEM solutions are in satisfactory agreement with the
experimental failure loads. Suitable effective width has been suggested for cold-formed 60° angles and for schifferized angles.

Eccentric tension is one of the most common types of loading for bracing members of lattice towers. Lately, block shear mode of failure of angle members has drawn the attention of the designers. The results of tests on 63 single angle members under eccentric tension of different sizes (with bolt holes located either on or outside the centre line of connected leg of the angles) are presented. The results are in agreement with Clause 13.2 of CAN/CSA-S16.1-M89.

The effect of pre-tensioning of bolts on tensile strength was also investigated. It was found that there is a 5% to 15% increase in the failure loads of angle specimens with pre-tensioned bolts as compared to specimens with snug tight bolts.
to my parents
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NOTATION

The following symbols are used in the thesis:

\( A_g \) = gross cross-sectional area

\( A_n \) = net cross-sectional area

\( A_{ne} \) = effective net cross-sectional area

\( A'_{ne} \) = reduced effective net cross-sectional area

\( a \) = length of unbent portion of the Schifflerized Angle (Fig. 1.2b)

\( b \) = length of bent portion of the Schifflerized Angle (Fig. 1.2b)

\( c \) = fillet radius of Schifflerized angle

\( C_r \) = compressive resistance of 60° angles

\( C_w \) = warping Constant

\( E \) = modulus of elasticity

\( F_u \) = Tensile strength

\( F_y \) = Yield stress of Steel

\( F_y' \) = effective yield stress

\( G \) = shear modulus

\( g \) = gauge distance
$I_u =$ moment of inertia about major (u-u) axis

$I_v =$ moment of inertia about minor (v-v) axis

$I_{pc} =$ polar moment of inertia of 60° angles about centroid

$I_{ps} =$ polar moment of inertia of 60° angles about shear centre

$J =$ Saint-Venant's torsion constant

$K_u =$ effective length factor of 60° angles about u-u axis

$K_v =$ effective length factor of 60° angles about v-v axis

$K_r =$ effective length factor of 60° angle for torsional buckling

$L =$ length of the column

$p =$ pitch

$P_u =$ Euler's buckling load about u-u axis

$P_v =$ Euler's buckling load about v-v axis

$R =$ outer bend radius of 60° cold-formed angle

$r_u =$ radius of gyration about u-u axis

$r_v =$ radius of gyration about v-v axis

$t, T =$ thickness of the leg of the angle member

$u_c =$ distance of the centroid of the 60° angle member from the heel (Fig.1.2a and 1.2b)

$u_s =$ distance of the shear centre from centroid of the 60° angle member (Fig.1.2a and 1.2b)
$w =$ flat width of the $60^\circ$ angle for use in width-to-thickness ratio

$\lambda =$ slenderness parameter

$\Phi =$ resistance factor
CHAPTER 1

INTRODUCTION

1.1 General

One of the most commonly used members in structural engineering are steel angles. Angles are used both as compression and tension members. These members play a very important role in resisting forces. Angles are widely used in latticed communication and electrical transmission towers.

1.2 Angles as compression members

One of the ways the communication and electrical transmission towers are constructed is to have one leg member at each vertex of an equilateral triangle (Fig.1.1) braced by diagonal and horizontal members. The two leg plates of each of the three angle leg members in lattice
towers with triangular plan are bent to 60° in order to have a connection at the vertices of the equilateral triangle without the use of bent gusset plates. This is achieved either by the use of cold-formed 60° angles (Fig.1.2a) or by schifflerizing hot rolled 90° angles. Schifflerization is the process in which each of the two leg plates of the angle member are bent inwards by 15° so as to make the included angle between the leg plates 60° (Fig.1.2b). This can be done by rerolling (Fig.1.3) or brake-pressing regular hot-rolled 90° angles. When compared to 90° angle, the moment of inertia about the minor axis (v-v axis) increases and moment of inertia about the major axis (u-u axis) decreases when the angle is schifflerized. This causes the schifflerized angle to be strong in flexural buckling (about the minor axis) but weak in torsional-flexural buckling (about the major axis).

Although compression tests have been conducted on regular 90° angles for a long time there was not much study on cold-formed 60° angles or schifflerized angles.
The only published work on the compressive strength of schifflerized angles was done at the University of Windsor (Adluri 1990) in which 18 specimens were tested with ideal pinned-end conditions. Of the eighteen specimens, only nine failed in torsional-flexural buckling. Also, in actual practice, the angles are bolted to other members and the capacity of schifflerized angles could be different from tests based on ideal pinned-end conditions. Therefore, it was felt necessary to conduct tests with fixed end conditions to get an upper bound to the compressive resistance of schifflerized angles.

Since cold-formed 60° angles are being increasingly used in short (up to 30 m height) guyed towers, there is also a need to study the behaviour of cold-formed 60° angles.

From earlier research on schifflerized angles (Fig. 1.2b), the flat width recommended in width-to-thickness ratio calculation was $W = b$ (unbent portion) for use in CAN/CSA-S37-M86. Since industries have their own standards for bent and unbent portions, a suitable width
for schifflerized angles which does not vary from fabricator to fabricator is required.

Cold-formed 60° angles do not have a fillet and are torsionally much weaker than the corresponding schifflerized angles. Flat width to be used in the w/t ratio computation of cold-formed 60° angles is so far not defined in any of the specifications. Therefore it is one of the objectives of the investigation to define the flat width 'w' to be used in calculating the compressive resistance according to different specifications.

1.3 Angles as tension members

Angle bracing members in latticed towers are usually connected to other members at the joints by one leg with other leg outstanding. These members are subjected to eccentric tension (Fig.1.4) or compression. The forces between the members are transmitted either through a direct connection between the members or through a gusset plate. The connections could be made using welds or bolts. The most popular fasteners used currently in
communication and electrical transmission tower industry are high strength bolts. In the case of bolted connections, the load at service conditions is transmitted to the member from the connection or vice-versa through the friction or bearing between bolts and the member. At failure, the loads are transmitted through bearing between the bolt shank and part of inner surface of bolt hole.

Until recently, design of single angles in eccentric tension was being carried out by computing the product of a typical contribution of the area of the cross-section and the material strength. Usually, this would mean checking for two cases, viz. the yielding of gross section or rupture of net section (with or without the effect of shear lag) as shown in Fig.1.5a. Only rarely is the design based on equations for biaxial bending and tension. This has been the practice for nearly a century before the mid 1980s. In recent years however, this practice has come under increasing scrutiny by researchers, utilities, and code writers. As a result of this, the block shear effect has been identified as a new and dominant mode of
failure in many practical cases. This has led to the revision of several North American design specifications and manuals to incorporate this mode of failure in routine design calculations. The block shear failure may be defined as failure along a section at least a part of which is parallel to the load and is hence assumed to be in shear (Fig. 1.5 b & c). The remaining part of the section will be in direct tension (perpendicular to the direction of the load application in the case of the angle members). The failure path in block shear mode generally follows the centres of the bolt holes farthest from the free corner of the connected leg. The present investigation is concerned with the design of angles for block shear failure. Angles in eccentric tension can also fail by edge (Fig. 1.6a), end (Fig. 1.6b) or combined (Fig. 1.6c) mode.

As mentioned earlier, block shear was not taken into account till recently while calculating the tensile resistance of members. CAN/CSA-S16.1 considered this type of failure in its 1989 edition based on investigations on
beams and gusset plates. Canadian Standards Association Technical Committee on Antenna Towers (S37) suggested that tests be conducted on angles to confirm the block shear failure for angles in latticed towers.

1.4 Objectives

The following are the objectives of the study:

1.4.1 Compressive Resistance:

(a) To carry out compression tests on different sizes of 60° angles (both cold-formed and schifferlerized angles) failing in torsional-flexural buckling.

(b) To compare the values obtained from finite element analysis with experimental results.


(d) To suggest a suitable flat width for schifferlerized angles and cold-formed angles for use in the
calculation of their compressive resistance.

1.4.2 Block Shear Strength:

(a) To determine block shear strength by conducting tension tests on eccentrically loaded angles with one or two bolts at the ends.

(b) To study the effect of pre-tensioning of the bolts on block shear strength of angle members, since the bolts in communication towers are pre-tensioned, while those in electrical transmission towers are usually snug tight.

1.5 Arrangement of the thesis

Chapter 2 deals with literature review consisting of five sections. Of these sections 2.1 to 2.3 deal with compressive resistance of 60° angles while sections 2.4 and 2.5 deal with block shear strength of 90° angles.

Chapter 3 deals with Finite Element Analysis of 60° angles.

Chapter 4 deals with Experimental Investigation of 60°
angles under compression and 90° angles under eccentric tension. Sections 4.1 to 4.3 deal with compressive resistance of 60° angles. Sections 4.4 and 4.5 deal with block shear strength of tension members. Section 4.6 deals with the effect of pre-tensioning of bolts.

Chapter 5 is about discussion of results. Sections 5.1 to 5.5 is about properties and results of 60° angles. Sections 5.6 and 5.7 deal with the mechanical properties and results of tension members.

Conclusions from the present investigation and recommendations for further research are given in Chapter 6.
CHAPTER 2

LITERATURE REVIEW

2.1 General

2.1.1 Concentric load : Elastic buckling

Concentrically loaded equal leg 60° angles (which are singly symmetric) fail either by:

(i) flexural buckling about the minor axis (v-v axis); or by

(ii) flexural buckling about the major axis (u-u axis) and simultaneous twisting about the centre of rotation, which lies on the major axis close to the shear centre (Fig.1.2a and 1.2b)

Euler flexural buckling load about minor axis is
Critical torsional-flexural buckling load \( P_{tf} \) can be obtained by solving the following quadratic equation (Timoshenko and Gere, 1961, equation 5.39)

\[
\frac{(I_u+I_v)}{(I_u+I_v+Au^2)}P_f^2-(P_u+P_f)P_f+P_uP_f=0
\]  \hspace{1cm} (2.2)

where,

\[
P_u = \frac{\pi^2EI_u}{(K_uL)^2}
\]  \hspace{1cm} (2.3)

\[
P_f = \frac{\pi^2E(A_Tr)}{(K_fL)^2}
\]  \hspace{1cm} (2.4)

The quadratic equation (2.2) provides two values for \( P_{tf} \), one of which is smaller and the other larger than \( P_u \) and \( P_t \). The smaller of the two values is the torsional-flexural buckling load.
2.1.2 Concentric Loading : Inelastic Buckling

In case of inelastic flexural buckling, the failure capacity can be estimated by replacing $E$ by the tangent modulus $E_t$ in the formula for elastic flexural buckling (Eq. 2.1). For torsional and torsional-flexural buckling also $E$ can be replaced by $E_t$, however there is no consensus on the value to be used for the shear modulus in the inelastic range. For the sake of simplicity, Bleich suggested the use of $G_t$, the tangent shear modulus, in place of $G$. This approach leads to conservative results and also makes it possible to use readily the equations of elastic torsional and torsional-flexural buckling for inelastic buckling by replacing $E$ by $E_t$ and $G$ by $G_t$.

Classical formulae are only applicable to ideal columns. For columns having very high $kL/r$ ratios, Euler's formula is applicable. Real columns have imperfections (residual stresses, out-of-straightness etc.) and the classical inelastic formulae cannot be applied.
2.2 Compression Tests on 60° Angles

To the best of the author’s knowledge no published literature was found on compressive tests on cold-formed angles.

The only published work found on concentrically loaded schifferlerized angles was carried out at the University of Windsor in 1989-90 (Adluri, 1990). A total of 18 schifferlerized angles were tested. The suggested formula for the flat width W for use in CAN/CSA-S37-M86 is the width of the bent portion of the schifferlerized angle (Dimension b of Fig 1.2). Out of the 18 specimens nine failed in torsional-flexural buckling and nine failed in flexural buckling. Five sizes of angles were tested viz. 127x127x8 mm, 102x102x6.4 mm, 89x89x8 mm, 76x76x3.2 mm, 76x76x6.4 mm, of 300 and 400 MPa nominal yield strength with slenderness ratios varying between 50 and 95.

Adluri, Madugula and Monforton (1990) conducted a finite element analysis on the compressive resistance of schifferlerized angles. The results of the finite element
analysis were compared to the experimental results. The angles were modelled using eight-node shell elements with six degrees of freedom at each node. A non-linear analysis was conducted. In this case, buckling is indicated by excessive deformations and consequent reduction in the load carrying capacity.

Adluri and Madugula (1991a) presented formulae for computing the properties of schifflerized angles. The geometric properties of the schifflerized angles were calculated by idealizing the cross-section into rectangular segments. The properties given were the cross-sectional area, moment of inertia, centroidal distance, distance between shear centre and the centroid, radius of gyration, Saint-Venant’s torsion constant, and warping constant.

Adluri and Madugula (1991 b,c) have presented tables which provide the design axial compressive strength of schifflerized angles according to AISC-LRFD(1986) and according to CAN/CSA-S37-M86. The tables demarcate between the flexural and torsional-flexural buckling
loads.

Adluri, Madugula, and Monforton (1992) published a paper regarding the proper selection of width to be used in the width-thickness ratio as there is ambiguity regarding the width to be considered in width-thickness ratio in ASCE Manual No.52.

2.3 Design Specifications

As there is no specified effective width $W$ for cold-formed 60° angles, the following various effective widths were tried to find an optimum one for various specifications (Fig.1.2a):

(i) $W = B$

(ii) $W = B + (R - 0.5 T)\pi/6$

(iii) $W = B + (R - 0.5 T)\pi/3$

For schifferizered angles the width tried was $W = a + b - t - c$ (Fig.1.2b) for CAN/CSA-S37-M86, AISC-LRFD and BS5950:Part 1:1985. For ASCE Manual No.52 (ANSI/ASCE 10-90) the width tried is $W = a + b$.

The following are some of design specifications for
computing compressive resistance $C_r$ of $60^\circ$ angles:

2.3.1 CAN/CSA-S37-M86

Maximum w/t ratio = 25

Effective yield stress $F_y'$

(1) When

$$\frac{w}{t} \leq \frac{200}{\sqrt{F_y}}; \quad F_y' = F_y$$

(2.5)

(2) When

$$\frac{200}{\sqrt{F_y}} < \frac{w}{t} \leq \frac{380}{\sqrt{F_y}}; \quad F_y' = F_y \left[ 1.677 - 0.677 \left( \frac{w}{200} \right) \right]$$

(2.6)

(3) When

$$\frac{380}{\sqrt{F_y}} < \frac{w}{t} \leq 25; \quad F_y' = \frac{56415}{\left( \frac{w}{t} \right)^2}$$

(2.7)
where, $F_y$ is the yield stress in MPa.

Factored axial compressive resistance - $C_r$

\begin{align*}
0 < \lambda \leq 0.15, \quad C_r &= \Phi A_x F_y' \\
0.15 < \lambda \leq 1.0, \quad C_r &= \Phi A_x F_y' (1.035 - 0.202 \lambda - 0.222 \lambda^2) \\
1.0 < \lambda \leq 2.0, \quad C_r &= \Phi A_x F_y' (-0.111 + 0.636 \lambda^{-1} + 0.087 \lambda^{-2}) \\
2.0 < \lambda \leq 3.6, \quad C_r &= \Phi A_x F_y' (0.009 + 0.877 \lambda^{-2}) \\
3.6 < \lambda, \quad C_r &= \Phi A_x F_y' \lambda^{-2} = \Phi A_x \left[ \frac{1970000}{(KL)^2} \right]
\end{align*}

where,

\begin{equation}
\lambda = \frac{KL}{r_v \sqrt{\frac{F_y'}{\pi^2 E}}}
\end{equation}

As can be seen, CAN/CSA-S37-M86 does not consider torsional-flexural buckling explicitly as a mode of failure. To overcome this, effective yield stress $F_y'$ is
used in place of $F_y$ to reduce the compression resistance of the member with large width-to-thickness ratios.

2.3.2 ANSI/ASCE 10-90 and ASCE Manual No.52

The ultimate compressive strength $P_{ul}$ of axially loaded compression members shall be:

$$P_{ul} = A_t \left[ 1 - \frac{1}{2} \left( \frac{KL}{r_v} \right)^2 \right] \text{ when } \frac{KL}{r_v} \geq C_c$$ (2.14)

$$P_{ul} = \frac{286000}{(KL/r_v)^2} \text{ when } \frac{KL}{r_v} \geq C_c$$ (2.15)

where

$$C_c = \pi \left( \frac{2E}{F_y} \right)^{1/2}$$ (2.16)

2.3.2.1 Maximum w/t Ratio

The ratio $w/t$, where $w=$ flat width and $t=$ thickness of the leg, shall not exceed 25. If $w/t$ exceeds $(w/t)_{lim}$ given by
\[ \frac{w}{t}_{\text{llm}} = \frac{80}{\sqrt{F_y}} \]  

\hspace{1cm} (2.17)

then the ultimate strength \( P_{\text{ult}} \) shall be computed with \( F_y \) replaced in the above eqs 2.14 and 2.16 with \( F_y' \) given by

\[ F_y' = [1.677 - 0.677 \frac{w}{t}] F_y \text{ when } (\frac{w}{t})_{\text{llm}} \leq \frac{w}{t} \leq \frac{144}{\sqrt{F_y}} \]  

\hspace{1cm} (2.18)

\[ F_y' = \frac{9500}{(\frac{w}{t})^2} \text{ when } \frac{w}{t} > \frac{144}{\sqrt{F_y}} \]  

\hspace{1cm} (2.19)

where \( F_y \) is the yield stress in ksi.

2.3.3 AISC-LRFD (1986)

\[ C_r = 0.85 A_g F_{cr} \]  

\hspace{1cm} (2.20)

For flexural buckling \( F_{cr} \) is given in section 2.3.3.1 and for torsional-flexural buckling in section 2.3.3.2.
2.3.3.1 Flexural Buckling

For $\lambda_c \sqrt{Q} \leq 1.5$,

$$F_{cr} = (0.658 Q_{1/2}) F_y Q$$

(2.21)

For $\lambda_c \sqrt{Q} > 1.5$,

$$F_{cr} = \frac{0.877}{\lambda_c^2} F_y$$

(2.22)

where,

$$\lambda_c = \frac{K L}{r x} \sqrt{\frac{F_y}{E}}$$

(2.23)

The values of factor $Q$ which depend on width-thickness ratios are given in section 2.3.3.3.

2.3.3.2 Torsional-Flexural Buckling

The nominal critical stress $F_{cr}$ is determined as follows:

a. For $\lambda_c \sqrt{Q} \leq 1.5$: 

...
\[ F_{cr} = Q(0.658 \Omega t^2) F_y \]  \hspace{1cm} (2.24)

b. For \( \lambda_e \sqrt{Q} > 1.5 \):

\[ F_{cr} = \left[ \frac{0.877}{\lambda_e^2} \right] F_y \] \hspace{1cm} (2.25)

where,

\[ \lambda_e = \frac{\sqrt{F_y}}{F_e} \] \hspace{1cm} (2.26)

\( F_y \) = specified minimum yield stress of steel

\( F_e \) = critical torsional elastic buckling stress

2.3.3.3 Reduction Factor Q

When \( w/t \leq 76/\sqrt{F_y} \), (where \( F_y \) is in ksi.),

\[ Q = 1.0 \] \hspace{1cm} (2.27)

When \( 76/\sqrt{F_y} < w/t < 155/\sqrt{F_y} \);
\[ Q = 1.340 - 0.00447(w/t)\sqrt{F_y} \quad (2.28) \]

When \( w/t \geq 155/F_y \):

\[ Q = 15500/[F_y(w/t)^2] \quad (2.29) \]

2.3.4 BS 5950 : Part 1:1985

The compressive resistance \( C_r \) of an angle section is given by

\[ C_r = A_t F_{cr} \quad (2.30) \]

When \( w/t \leq 11.5 \sqrt{275/F_y} \),

\( F_{cr} \) is obtained from Perry strut formula.

\[ F_{cr} = \frac{F_E F_y}{F + \sqrt{F^2 - F_E F_y}} \quad (2.31) \]

where,

\[ F = \frac{F_y + (\eta + 1)F_E}{2} \quad (2.32) \]
where, \( F_E \) is Euler strength; \( F_y \) is the design strength; \( \eta \) is the Perry factor.

The Perry factor \( \eta \) for flexural buckling under load should be obtained from:

\[
\eta = 0.0055(\lambda - \lambda_0) \geq 0
\]  
(2.33)

where the limiting slenderness ratio,

\[
\lambda_0 = 0.2 \sqrt{\frac{\pi^2 E}{F_y}}
\]  
(2.34)

When \( w/t \) exceeds the limiting value, a reduction factor given by the lesser of

\[
\frac{11}{a+b} - \frac{4}{t\sqrt{\frac{275}{F_y}}}
\]  
(2.35)

and
2(a+b) - 4
\[ \frac{19}{2} \]
\[ \frac{275}{F_y} \]

is applied to yield stress (a and b as per Fig.1.2).

2.4 Tension Members

A member (without any holes) when subjected to concentric axial tension, attains its strength when all the fibres of the cross-section have yielded (uniform tensile stress). The strength in tension \( T_u \) is given by

\[ T_u = F_y A_g \]

where,

\( A_g = \) gross cross-sectional area

If a tension member has holes (for rivets and bolts etc.), the gross area should be reduced. This reduced area is referred to as net area. In this case the tensile stress distribution is not uniform, but the stress reaches a constant value \( F_y \), as the fibres reach yield strain.
The strength in tension of the members with holes is given by

\[ T_u = F_u A_n \]

where \( F_u \) = Tensile strength of the material

\( A_n \) = Net area of the cross-section

However it is possible that the gross area may yield before the net section reaches tensile strength. This means that the gross area yielding \((F_y A_g)\) could control the design of the member.

Concentric axial load in tension members can seldom be obtained in actual structures: either the connections may not be concentric or the member itself may not be straight, resulting in eccentric loading.

In case bending takes place about some plane other than one through a principal axis then, stress \( f \) at any point is given by

\[ f = \frac{P}{A_g} \pm \frac{M_u v}{I_u} \pm \frac{M_v u}{I_v} \]

where \( M_u \) and \( M_v \) are components of moment about \( u \) and \( v \) principal axes respectively, \( I_u \) and \( I_v \) are the respective moments of inertia about these axes, and \( u, v \) are the coordinates of any point.
2.5 Tensile tests on angles

One of earliest tests on angles in tension was carried out by Batho(1915). During the experiments on single angle members, it was found that the effect of end constraints was insignificant.

Young(1935) after a study suggested that for a single angle connected by one leg the following factor 'e' be applied to the net section area:

\[ e = 1.0 - 0.18\left(\frac{u}{c}\right) \]

in which 'u' and 'c' are the widths of the unconnected and connected legs, respectively.

Nelson(1953) carried out tension tests on eighteen angles. He noted that the length of the specimen or the end attachment had no significant effect.

In 1969, Marsh developed a theoretical expression for the net effective area of an angle connected by one leg subjected to a tensile force. Theoretically obtained results were compared with some of the test results on aluminium angles; good agreement was observed for angles connected by the longer leg, whereas the prediction was
optimistic for angles connected by the short leg.

Kennedy and Sinclair (1969) tested 721 single angle connections in tension using a single 5/8 in. (16 mm) bolt to determine the relationship between end and edge distances, mode of failure and the ultimate load of the connections. The conclusions made were:

1. Failure through either the end or edge is a distinct function of end and edge distance and can be predicted.

2. For end type failures the ultimate load $P$ (kips) can be reasonably predicted by

$$P = 5 t \sigma_y \left(2.011 x + 0.279\right)/8$$

where

$x = \text{end distance in inches,}$

$t = \text{thickness of the angle in inches.}$

$\sigma_y = \text{yield stress in ksi}$

3. For edge type failure the ultimate load $P$ (kips) can be reasonably predicted by

$$P = 5 t \sigma_y \left(4.024 y - 0.941\right)/8$$

where
\[ y = \text{the edge distance in inches}. \]

4. Failure in bearing occurs at a nominal bearing stress equal to approximately 4.5 times the yield stress.

5. Bearing stress equal to 2.25 times the yield stress can produce insignificant hole elongation depending upon end and edge distances. However such bearing stress cannot be developed with an end distance less than one inch (25.4 mm) or edge distance less than 5/8 in. (16 mm).

6. The development of local stresses in the immediate neighbourhood of the hole, equal to or greater than the yield stress, is not a reliable indication of approaching failure of the connections.

Hardash and Bjorhovde (1985) demonstrated that all segments in a fracture path which may consist of a series of segments, some loaded primarily in tension and others primarily in shear, fracture simultaneously.

Madugula and Mohan (1989) presented results of tests
conducted by Western Area Power Administration (WAPA) and Bonneville Power Administration (BPA) separately on various 90° angles. WAPA conducted tests on 29 specimens. Twelve specimens failed at loads less than the various code specifications; this is due to failure of specimens in block shear (which was not considered with the earlier specifications) rather than net section failure. The net area taken by various specifications is different. Most of the specifications take the effective area of the angles in tension connected by one leg to be less than the actual net area. The conclusions of the paper are given below:

(1) In the case of unequal leg angles in tension connected by one leg, distinction should be made between failure loads of angles connected by long leg and by short leg. Failure loads are higher if long leg is connected.

(2) For certain combinations of edge distance, end distance, and pitch, block shear mode of failure may govern instead of net section failure.

(3) The specification give unduly conservative values
for failure loads of angles in eccentric tension connected by close tolerance bolts (fitted bolts).

Epstein (1992) conducted tests on 38 different connections for various types of failure. There were 3 specimens, each consisting of a pair of angles, tested for each connection. The angle sizes tested were 152x152x8 mm, 152x102x8 mm, 152x89x8 mm, 127x127x8 mm, 127x89x8 mm, and 127x76x8 mm. Epstein also studied the effect of staggering of bolts. The connection consisted of a pair of angles connected by two rows of 19 mm (3/4 in.) bolts, each row having two bolts. The edge distance for all connections was 38 mm (1.5 in.) and the pitch was 76 mm (3 in.). The failures of the angle members were classified into five different categories:

(a) block shear,
(b) predominantly block shear with some net section,
(c) predominantly net section with some block shear,
(d) net section, and
(e) boltshear plus block shear.

Failure of equal leg angles in short connections were
all by block shear. As the length of the connection increased, the failure mode was tending to be predominantly through net section. Failures for longest connection were all through net section.

2.6 Specifications for Members Under Tension

2.6.1 CAN/CSA-S16.1-M89

According to Clause 12.3.1 the effective net area shall be calculated as follows:

(a) for a segment normal to the force

\[ A_{n1} = W_n T \]  \hspace{1cm} (2.37)

\( W_n \) = Width of the member under tension
\( T \) = Thickness of the member

(b) For a segment parallel to the force

\[ A_{n2} = 0.6L_n T \]  \hspace{1cm} (2.38)

\( L_n \) = Length of the member under shear

The factored tensile resistance, \( T_r \), developed by a member subjected to an axial tension force shall be taken as the least of
(i) \( T_{r1} = \phi A_x F_y \) \hspace{1cm} (2.39)

(ii) \( T_{r2} = 0.85 \phi A_{ne} F_u \) \hspace{1cm} (2.40)

(iii) \( T_{r3} = 0.85 \phi A'_{ne} F_u \) \hspace{1cm} (2.41)

where
\[
A_{ne} = A_{n1} + A_{n2}
\]

In the above equation \( A_{ne}' = 0.75 A_{ne} \) for angles having one or two bolts.

The angles can also fail by bearing. Bearing resistance is given by

\[
B_r = \phi T_{ne} F_u \hspace{1cm} (2.42)
\]

where,

\( A_{ne} = \) Net cross-sectional area

\( A_g = \) Gross cross-sectional area
\[ F_y = \text{Yield stress} \]
\[ F_u = \text{Tensile strength} \]
\[ e = \text{end distance}. \]

2.6.2 ANSI/ASCE 10-90 and ASCE Manual No.52

In case of ASCE Manual No.52 block shear failure is to be taken into account only if the centroid of the bolt pattern is not located between the heel of the angle and the centre line of the connected leg. However according to ANSI/ASCE-10-90 block shear failure is to be taken into account if the centroid of the bolt pattern on the connected leg is outside the centre of gravity of the angle. In the majority of cases the centroid of the bolt hole is outside the centre of gravity of the member, hence according to ANSI/ASCE 10-90 it is almost always necessary to consider block shear mode of failure for members in tension.
CHAPTER 3

FINITE ELEMENT ANALYSIS

3.1 Introduction

Finite element analysis was conducted and the results compared with the experimental results. If there is good agreement between the two, compression resistance of any size and length angle can be predicted by a finite element analysis without the need to carry out physical tests. This chapter deals with the finite element analysis of 60° angles.

Even though cold-formed 60° angles and schifflerized angles could be considered simple members, it is difficult to determine their compressive strength. A commercial finite element package "ABAQUS" has been used to analyze the members.
Non-linearity of both material and geometry was considered in the analysis of both cold formed and schifferlized angles. The variation of residual stresses along the leg width and through the thickness was assumed similar to ECCS recommendations (ECCS 1985) as shown in Fig.3.3 and Fig.3.4.

3.2 Finite Element Analysis

First the data regarding the member known as model data was created. This consists of input for nodes, elements, boundary conditions, element properties, and material properties.

After the data regarding the model was created, data regarding the loading was given. A non-linear analysis was carried out for a static case of loading. The loading was applied in several increments. At each increment, non-linear equations were solved using Newtonian techniques.

A series of linear problems were solved to obtain non-linear solution. Many techniques such as Matthies and
Strang (1979) are available to tackle non-linear problems. Presently Newtonian techniques are widely being used. For any displacement configuration the nodal displacements were represented by the vector \( \{U\} \) and \( \{F_e\} \) and \( \{F_i\} \) represent vectors of external nodal loads and internal resisting forces respectively. \([K]\) represents the tangent stiffness.

The vector of the unbalanced portion of the nodal forces is given by

\[
\{F_u\} = \{F_e\} + \{F_i\}
\]

where \( \{F_u\} \) gives the error in the solution.

The iterative solution according to Newtonian techniques is as follows:

\[
\{F_u\} = \{F_e\} + \{F_i\}
\]

\[
\Delta r^j = [K^j]^{-1} F_u^j
\]

\[
r^{j+1} = r^j + \Delta r^j
\]

\[
F_i^{j+1} = \text{function of } r^{j+1}
\]

### 3.3 Modelling of the test setup

The first step in modelling of a 60° angle (Fig. 4.1)
was to generate nodes along the cross-section of the angle. The cross-section was divided into sixteen parts i.e., having seventeen nodes along the cross-section. This set of nodes were copied to form a new set of nodes such that the distance between the two sets of nodes was the length of the 60° angle. Nodes were generated between these two sets of nodes. The number of rows of nodes generated was eleven (Appendix A).

The next step was to generate elements along the cross-section. Eight-node shell elements were used to model the 60° angle. Eight elements were generated along the cross-section and five such rows of elements were generated along the length of the 60° angle (Fig. 3.1). When shell elements are used, the elements are generated along the centre plane of the member and the thickness is provided to the member using shell section options.

Next the nodes of the base plate (Fig. 4.1) were developed. Five nodes each were generated along the length and the width of the plate, consisting of 25 nodes in total. Four-node shell elements were used to develop
the elements of the base plate. The base plate consisted of 16 elements. This set of nodes and elements were copied to generate a new set of nodes and elements representing the top base plate. The nodes and elements of the base plate represent the centre plane of the base plate. The distance between two base plates represents the thickness of the plate plus the length of the test specimen. The model was generated such that the centroid of the base plate passes through the centroid 60° angle.

Solid steel blocks (Fig.4.1) supporting the angle were modelled as trapezoidal blocks. Four-node shell elements were used to generate the elements along the centre plane of the blocks. The solid blocks were in contact with the 60° angle.

Next the lateral supporting 90° angles (Fig.4.1) were modelled using four-node shell elements. These lateral supporting angles were in contact with the 60° angle.

As mentioned earlier when shell elements are used, centre plane elements of the members are generated. Therefore the thicknesses of the members are provided.
using shell properties. The complete set up is shown in Fig. 4.1.

The boundary condition option was used to fix the base plates. Five nodes of the each base plate (four corner nodes and centre node) were fixed. The bottom base plate was free to move in the direction of the application of the load.

For the nodes of the base plates, solid blocks, and lateral supporting angles which are in contact with the 60° angle, multi-point constraints were used so that displacements and rotations were the same for all the nodes at the point of contact. Multi-point constraint was also used to fix the solid blocks and supporting angles to the base plates. Next the material properties of the members were described. The elastic and plastic properties of the 60° angles were the modulus of elasticity and the yield stress.

A Fortran program was written for the residual stresses in hot-rolled schifflerized angle.

While conducting the experiments the loads are
invariably eccentric. This eccentricity can be achieved in different ways. In this investigation a load of magnitude 1/1000th of the expected failure load was applied at mid height at the heel in the direction of the minor axis for torsional-flexural buckling. This method gave better results when compared to others.

The model was loaded through the centroid of the 60° angle.

The model of the member is shown in Fig.3.1 and Fig.3.2. The ABAQUS input is shown in Appendix A, data check run is given in the Appendix B and the last page of the output is given in Appendix C.

From the input in Appendix A, the expected failure load is applied in increments. For example for the 4x4x1/4 in. (102x102x6.4 mm) angle of length 1520 mm the expected failure load was 350 kN. This expected failure load was applied in increments on the specimen. Form the last page of the output in Appendix C the total time completed is 0.725. Therefore the failure load was calculated as 0.725 times 350 kN i.e., 254 kN.
CHAPTER 4

EXPERIMENTAL INVESTIGATION

4.1 General

The sizes of 60° angles tested for compression are 38x38x3.2 mm (1.5x1.5x1/8 in.), 51x51x4.8 mm (2x2x3/16 in.), 76x76x4.8 mm (3x3x3/16 in.), 76x76x6.4 mm (3x3x1/4 in.) and 102x102x6.4 mm (4x4x1/4 in.) nominal dimensions. Out of these 38x38x3.2 mm and 51x51x4.8mm angles were cold-formed, while 76x76x4.8 mm, 76x76x6.4 mm and 102x102x6.4 mm angles were hot-rolled. The column l/r ratios varied from 40 to 89. A total of 38 specimens were tested, 19 each of cold-formed and hot-rolled angles.

4.2 Fabrication of Test Specimens

Different slenderness ratios for each specimen were
included in the test program. The lengths of the specimens varied from 370 mm to 2140 mm depending on the size of the 60° angle. The cold-formed specimens were marked with starting letter "C" (for example C1#1) and hot-rolled specimens are marked with starting letter "H" (for example H1#1).

The following are the different lengths included in the investigation:

- **38x38x3.2 mm Cold-formed 60° angles**
  - C1 : Length of the specimen of group C1 900 mm
  - C2 : Length of the specimen of group C2 630 mm
  - C3 : Length of the specimen of group C3 370 mm

- **51x51x4.8 mm Cold-formed 60° angles**
  - C4 : Length of the specimen of group C4 1350 mm
  - C5 : Length of the specimen of group C5 874 mm
  - C6 : Length of the specimen of group C6 404 mm

- **76x76x6.4 mm Schifflerized angle**
  - H1 : Length of the specimen of group H1 1220 mm

- **76x76x4.8 mm Schifflerized angle**
  - H2 : Length of the specimen of group H2 1870 mm
H3 : Length of the specimen of group H3 1530 mm
H4 : Length of the specimen of group H4 732 mm
102x102x6.4 mm Schifferlerized angle
H5 : Length of the specimen of group H5 2140 mm
H6 : Length of the specimen of group H6 1960 mm
H7 : Length of the specimen of group H7 1520 mm
Third number denotes the number of the specimen for that
group. The ends of the members were milled to be
perfectly parallel to each other and perpendicular to the
longitudinal axis.

4.3 Testing of Compression Members

Fixed end conditions were created by using assembly
of plates and blocks. A solid block and two 90° angles of
size 63x63x8 mm and length 100mm were used to create fixed
eend conditions at each end. A circular plate of diameter
330 mm and thickness 40 mm was fastened to the Gilmore
platen (Fig.4.2). A 445 kN capacity hydraulic jack was
then placed on the circular plate. On top of the
hydraulic jack was placed a square base plate (280x280x24
mm) with a circular collar (Fig. 4.3). This circular collar was used to centre and support the square base plate on top of the hydraulic jack. The solid block was bolted to the base plate such that it directly passed through the centroid of the base plate which coincided with the centroid of the angle specimen. Separate solid blocks were used depending on the size of the 60° angle (Fig. 4.4). The 60° angle was then placed in contact with the solid block such that the centroid of the angle specimen passed through the centroid of the base plate (Fig. 4.5). Two 90° angles (lateral supports) which can slide over the base plate were brought in contact with the 60° angle specimen and bolted to the base plate. A similar arrangement of solid block and 90° angles was also made at the top. The top base plate was connected to the load cell. The base plate was supported and positioned through the centre line of the load cell by a collar screw (Fig. 4.3). The load cell was in turn connected to the circular plate of diameter 330 mm and thickness 40 mm. This circular plate was connected to top Gilmore platen.
Fig. 4.1 shows the drawing of the set-up and Fig. 4.8 shows a 60° angle specimen under test. A 445 kN hydraulic jack was used to apply compressive loadings. A 900 kN universal load cell was connected to a precalibrated load indicator. The specimens were initially loaded in 11 kN increments. As the applied load approached failure limit, the load increment was reduced to approximately 5 kN increments. In all the specimens tested failure occurred suddenly. A loud noise accompanied the failure. The specimens either failed by twisting at the mid height or twisting at the top. The reason of the twisting at the top could be due to the fact that specimens failed while trying to twist the bottom plate assembly which sits on the jack. In such cases there could be a possibility of some rotation about longitudinal axis taking place at the bottom end at failure. During testing, it was observed that there was slight rotation about u-u and v-v axes of the circular collar resting on the top of the hydraulic jack.

Fig. 4.9 and Fig. 4.10 show cold-formed 38x38x3.2 mm (1.5x1.5x1/8 in.) angle of length 900 mm (specimen Cl#1)
and 102x102x6.4 mm (4x4x1/4 in.) schifflerized angle of length 1520mm (specimen H7#1) after failure respectively. For the purpose of comparison, an untested specimen from the same group was placed beside the failed specimen in both the photographs. All the specimens tested after failure are shown in Fig.4.11.

SSRC(1988) guidelines for column testing recommend loading by a mechanical pump so as to facilitate constant rate of loading. However in the present case loading was done using a flat jack operated using a hand pump. The load cell was calibrated twice during test program and the average of both was taken to calculate test failure loads.

The end plate assembly described was found to be adequate for torsional-flexural buckling. The solid blocks and support angles provided enough support. The solid blocks also prevented against accidental kicking-out of the member.

4.4 Testing of Tension Members

The sizes of 90° angles tested in tension were
51x38x4.8 mm (2x1.5x3/16 in.), 51x51x6.4 mm (2x2x1/4 in.),
51x51x4.8 mm (2x2x3/16 in.), 63x51x4.8 mm (2.5x2x3/16
in.), 63x51x6.4 mm (2.5x2x1/4 in.), 63x63x4.8 mm
(2.5x2.5x3/16 in.), 63x63x6.4 mm (2.5x2.5x1/4 in.),
63x63x8.0 mm (2.5x2.5x5/16 in.), 76x76x4.8 mm (3x3x3/16
in.), 76x76x6.4 mm (3x3x1/4 in.), 89x89x6.4 mm
(3.5x3.5x1/4 in.), and 89x89x8.0 mm (3.5x3.5x5/16 in.)
nominal dimensions.

Three specimens were tested for each size. To
accommodate the specimens in the test equipment available
the specimens were of length 11 1/4 in. for single bolt
connection or 15 1/4 in. for double bolts connection.
The specimens were fabricated in an actual tower
fabricating facility so as to resemble the field
conditions. The holes were punched at gauges normally
used by the industry. The size of the hole punched was
for 5/8 in. (16 mm) bolts. Out of 63 specimens tested 45
were single bolt connection and 18 were double bolt
connections. The end distance was 29 mm (1 1/8 in.) for
specimens connected by single bolt and in case of double
bolts end distance was 25.4 mm (1 in.), the pitch was 50.8 mm (2 in.). The edge distance varied from 29 mm (1 1/8 in.) to 44 mm (1 3/4 in.). Two bolts were used where it was felt that the single bolt would fail before the failure of the member. Table 5.11 and 5.13 give the properties of the angle specimens connected by single and double bolts respectively.

4.5 Tension Test setup

The end fixtures consisted of bar of size 100x100x25 mm. To this bar, two bars of sizes 108x100x25 and 108x48x25 mm were welded 10 mm apart. On these two bars, two 18.2 mm size holes were drilled. To the bar of size 100x100x25 mm a rod of diameter 36 mm and length of 210 mm was welded. This rod was fastened to the Tinius Olsen testing machine. Fig.4.12,4.13 and 4.14 show the isometric, front and top view of the end fixture giving the details of the dimensions. Fig.4.15a shows the photo of the end fixture attached to the testing machine. The angle specimen to be tested was placed in between the two
bars which are 10 mm apart. The angle specimen was held in position with help of 5/8 in. (16 mm) size bolts at each end and tensile force was applied (Fig. 4.15b).

The failure occurred in one of the following three ways:

(1) Edge mode

(2) End mode (bearing failure)

(3) Combined mode (block shear failure)

For many of the members, considerable elongation of the bolt hole was observed after the test.

In case of edge failure (Fig 4.16) splitting of the member started at the hole and progressed to the toe of the member. On further application of load, specimens failed by combined mode. In case of end failure (Fig. 4.17) material piled up in front of the hole or the end tore out. The combined mode (Fig. 4.18) (block shear failure) is one in which both edge and end failure occurred.
4.5 Tests on the Effect of Pre-tensioning of Bolts

The effect of pre-tensioning of bolts on the strength was also investigated. Seven tests consisting of 14 specimens were conducted.

For studying the effect of pre-tensioning, the specimens were chosen from a stock of cold-formed and hot-rolled 90° angles. The cold-formed angles were of size 65x65x4 mm and 55x35x3 mm while the hot-rolled angles were of size 63x63x6.4 mm (2 1/2x2 1/2x1/4 in.). The specimens were cut using a band saw. Holes were drilled in the specimens for 19 mm bolts. These were loaded in double shear. The cold-formed angle of size 65x65x4 mm and hot-rolled angle of size 2 1/2x2 1/2x1/4 in. (63x63x6.4 mm) had both the end and edge distance equal to 28 mm. Two sets of specimens were tested for each size. One set for each size were snug tight while the other set had been pretensioned. The total elongation of the joints at either end was measured using dial gauges. This movement is also representative of the movements of the bolts. The specimens were mounted on to the test setup which
consisted of a 25 mm (1 in.) top gusset plate attached to a test frame. A similar bottom gusset plate was attached to the load cell mounted on a 900 kN (200 kip) hydraulic pump (Fig.4.19). One angle was placed on either side of the 25 mm gusset plate. The load was applied in gradual increments and the elongations had been noted. As expected all the specimens exhibited hole elongations. The failed specimens are shown in Fig.4.20 to Fig.4.23. The mode of failure of the specimens was by block shear. Figure 4.24 shows the load-elongation curves for 65x65x4 mm cold-formed 90° angles with snug tight and pre-tensioned bolts while Fig.4.25 shows similar curves for 63x63x6.4 mm hot-rolled 90° angles.

For both compression and tension specimens tension tests were conducted on the standard size coupons. The coupons were taken from the same stock as that of specimens. The stress versus strain curve was plotted on a X-Y electronic recorder. Stress-strain curves for tensile coupons are shown in Figs. 4.26 and 4.27.
CHAPTER 5

DISCUSSION OF RESULTS

5.1 General

This chapter deals with the results of the experimental investigation regarding the compressive resistance of the $60^\circ$ angles and the block shear strength of $90^\circ$ angles under eccentric tension. This chapter also compares the results obtained from finite element modelling of the $60^\circ$ angles. The finite element analysis results and calculated values according to various specifications are compared with the experimental results, thus helping to understand the results and interpret their implication.

5.2 Properties

5.2.1 Geometric Properties of Cold-formed $60^\circ$ Angles

The cold-formed angles are manufactured by bending a
plate which has a width equal to twice the nominal width of the angle leg.

The geometric properties of cold formed angles are derived and are given below:

\[ N = \text{one half of the length of the curved line } xyz \text{ in Fig. } 1.2a \]

\[ R_1 = R - 0.5 T \]

\[ B = N - 0.5 T\frac{\pi}{3} - R_1\frac{\pi}{3} \]

\[ A_x = \text{cross-sectional area} = 1.047T(2.0R_1) + 2.0BT \quad (5.1) \]

\[ u_c = \frac{0.866B^2 + B(R_1 + T) + 0.362R_1^2 + 1.05T + R_1}{2B + 2.1R_1} \quad (5.2) \]

\[ I_u = 0.614TR^3 + \frac{B^3T}{24} + \frac{BT^3}{8} + \frac{BT(3.46R_1 + B)^2}{8} \quad (5.3) \]

\[ I_v = \frac{B^3T}{8} + \frac{BT^3}{24} + \frac{BT}{8}(1.73B + 2R_1 + 2T)^2 \]

\[ + 0.0478TR_1^3 + 2.1TR_1(0.173R_1 + 0.5T)^2 - A_xu_c^2 \quad (5.4) \]
\[ R_u = \sqrt{\frac{I_u}{A_s}} \]  

(5.5)

\[ R_v = \sqrt{\frac{I_v}{A_s}} \]  

(5.6)

\[ J = A_s T^2 / 3 \]

\[ u_z = \frac{TR_1}{6I_u} (2B^3 + 8.34B^2R_1 + 10.9BR_1^2 + 4.11R_1^3) - R_1 - T/2 + U_c \]

\[ I_p = I_u + I_v + Au_r^2 \]

5.2.2 Geometric Properties of Schifflerized angles

The geometric properties of hot-rolled schifflerized angles taken from technical note "Geometric Properties of Schifflerized angles" by Adluri and Madugula (1991a) are given below. The cross-section is idealized into segments.
of four rectangles by ignoring the toe and filet radii.

\( A_g = \text{cross-sectional area} = 2t(a+b-t/2) \)

d=leg width= a+b

\[
u_c = \frac{2(a-t/2)^2 + 4b(a-t/2) + \sqrt{6}b^2}{4\sqrt{2}(a+b-t/2)} + \frac{t}{\sqrt{2}} \quad (5.7)
\]

\[I_u = 2(I_{u1} + I_{u2}) \]

where,

\[
I_{u1} = \frac{bt^3}{16} + \frac{b^3t}{48} + bt\left[\frac{b}{4} + \frac{(a-t/2)}{\sqrt{2}}\right]^2 \quad (5.8)
\]

and

\[I_v = 2(I_{v1} + I_{v2}) - A_r\left(u_c - t/\sqrt{2}\right)^2 \quad (5.9)\]

where,

\[
I_{v1} = \frac{bt^3}{48} + \frac{b^3t}{16} + bt\left[\frac{(a-t/2)}{\sqrt{2}} + \sqrt{3}\frac{b}{4}\right]^2 \quad (5.10)
\]

and
\[ I_{v2} = \frac{(a-t/2)t^3}{24} + \frac{t(a-t/2)^3}{6} \]  

(5.11)

\[ I_{pc} = I_u + I_v \]  

(5.12)

\[ I_{ps} = I_{pc} + A_g u_s^2 \]  

(5.13)

\[ u_s = \frac{(\sqrt{3}-1)(3c^2b^2 + \sqrt{2}cb^3)}{4c^3 + 12c^2b + 6\sqrt{2}cb^2 + 2b^3} - \frac{t}{\sqrt{2}} + u_c \]  

(5.14)

where \( c = a - t/2 \)

\[ r_u = \sqrt{\frac{I_u}{A_g}} \]  

(5.15)

\[ r_v = \sqrt{\frac{I_v}{A_g}} \]  

(5.16)

\[ J = \frac{2}{3}(a+b-t/2)t^3 \]  

(5.17)

The properties used in computing member strengths are...
listed in Tables 5.1a to 5.4c. Appendix D gives the Fortran programs used to calculate the properties.

5.2.3 Mechanical Properties

The mechanical properties of the material tested were taken from the results obtained from the tensile coupons (as per ASTM standards). The sample stress-strain curves for the coupons are presented in Fig. 4.26 and 4.27. The actual yield stress values are found to be 0.33% to 26.33% higher than the corresponding nominal values. The value of Young's Modulus is taken as 200 GPa as per established practice. The yield stress of the tensile coupons of 60° angles is given in Table 5.5.

5.3 Comparison of Experimental & Finite Element Loads

The finite element results of the members have been given in Table 5.6. Out of the several specimens tested for each length, the geometrical properties of one were selected arbitrarily and a finite element analysis was carried out on this specimen. For most of the specimens
FEM results were within 10% of the experimental results. The reason for large angle specimens having low value could be due to the end fixtures not able to provide full fixity. In the finite element analysis the support angles might have yielded. The finite element analysis gave satisfactory results when compared to the experimental load values.

5.4 Comparison of Experimental Loads with Specifications

Appendix E gives the Fortran programs used to calculate the compressive resistance using various specifications.

5.4.1 CAN/CSA-S37-M86

CAN/CSA-S37 refers to clause 13.3.1 of S16.1 to calculate the member capacities. As mentioned in chapter 2, three different widths were used for cold formed 60° angles to find a suitable width to be used in calculation. These are shown as Case-I, Case-II, Case-III in Table 5.7a. Since rotation might have taken place at the bottom
support, for each case the \( k \) has been taken as 0.7. For 38x38x3.2 mm cold-formed angles the values obtained for Case-III are closer to the experimental values. In case of 51x51x4.8 mm cold-formed angles width-thickness ratio was below \( 200/\sqrt{F_y} \). Therefore \( F_y' = F_y \) for all the cases.

For schifflerized angles, width used for calculating failure loads is \( W = b + a - t - c \) (Fig.1.2b) (shown as Case-I in table 5.7b). In case of schifflerized angles the lateral supports are not sufficient to provide full fixity.

The value of \( \Phi \) was taken as 1. Table 5.7a and 5.7b compares the values with the experimental results for cold-formed and schifflerized angles respectively.

5.4.2 ANSI/ASCE 10-90 and ASCE Manual No.52

The Structural Stability Research Council formula for ultimate strength of the concentrically loaded columns in the inelastic range was adopted to calculate the capacities of the members.

For cold-formed angles as mentioned earlier three
different widths in width-thickness ratios were chosen to determine the compressive resistance. These are shown as Case-I, Case-II, Case-III in Table 5.8a. For each case the values are shown for $k=0.7$. The values obtained for cold formed angles using $W=B+(R-0.5\ T)\pi/3$ gave values very close to the experimental failure loads. In case of schifferlerized angles the width used was $W=a+b$ (Fig.1.2b). The column strength calculated from the formulae for $K=0.7$ are given in the Table 5.8b. For schifferlerized angles also the results obtained with the above mentioned width gave values which are in satisfactory agreement with the experimental results.

5.4.3 AISC-LRFD (1986)

Calculation of the compressive resistance for concentrically loaded columns according to AISC load and resistance factor design is based on a modified approach compared to the allowable stress design (ASD). The ASD is based on SSRC column curve No.2. AISC takes in to account the torsional-flexural buckling which was not the case.
with CAN-CSA-S37.

The width assumed for cold-formed angles is the same as mentioned in the above two specifications. These are shown as Case-I, Case-II and Case-III in Table 5.9a. In case of cold-formed members with $w = B + (R - 0.5 T)\pi/3$, the results from the formulae for $K=0.7$ were very close to the experimental values.

For Schifflerized angles the width taken is $W = a + b - t - c$ (Fig. 1.2b). The values obtained are in very good agreement with the experimental results.

The value of $\Phi$ taken was 1.0. The column capacities were calculated from the formulae for $K=0.7$. Table 5.9a and 5.9b compare the experimental results and the results obtained from AISC load and resistance factor design specifications for cold-formed and Schifflerized angles respectively.

5.4.4 BS 5950: Part1 :1985

According to British practice, Perry-Robertson formulae were used for calculating the critical stresses.
for different slenderness ratio and yield stresses.

Three different widths were assumed for cold-formed angles to find a suitable width for calculation. These are shown as Case-I, Case-II and Case-III in Table 5.10a. For cold-formed angles for \( W=B \) (Fig.1.2a) the experimental results were very close to the experimental results obtained for \( K=0.7 \).

In case of schifflerized angles the width assumed is \( W=a+b-t-c \) (Fig.1.2b). The calculated values are in satisfactory agreement with experimental results.

The experimental results and values obtained from the specification for values of \( K=0.7 \) are given in Table 5.10a and 5.10b for cold-formed and schifflerized angles respectively.

5.5 Torsional-flexural Buckling

Clause 6.2.5.1 of CAN/CSA-S37-M86 does not consider torsional-flexural buckling as a mode of failure. To overcome this phenomena, effective yield stress is taken into consideration so as to reduce the compressive
resistance of the member.

It is necessary to take into account torsional-flexural buckling to calculate the compressive resistance. The suggested modification by Sankisa, Adluri and Madugula (1993) is to replace slenderness parameter $\lambda$ by $\lambda_{tr}$

\[
\lambda_{tr} = \sqrt{\frac{F_y'}{F_y}}
\]

where $F_y'$ is given by eq. (2.5) to (2.7) depending on w/t ratios.

\[
F_y' = \frac{F_u + F_t}{2H} \left(1 - \sqrt{1 - \frac{4F_u F_t H}{(F_u + F_t)^2}}\right)
\]

Euler buckling stress about major axis

\[
F_u = \frac{\pi^2 E}{(KL/r_u)^2}
\]

Torsional elastic buckling stress
Table 5.7a and 5.7b show the values calculated by taking into account torsional-flexural buckling for cold-formed and schifflerized angles respectively. Case-IV in Table 5.7a shows the value calculated for cold-formed angles for \( k=0.7 \). Case-II in Table 5.7b shows the values obtained for schifflerized angles. The values calculated by accounting for torsional-flexural buckling for both cold-formed and schifflerized angles were less than the experimental values. For cold-formed, \( W=B+(R-0.5T)\pi/3 \) was used in calculating the values.

Sankisa, Adluri, Madugula (1993) presented the above results obtained at SSRC conference held at Milwaukee.

5.6 Mechanical Properties of Angles Under Tension

The value of the Young’s modulus is taken as 200 000
MPa as per established practice. Tables 5.12 and 5.14 list the yield and tensile strength of the members under tension.

5.7 Tension Members

5.7.1 Comparison of Experimental Loads with CAN/CSA-S16.1

The member capacities were calculated according to the CAN/CSA-S16.1-M89. Except for size 2 1/2x2 1/2x1/4 in. angle specimens (Mark No. 29#1, #2 and #3) for a gauge distance of 1 1/4 in. the experimental values were higher than those calculated from the specifications. The failure loads calculated are according to the gross area yielding, net section fracture with shear lag, block shear failure and bearing failure. TR1 in Tables 5.15 and 5.16 is the failure load calculated according to gross area yielding. TR2 in the Tables 5.15 and 5.16 is the failure load calculated according to block shear failure. TR3 in the Tables 5.15 and 5.16 is the failure load calculated according to net section failure with shear lag. BR in the tables is the failure load calculated according to...
bearing (end mode) failure. MIN in the above mentioned tables is the minimum failure load obtained from the mentioned failure modes. \( P_{\text{exp}} \) is the experimental failure load.

### 5.7.2 ANSI/ASCE 10-90 and ASCE Manual No.52

ASCE Manual No. 52 mentions that block shear should be taken into account as failure mode only if centre of the bolt holes is outside the centre line of the connected leg. According to ANSI/ASCE 10-90, block shear mode of failure is to be taken in to account if the centroid of the bolt hole is outside the centre of gravity of the member.

Tables 5.15 and 5.16 give the failure loads. Thirty-six specimens under eccentric tension failed in end failure mode, twenty-seven failed by block shear (combined mode of edge and end failure).

The results from the above tests were presented as technical report to Canadian Standards Association Technical committee on Antenna, Towers and Antenna

5.7.3 Effect of Pre-tensioning of Bolts

The experimental results of the effect of pre-tensioning of the bolts are discussed here. The increase in load between the case of the bolt pre-tensioning and snug tight is about 5.12% to 15.12%. Table 5.17 presents the results of the tests. Typical load elongation curves are shown in Fig.4.24 and 4.25. Curves for pre-tensioned and snug tight cases show essentially similar trends. The results could be interpreted as an indication to the possibility of a slight increase in the load capacity for angles with pre-tensioned bolts. This increase can be attributed to the friction due to some residual tensile force in the bolt at the time of failure. This might explain the seemingly constant difference in load capacity for all the seven tests.

Results obtained from this investigation were presented as technical report to Canadian Standards Association Committee on Antenna, Towers and Antenna
CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

Tests on 19 cold-formed 60° and 19 schifflerized angles under axial compression and 63 tests on 90° angles under eccentric tension were conducted.

From the observations and results obtained from the investigation, the following conclusions are made:

6.2 Compressive Resistance

1. The FEM results of the 60° angles obtained are in satisfactory agreement with the experimental results.

2. Based on a "flat width" used for cold-formed and schifflerized angles, the compressive strength calculated using various standard specifications for 60° angles is
summarized below:

(a) For calculating the compressive resistance according to CAN/CSA-S37-M86 the flat width recommended for cold-formed angles is \( W = B + (R - 0.5 T) \pi /3 \) (Fig.1.2b) and for schifferlized angles is \( W = a + b - t - c \) (Fig.1.2a).

(b) The flat width suggested for ASCE Manual No.52 (ANSI/ASCE 10-90) for compressive capacity for cold-formed 60° angles is \( W = B + (R - 0.5 T) \pi /3 \) (Fig.1.2a). In case of schifferlized angles \( W = a + b \) (Fig.1.2b) is recommended.

(c) For calculating the compressive resistance according to AISC-LRFD-1986 for cold-formed 60° angles, flat width recommended is \( W = B + (R - 0.5 T) \pi /3 \) (Fig.1.2b). For schifferlized angles the suggested flat width is \( W = a + b - t - c \).

(d) For BS 5950: Part 1: 1985, the flat width to be used for calculating the compressive resistance for cold-formed 60° angles is \( W = B \) (Fig.1.2a) and for schifferlized angles, \( W = a + b - t - c \).

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6.3 Block shear Strength:

(a) The experimental results are in good agreement with the values predicted by CAN/CSA-S16.1-M89.

(b) When the gauge line is outside the centre of gravity of the member, block shear should be taken into account. Therefore the Clause 5.10.2 in ANSI/ASCE 10-90 is more appropriate.

(c) Special attention should be given for end mode of failure for angles connected by single or double bolts.

(d) The cold-formed 90° angles exhibited a 10% to 15% increase in the load capacity for the case of pre-tensioned bolts when compared to the case of snug-tight bolts. Hot-rolled 90° angles showed 5% increase in ultimate load of members with pre-tensioned bolts relative to that of members with snug tight bolts. Difference in results between cold-formed and hot-rolled angles is due to the difference in plastic flow under the head.
6.4 Recommendations for Further Research

From the experience obtained from the present investigation, the following issues can be pursued for further research:

Compressive Resistance:
1. Extensive tests should be conducted on 60° angles with bolted connections.
2. The actual residual stress pattern should be determined for the 60° angles.

Block shear Strength:
1. In case of tension members extensive experiments should be conducted with varying end distance and using more bolts to have a better understanding of various types of failures.
2. Due to machine limitations, short specimens were used in the present investigation. Longer specimens should be tested to represent realistically members used in actual structures.
3. In case of pre-tensioning of bolts, the present study is limited in scope and hence is not completely
conclusive. It is necessary to conduct more extensive research.
REFERENCES


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Table 5.1a Properties of Cold-formed 60° Angle  
Nominal size 38x38x3.2 mm

<table>
<thead>
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<th>PROPERTY</th>
<th>C1#1</th>
<th>C1#2</th>
<th>C1#3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flat Leg</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Width 'b' (mm)</td>
<td>27.48</td>
<td>27.36</td>
<td>27.24</td>
</tr>
<tr>
<td>Radius (mm)</td>
<td>13.0</td>
<td>13.0</td>
<td>13.0</td>
</tr>
<tr>
<td>Thickness (mm)</td>
<td>3.18</td>
<td>3.16</td>
<td>3.06</td>
</tr>
<tr>
<td>Area (mm$^2$)</td>
<td>250.8</td>
<td>248.5</td>
<td>240.2</td>
</tr>
<tr>
<td>$u_e$ (mm)</td>
<td>14.40</td>
<td>14.40</td>
<td>14.30</td>
</tr>
<tr>
<td>$I_{xx}$ ($10^6$ mm$^4$)</td>
<td>0.023</td>
<td>0.022</td>
<td>0.021</td>
</tr>
<tr>
<td>$I_{yy}$ ($10^6$ mm$^4$)</td>
<td>0.056</td>
<td>0.055</td>
<td>0.054</td>
</tr>
<tr>
<td>$I_{xy}$ ($10^6$ mm$^4$)</td>
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<td>0.092</td>
<td>0.089</td>
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<tr>
<td>$r_x$ (mm)</td>
<td>15.0</td>
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<td>14.9</td>
</tr>
<tr>
<td>$r_y$ (mm)</td>
<td>9.5</td>
<td>9.5</td>
<td>9.5</td>
</tr>
<tr>
<td>$J$ ($10^6$ mm$^4$)</td>
<td>0.0845</td>
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<td>0.0750</td>
</tr>
<tr>
<td>Length (mm)</td>
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**Note:** Refer to Fig. 1.2a for explanation of notations.
Table 5.1b Properties of Cold-formed 60° Angle  
Nominal size 38x38x3.2 mm

<table>
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<th>PROPERTY</th>
<th>C2#1</th>
<th>C2#2</th>
<th>C2#3</th>
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<tbody>
<tr>
<td>Flat Leg</td>
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</tr>
<tr>
<td>Width ‘b’ (mm)</td>
<td>26.38</td>
<td>26.90</td>
<td>26.30</td>
</tr>
<tr>
<td>Radius (mm)</td>
<td>13.0</td>
<td>13.0</td>
<td>13.0</td>
</tr>
<tr>
<td>Thickness (mm)</td>
<td>3.02</td>
<td>2.98</td>
<td>3.04</td>
</tr>
<tr>
<td>Area (mm²)</td>
<td>232.0</td>
<td>232.1</td>
<td>233.0</td>
</tr>
<tr>
<td>( u_e ) (mm)</td>
<td>13.90</td>
<td>14.10</td>
<td>13.90</td>
</tr>
<tr>
<td>( I_{x'x'} ) (10^6 \text{ mm}^4)</td>
<td>0.020</td>
<td>0.021</td>
<td>0.020</td>
</tr>
<tr>
<td>( I_{x'e} ) (10^6 \text{ mm}^4)</td>
<td>0.050</td>
<td>0.051</td>
<td>0.050</td>
</tr>
<tr>
<td>( I_{x'e} ) (10^6 \text{ mm}^4)</td>
<td>0.083</td>
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<tr>
<td>( r_e ) (mm)</td>
<td>14.70</td>
<td>14.80</td>
<td>14.70</td>
</tr>
<tr>
<td>( r_{e'} ) (mm)</td>
<td>9.30</td>
<td>9.40</td>
<td>9.30</td>
</tr>
<tr>
<td>( J ) (10^6 \text{ mm}^4)</td>
<td>0.0705</td>
<td>0.0687</td>
<td>0.0717</td>
</tr>
<tr>
<td>Length (mm)</td>
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Note: Refer to Fig. 1.2a for explanation of notations
Table 5.1c Properties of Cold-formed 60° Angle
Nominal size 38x38x3.2 mm

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<th>C3#3</th>
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</thead>
<tbody>
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<td>Flat Leg</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>Width 'b' (mm)</td>
<td>26.12</td>
<td>25.76</td>
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</tr>
<tr>
<td>Radius (mm)</td>
<td>15.0</td>
<td>15.0</td>
<td>15.0</td>
</tr>
<tr>
<td>Thickness (mm)</td>
<td>3.00</td>
<td>2.94</td>
<td>3.06</td>
</tr>
<tr>
<td>Area (mm²)</td>
<td>228.9</td>
<td>222.4</td>
<td>235.5</td>
</tr>
<tr>
<td>$u_c$ (mm)</td>
<td>13.80</td>
<td>13.60</td>
<td>14.00</td>
</tr>
<tr>
<td>$I_{min}$ ($10^6$ mm⁴)</td>
<td>0.019</td>
<td>0.019</td>
<td>0.020</td>
</tr>
<tr>
<td>$I_{max}$ ($10^6$ mm⁴)</td>
<td>0.049</td>
<td>0.047</td>
<td>0.051</td>
</tr>
<tr>
<td>$I_{ps}$ ($10^6$ mm⁴)</td>
<td>0.081</td>
<td>0.078</td>
<td>0.084</td>
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<tr>
<td>$r_a$ (mm)</td>
<td>14.60</td>
<td>14.50</td>
<td>14.70</td>
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<tr>
<td>$r_v$ (mm)</td>
<td>9.20</td>
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<td>$J$ ($10^4$ mm⁴)</td>
<td>0.0687</td>
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<td>Length (mm)</td>
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Note: Refer to Fig. 1.2a for explanation of notations.
### Table 5.2a Properties of Cold-formed 60° Angle  
Nominal size 51x51x4.8 mm

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<td>Width 'b' (mm)</td>
<td>39.58</td>
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<td>15.0</td>
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<td>Thickness (mm)</td>
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<td>$I_{axi}$ (10⁶ mm⁴)</td>
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<td>$I_{Mas}$ (10⁶ mm⁴)</td>
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<td>$I_{ps}$ (10⁶ mm⁴)</td>
<td>0.264</td>
<td>0.298</td>
<td>0.282</td>
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<td>$r_a$ (mm)</td>
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<td>19.10</td>
<td>18.90</td>
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<tr>
<td>$r_v$ (mm)</td>
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Note: Refer to Fig. 1.2a for explanation of notations.
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<td>(u_c) (mm)</td>
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<td>20.20</td>
<td>20.50</td>
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<td>(I_{min}) (10^6 mm^4)</td>
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<td>0.079</td>
<td>0.083</td>
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<tr>
<td>(I_{max}) (10^6 mm^4)</td>
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<td>(I_{xx}) (10^6 mm^4)</td>
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<td>0.292</td>
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<td>(r_c) (mm)</td>
<td>19.10</td>
<td>19.00</td>
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<tr>
<td>(r_s) (mm)</td>
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<td>12.60</td>
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<td>(J) (10^4 mm^4)</td>
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Note: Refer to Fig. 1.2a for explanation of notations
Table 5.2c Properties of Cold-formed 60° Angle
Nominal size 51x51x4.8 mm

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<tr>
<td>Thickness (mm)</td>
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<td>4.62</td>
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<tr>
<td>Area (mm²)</td>
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<td>$u_c$ (mm)</td>
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<td>20.00</td>
<td>19.90</td>
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<tr>
<td>$I_{min}$ ($10^6$ mm⁴)</td>
<td>0.073</td>
<td>0.071</td>
<td>0.078</td>
<td>0.075</td>
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<td>$I_{max}$ ($10^6$ mm⁴)</td>
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<td>0.164</td>
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<td>$I_{ps}$ ($10^6$ mm⁴)</td>
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<td>0.266</td>
<td>0.289</td>
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<td>$r_x$ (mm)</td>
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<td>$r_y$ (mm)</td>
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Note: Refer to Fig. 1.2a for explanation of notations
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<td>b (mm)</td>
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<td>50.88</td>
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<td>t (mm)</td>
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<td>6.54</td>
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<td>c (mm)</td>
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<td>8.0</td>
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<tr>
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<td>32.90</td>
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<td>0.561</td>
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<td>Iₚₑₘ (10⁶ mm⁴)</td>
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<td>Iₚₑₚ (10⁶ mm⁴)</td>
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<td>rᵥ (mm)</td>
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<td>J (10⁶ mm⁴)</td>
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Note: Refer to Fig. 1.2b for explanation of notations.
### Table 5.3b Properties of Schifflerized Angles
Nominal size 76x76x4.8 mm

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<td>b (mm)</td>
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<td>t (mm)</td>
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<td>c (mm)</td>
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<td>Area (mm²)</td>
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<tr>
<td>Iₚₛ (10⁶ mm⁴)</td>
<td>0.395</td>
<td>0.391</td>
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<tr>
<td>Iₚₘₐₙ (10⁶ mm⁴)</td>
<td>0.230</td>
<td>0.228</td>
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<tr>
<td>Iₚₙ (10⁶ mm⁴)</td>
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<td>r₀ (mm)</td>
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<tr>
<td>rᵥ (mm)</td>
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<td>J (10⁴ mm⁴)</td>
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<td>Length (mm)</td>
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Note: Refer to Fig. 1.2b for explanation of notations
Table 5.3c Properties of Schifflerized Angles  
Nominal size 76x76x4.8 mm

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<td>a (mm)</td>
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<td>16.00</td>
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<td>b (mm)</td>
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<td>59.98</td>
<td>59.72</td>
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<tr>
<td>t (mm)</td>
<td>4.82</td>
<td>4.72</td>
<td>4.84</td>
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<tr>
<td>c (mm)</td>
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<td>8.0</td>
<td>8.0</td>
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<tr>
<td>Area (mm²)</td>
<td>711.3</td>
<td>695.0</td>
<td>709.5</td>
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<tr>
<td>$u_c$ (mm)</td>
<td>33.40</td>
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<tr>
<td>$I_{xx}$ ($10^6$ mm^4)</td>
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<td>$I_{xx}$ ($10^6$ mm^4)</td>
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<td>$I_{yy}$ ($10^6$ mm^4)</td>
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<td>$r_e$ (mm)</td>
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Note: Refer to Fig. 1.2b for explanation of notations
Table 5.3d Properties of Schifflerized Angles  
Nominal size 76x76x4.8 mm

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<td>16.00</td>
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<td>16.00</td>
</tr>
<tr>
<td>b (mm)</td>
<td>60.62</td>
<td>60.44</td>
<td>60.06</td>
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<tr>
<td>t (mm)</td>
<td>4.68</td>
<td>4.66</td>
<td>4.72</td>
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<tr>
<td>c (mm)</td>
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<td>8.0</td>
<td>8.0</td>
</tr>
<tr>
<td>Area (mm(^2))</td>
<td>695.3</td>
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<td>( u_c ) (mm)</td>
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<td>( I_{zz} ) (10(^6) mm(^4))</td>
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<td>( I_{xx} ) (10(^4) mm(^4))</td>
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<tr>
<td>( r_v ) (mm)</td>
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<tr>
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<td>Length (mm)</td>
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Note: Refer to Fig. 1.2b for explanation of notations.

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Table 5.4a Properties of Schifflerized Angles
Nominal size 102x102x6.4 mm

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<td>101.26</td>
<td>102.06</td>
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<td>24.17</td>
<td>24.17</td>
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<tr>
<td>b (mm)</td>
<td>77.25</td>
<td>77.09</td>
<td>77.89</td>
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<td>t (mm)</td>
<td>6.56</td>
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<td>6.52</td>
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<td>c (mm)</td>
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<td>10.0</td>
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<td>Area (mm²)</td>
<td>1287.6</td>
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<td>u, (mm)</td>
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<td>Iₐₐ (10⁴ mm⁴)</td>
<td>1.327</td>
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<td>Iₑₑ (10⁴ mm⁴)</td>
<td>0.744</td>
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<td>Iₚₚ (10⁴ mm⁴)</td>
<td>4.796</td>
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<td>rₑ (mm)</td>
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<td>rᵥ (mm)</td>
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<td>24.00</td>
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Note: Refer to Fig. 1.2b for explanation of notations
Table 5.4b Properties of Schifflerized Angles  
Nominal size 102x102x6.4 mm

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Note: Refer to Fig. 1.2b for explanation of notations.
Table 5.4c Properties of Schifferized Angles
Nominal size 102x102x6.4 mm

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<th>H7#3</th>
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<td>u_c (mm)</td>
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Note: Refer to Fig. 1.2b for explanation of notations.
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Table 5.6
Finite Element Analysis Failure Loads of 60° Angles

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Table 5.7a Failure Loads of Cold-formed 60° Angles According to CAN/CSA-S37

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<td>C5#3 130.31</td>
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<td>C6#3 149.81</td>
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<td>C6#4 145.32</td>
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Case-I: \( W = B \)
Case-II: \( W = B + (R - 0.5 \ T) R / 6 \)
Case-III: \( W = B + (R - 0.5 \ T) R / 3 \)
Case-IV: \( W = B + (R - 0.5 \ T) R / 3 \) (Torsional-Flexural Buckling)
Table 5.7b
Failure Loads of Schifflerized Angles
According to CAN/CSA-S37

<table>
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<th>P_{\text{wp}} (kN)</th>
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<tbody>
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Case-I = Flexural Buckling
Case-II = Torsional-Flexural Buckling

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## Table 5.8a Failure Loads of Cold-formed 60° Angles According to ASCE-Manual No.52

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Case-I: \( W = B \)
Case-II: \( W = B + (R - 0.5 T) \pi / 6 \)
Case-III: \( W = B + (R - 0.5 T) \pi / 3 \)

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<td>(kN)</td>
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Table 5.9a Failure Loads of Cold-formed 60° Angles According to AISC-LRFD

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Case-I: $W=B$
Case-II: $W=B+(R-0.5 T)\pi/6$
Case-III: $W=B+(R-0.5 T)\pi/3$
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Table 5.10a Failure Loads of Cold-formed $60^\circ$ Angles According to BS-5950 Part 1

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Case-I: $W=B$
Case-II: $W=B+(R-0.5 T)\pi/6$
Case-III: $W=B+(R-0.5 T)\pi/3$

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Table 5.11
Dimensions and Properties of 90° Angles with one bolt connection

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A = CONNECTED LEG  
B = UNCONNECTED LEG  
T = THICKNESS  
E = END DISTANCE  
D = EDGE DISTANCE  
Gauge = A - D

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.
Table 5.12
Yield and Tensile Strength of 90° Angles with one bolt connection

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Note: Refer Table 5.11 for Specimen Mark contd.
### Table 5.12
Yield Stress and Tensile Strength of 90° Angles with one bolt connection

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Note: Refer Table 5.11 for Specimen mark

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Table 5.13
Dimensions and Properties of 90° Angles with Two bolt Connection

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A = CONNECTED LEG
B = UNCONNECTED LEG
T = THICKNESS
E = END DISTANCE
D = EDGE DISTANCE
P = PITCH
Gauge = A - D
Table 5.14
Yield stress and Tensile Strength of 90° Angles with Two bolt Connection

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Note: Refer Table 5.13 for Specimen Mark
Table 5.15: Failure Loads of 90° Angles with one bolt connection

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Note: Refer Table 5.11 for Specimen Mark

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Table 5.15: Failure Loads of 90° Angles with one bolt connection

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<th>Size (in.)</th>
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Note: Refer Table 5.11 for Specimen mark

TR1 = A_y F_y
TR2 = 0.85 A_w F_v
TR3 = 0.85 A_w F_v
BR = T F_v E
MIN = Minimum load from the above formulae
Table 5.16 : Failure Loads of 90° Angles with two bolt connection

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Note: Refer to Table 5.13 for Specimen Mark

$TR1 = A_g F_t$
$TR2 = 0.85 A_n F_u$
$TR3 = 0.85 A_n F_u$
$BR = 2.0 T F_u E$
$MIN = Minimum from the above formulae$
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Fig. 1.1
Typical self-supporting antenna tower
U-U = Major axis
V-V = Minor axis
B = Width of flat portion
R = Outside bend radius
T = Thickness
C = Centroid of the Angle
S = Shear Centre

Fig. 1.2a Typical cross-section of a cold-formed 60° angle
Fig. 1.2b Typical cross-section of a schifflerized angle

U-U = Major axis
V-V = Minor axis
a = Unbent portion of the leg
b = Bent portion of the leg
t = Thickness
c = Fillet radius
C = Centroid of the Angle
s = Shear Centre
Fig. 1.3 Rollers used for schifflerization
Fig. 1.5 Different types of failure of tension members

(a) Gross-section

(b) Net-section

(c) Block shear failure
Fig. 1.6 End, edge and combined mode of failure
Fig. 3.1 Finite element model of test set-up for 60° angle
Fig. 3.2 Finite element model showing the elements of 60° angle
Fig. 3.4 Residual stress distribution over the thickness of schifflerized angle
Fig. 4.1 Experimental set-up for testing 60° angle
Fig. 4.2
Circular plates bolted to the Gilmore plate

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Fig. 4.3
Top base plate with load cell screw and bottom base plate with jack cover

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Fig. 4.4
Solid blocks, base plates and circular plates used in the set-up
Fig. 4.5
Close-up of base plate with solid and supporting angles
Fig. 4.6
Circular plate, hydraulic jack and end set-up
Fig. 4.7
Close-up of end set-up with 60° angle
Fig. 4.8
60° Angle under test
Fig. 4.9
Cold-formed 1.5x1.5x1/8 in. Cl#1 angle after failure by torsional-flexural buckling
Fig. 4.10
Schifflerized angle (4x4x1/4 in.) after failure by torsional-flexural buckling

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Fig. 4.11
All the 60° angles after failure
Fig. 4.12 End-fixture for testing angles under eccentric tension
Fig. 4.13 Front view of the End-fixture for testing angles under tension

Dimension in mm.
Fig 4.15a Photo showing the End-fixture for testing angles under eccentric tension
Fig. 4. 15b
Angle under eccentric tension
Fig. 4.16. Edge mode of failure of 51x51x4.8 mm angles
3.5*3.5*5/16 in. Angle
Fy=333 MPa
Fu=514 MPa
End Dist. Sp. 3=25.26 mm
Edge Dist. Sp. 3=45.60 mm
Pitch Sp. 3=51.70 mm
Fail Load Sp. 3=228.0 kN

Fig.4.17 End mode of failure of 79x79x8 mm angle
Fig. 4.18 Combined mode of failure of 79x79x6.4 mm angle

3.5\(^\circ\) x 3.5\(^\circ\) 1/4 in. Angle
Fy = 328 MPa
Fu = 544 MPa

End Dist. Sp. 3 = 24.50 mm
Edge Dist. Sp. 3 = 44.00 mm
Pitch Sp. 3 = 51.32 mm

Full Load Sp. 3 = 174.0 kN
Fig. 4.19

Bottom end fixture for testing effect of pre-tension on angles
Fig. 4.20 Cold-formed angles (65x65x4 mm) with snug tight bolts after failure
Fig. 4.21 Cold-formed angles (65x65x4 mm) with pre-tensioned bolts after failure

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Fig. 4.22 Hot-rolled angles (65x65x6.4 mm) with snug tight bolts after failure
Fig. 4.23 Hot-rolled angles (65x65x6.4 mm) with pre-tensioned bolts after failure

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Fig. 4.24 Graph showing load-elongation curves for 65x65x4 mm angles with snug tight and pre-tensioned bolts.

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Average Joint Elongation (mm)

--- SNUG TIGHT  --- PRETENSIONED

LOAD - ELONGATION CURVE FOR 90 HOT ROLLED ANGLES

Fig. 4.25 Graph showing load-elongation curves for 65x65x6.4 mm angles with snug tight and pre-tensioned bolts

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Fig. 4.26 Stress-strain curve for 60° angles
**Fig. 4.27** Stress-strain curve for 90° angles
Appendix A

FINITE ELEMENT MODEL

OF 60° ANGLES

USING ABAQUS
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<td>57.50</td>
<td></td>
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<tr>
<td>20009</td>
<td>1516.00</td>
<td>57.50</td>
<td>57.50</td>
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<tr>
<td>20010</td>
<td>1516.00</td>
<td>57.50</td>
<td>57.50</td>
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</tbody>
</table>

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* USER SUBROUTINES SUBROUTINE SIGINI(SIGMA,COORDS,NTENS,NCRDS,NOEL)  
DIMENSION SIGMA(NTENS),COORDS(NCRDS)

W = 0.0  
A = 0.0  
C = 0.0

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**EQUATION 1.**

\[ E = \frac{1}{2} k x^2 \]

**EQUATION 2.**

\[ T = \frac{1}{2} k \omega^2 x \]

**EQUATION 3.**

\[ F = k x \]

**EQUATION 4.**

\[ \Delta E = \frac{1}{2} k \Delta x^2 \]

**EQUATION 5.**

\[ \Delta T = \frac{1}{2} k \Delta \omega^2 x \]

**EQUATION 6.**

\[ \Delta F = k \Delta x \]

**EQUATION 7.**

\[ E = \int_{x_1}^{x_2} k x \, dx \]

**EQUATION 8.**

\[ T = \int_{\omega_1}^{\omega_2} \frac{1}{2} k \omega^2 x \, d\omega \]

**EQUATION 9.**

\[ F = \int_{x_1}^{x_2} k x \, dx \]

**EQUATION 10.**

\[ \Delta E = \int_{x_1}^{x_2} \frac{1}{2} k \Delta x^2 \, dx \]

**EQUATION 11.**

\[ \Delta T = \int_{\omega_1}^{\omega_2} \frac{1}{2} k \Delta \omega^2 x \, d\omega \]

**EQUATION 12.**

\[ \Delta F = \int_{x_1}^{x_2} k \Delta x \, dx \]
Appendix B

FINITE ELEMENT ANALYSIS

FOR SCHIFFLERIZED

102x102x6.4 MM ANGLE

DATA CHECK RUN
ABACUS PRODUCTION VERSION 4-7-22  DATE 28APR 93  TIME 1:47:44  PAGE

ABACUS

THIS PROGRAM HAS BEEN DEVELOPED BY
HIBBITT, KARLSSON AND SORENSEN, INC.
100 MEDWAY STREET
PROVIDENCE, R.I. 02906

THIS IS A PROPRIETARY PROGRAM AND IS MADE
AVAILABLE FOR INTERNAL USE AT UNIVERSITY OF WINDSOR
UNDER THE TERMS OF THE ACADEMIC LICENSE AGREEMENT
WITH H.K.S. ALL USAGE MUST BE UNDER THE DIRECT
SUPERVISION AND CONTROL OF THE DESIGNATED USER.
THE DESIGNATED USER IS DR. GEORGE ABDEL-SAYED.
ANY NON-ACADEMIC USAGE OF THE PROGRAM REQUIRES
PAYMENT OF A MONTHLY CHARGE. ASSISTANCE AND
OTHER INFORMATION MAY BE OBTAINED FROM THE
DESIGNATED USER.

FOR ASSISTANCE OR ANY OTHER INFORMATION CALL
401-861-0220

********** NOTICE **********

THIS IS ABACUS VERSION 4-7.

SOME INPUT DATA OPTIONS AND USER SUBROUTINE INTERFACES
ARE INCOMPATIBLE WITH VERSION 4-5.
PLEASE MAKE SURE YOU ARE USING VERSION
4-6 OR 4-7 MANUALS.
OPTIONS BEING PROCESSED

**HEADING**
SCHIFFLERISED 60 DEGREE ANGLE, SIZE 4 x 4 x 1/4 x LENGTH

**NODE**
*NGEN, NSET=SP1
*NGEN, NSET=SP2
*NPRINT, NSET=TOPPL

THE FOLLOWING NODES WILL BE USED IN THE NFILL GENERATION

BOUND 1
141 143 145 147 149

BOUND 2
181 183 185 187 189

*NGEN, NSET=ROTAN
*NGEN, CHANGE NUMBER=10000, OLD SET=ROTAN, SHIFT=NEWSET, TOPAN
*NGEN, CHANGE NUMBER=10000, OLD SET=TOPPL, SHIFT=NEWSET, TOPPL
*NSET, NSET=TOPAN, GENERATE
*NPRINT, NSET=SPAR

THE FOLLOWING NODES WILL BE USED IN THE NFILL GENERATION

BOUND 1
31 33 3 5 7 9 11 13 15 17 19 21 23 25 27

BOUND 2
10001 10003 10005 10007 10009 10011 10013 10015 10017 10019 10021 10023 10025 10027

*NODE
*NGEN, NSET=B1
*NGEN, NSET=SP1
*NPRINT, NSET=BB1

THE FOLLOWING NODES WILL BE USED IN THE NFILL GENERATION

BOUND 1
10501 10503 10505

BOUND 2
10551 10553 10555

*NGEN, NSET=BB
*NGEN, NSET=BB2
*NPRINT, NSET=BB2
THE FOLLOWING NODES WILL BE USED IN THE NFILL GENERATION

BOUND 1 12501 12503 12505
BOUND 2 12551 12553 12555

BOUND 1 18001 18003 18005 18007 18009
BOUND 2 18301 18303 18305 18307 18309

BOUND 1 18009 18011 18013 18015 18017
BOUND 2 18309 18311 18313 18315 18317

BOUND 1 18301 18303 18305 18307 18309
BOUND 2 18601 18603 18605 18607 18609

BOUND 1 18309 18311 18313 18315 18317
BOUND 2 18609 18611 18613 18615 18617
THE FOLLOWING NODES WILL BE USED IN THE NFILL GENERATION

BOUND 1
18601 18603 18605 18607 18609

BOUND 2
18901 18903 18905 18907 18909

THE FOLLOWING NODES WILL BE USED IN THE NFILL GENERATION

BOUND 1
18609 18611 18613 18615 18617

BOUND 2
18909 18911 18913 18915 18917

*NSSET=ALL1
*NCOPY,CHANGE NUMBER=1000,OLDSET=ALL1,REFLECT=MIRROR,NEWSET=ALL2

*NODE
*NGEN,NSSET=PS1
*NGEN,NSSET=PS2
*NGEN,NSSET=PS3
*NGEN,NSSET=PS5
*NFILL,NSSET=PS5

THE FOLLOWING NODES WILL BE USED IN THE NFILL GENERATION

BOUND 1
20001 20003 20005 20007 20009

BOUND 2
20301 20303 20305 20307 20309

*NFILL,NSSET=PS6

THE FOLLOWING NODES WILL BE USED IN THE NFILL GENERATION

BOUND 1
20009 20011 20013 20015 20017

BOUND 2
20309 20311 20313 20315 20317

*NGEN,NSSET=PS7
*NGEN,NSSET=PS8
*NFILL,NSSET=PS9

THE FOLLOWING NODES WILL BE USED IN THE NFILL GENERATION

BOUND 1
20301 20303 20305 20307 20309

BOUND 2
20601 20603 20605 20607 20609
THE FOLLOWING NODES WILL BE USED IN THE NFILL GENERATION

BOUND 1  
20309 20311 20313 20315 20317

BOUND 2  
20609 20611 20613 20615 20617

*NFILL,NSET=PS10

THE FOLLOWING NODES WILL BE USED IN THE NFILL GENERATION

BOUND 1  
20601 20603 20605 20607 20609

BOUND 2  
20901 20903 20905 20907 20909

*NFILL,NSET=PS14

THE FOLLOWING NODES WILL BE USED IN THE NFILL GENERATION

BOUND 1  
20609 20611 20613 20615 20617

BOUND 2  
20909 20911 20913 20915 20917

*MATERIAL,NAME=M1
*MATERIAL,NAME=M2
*ELASTIC
*PLASTIC
MESH PLOTTING

*PLOT

*DRAW.ELNUM

PLOT FRAME 1
CONSISTS OF ALL ELEMENTS
ALL SIZES ARE IN PLOTTER UNITS
FRAME SIZE
PICTURE SIZE
POSITION OF LOWER LEFT HAND CORNER OF PICTURE
POSITION OF START OF TITLE
POSITION OF UPPER LEFT HAND CORNER OF VARIABLE LIST
POSITION OF AXIS
SIZE OF CHARACTERS
WIDTH OF PAPER STRIP BETWEEN FRAMES

X-DIRECTION
10.000
7.0000
3.0000
1.5000
1.0000
1.5000
10.000
7.0000
2.0000
1.0000
1.5000

Y-DIRECTION
10.000
7.0000
2.0000
1.0000
1.5000
0.16000
3.0000

NUMBER OF COLORS AVAILABLE
1

*VIEW POINT. DEFINITION: MODAL AXIS ROTATION
*VIEW POINT. DEFINITION: MODAL AXIS ROTATION
*VIEW POINT. DEFINITION: MODAL AXIS ROTATION
*VIEW POINT. DEFINITION: MODAL AXIS ROTATION
*VIEW POINT. DEFINITION: MODAL AXIS ROTATION
*VIEW POINT. DEFINITION: MODAL AXIS ROTATION

*PLOT

*DRAW.MODENUM

PLOT FRAME 2
CONSISTS OF ALL ELEMENTS
ALL SIZES ARE IN PLOTTER UNITS
FRAME SIZE
PICTURE SIZE
POSITION OF LOWER LEFT HAND CORNER OF PICTURE
POSITION OF START OF TITLE
POSITION OF UPPER LEFT HAND CORNER OF VARIABLE LIST
POSITION OF AXIS
SIZE OF CHARACTERS

X-DIRECTION
10.000
7.0000
3.0000
1.5000
1.0000
1.5000
10.000
7.0000
2.0000
1.0000
1.5000

Y-DIRECTION
10.000
7.0000
2.0000
1.0000
1.5000
0.16000
3.0000

0.16000
The view angles are: 1.57, 0.785, 0.0000E+00

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<th>Definition</th>
<th>Modal Axis</th>
<th>Rotation</th>
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Plot frame 3 consists of all elements.

All sizes are in plotter units.

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<th>3</th>
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</table>

Frame size: 10.000

Picture size: 7.0000

Position of lower left hand corner of picture: 1.0000

Position of upper left hand corner of variable list: 0.1000

Position of axis: 1.0000

Size of characters: 1.5000

Width of paper strip between frames: 5.0000

The view angles are: 1.57, 0.785, 0.0000E+00

Number of colors available: 1

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<th>View Point</th>
<th>Definition</th>
<th>Modal Axis</th>
<th>Rotation</th>
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WAVEFRONT MINIMIZATION

NUMBER OF NODES 593
NUMBER OF ELEMENTS 380
ORIGINAL MAXIMUM D.O.F. WAVEFRONT ESTIMATED AS 162
ORIGINAL RMS D.O.F. WAVEFRONT ESTIMATED AS 108

SEARCH FOR POSSIBLE STARTING NODES IS COMPLETED

PHERIPHERAL DIAMETER IS DEFINED BY NODES 19017 20917

WAVEFRONT OPTIMIZED BY CHOOSING 20917 AS THE STARTING NODE

PROBLEM SIZE

NUMBER OF ELEMENTS IS 380
NUMBER OF NODES IS 593
TOTAL NUMBER OF VARIABLES IN THE MODEL 3598
MAXIMUM D.O.F. WAVEFRONT ESTIMATED AS 164
RMS WAVEFRONT ESTIMATED AS 96

FILE SIZES - THESE VALUES ARE IN WORDS AND ARE CONSERVATIVE UPPER BOUNDS

UNIT

LENGTH

2 95539
19 11444
13 105380
22 185380
25 69400
28 22800

IF THE RESTART FILE IS WRITTEN, ITS LENGTH WILL BE APPROXIMATELY
109102 WORDS WRITTEN IN THE PRE PROGRAM
PLUS 75240 WORDS WRITTEN AT THE BEGINNING OF EACH STEP
PLUS 232007 WORDS FOR EACH INCREMENT WRITTEN TO THE RESTART FILE

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null
Appendix C

FINITE ELEMENT ANALYSIS

FOR SCHIFFLERIZED

102x102x6.4 MM ANGLE

LAST PAGE OF THE OUTPUT
THE ANALYSIS IS ENDED BECAUSE OF THE PROBLEMS ENCOUNTERED.

LOOK AT THE MESSAGES SINCE THE START OF THE LAST INCREMENT FOR THE CAUSE OF THE TERMINATION.

END OF RUN.

RUN SUMMARY:

19 INCREMENTS

7 CUTBACKS IN AUTOMATIC INCREMENTATION

115 ITERATIONS

0 REPRESENTS DECOMPOSITION OF THE MASS MATRIX.
Appendix D

FORTRAN PROGRAM

TO CALCULATE

THE PROPERTIES OF

60° ANGLES
C** PROPERTY OF COLD FORMED ANGLES
C** REAM I.VI.P.C.IP.S.J.P.L
OPEN (UNIT=9, FILE=’PCOLO DATA A’)

100 READ (9, *) A, T, L, R

C** R=13 MM FOR 1.5X1.5X1/8 IN., R=15 MM FOR 2X2X3/16 IN. ANGLES
C**
C** AT=3.1416
R1=0.5*T
R=0.5*T=0.5*PT/3.0
A1=0.5*AT=(A1+0.5*I1)
U=0.5*I1=(T=3)*8.0+(B=3)*T/24.0+(B=3)/9.0*(B+
3.464*(R1))**2
1.05*(T*(R1))/2.5=2.1*(R1))
IV=T=B0+*T*(T=3)/24.0+(B=3)/4.0*(1.73=T=2.5*G=3.2*2)**2
-=0.73*T*(T=3)+2.1*T=61.0.173=R0.25*T**2-A1*(UC)**2
J=1.05*(T**3)+3.0
US=(T**3)+3.0*(B=3)+R=3.4*(R=2)+R=10.9*B=R=2)**

IPS=IIV+IIV+IIV=11*(US++2)
IPC=IV=I
UC=S=I(T=I/A1)
RV=S=I(T=I/A1)
J=IIV/(1000000.0
IV=IIV/(1000000.0
IPS=IPS/(1000000.0
IPC=IPC/(1000000.0
WRITE (6, 20) UC, US, A1, IU, IV, IPC, IPS, RV, RV
20 FORMAT (F5.2, I4, F4.1, 1X, F6.1, 12(F12.9), 2(I1, F4.1), 1X, F6.1)

IF (I.LE. 19) GO TO 100
STOP
END

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FILE: KK10  FORTRAN AI UNIVERSITY OF WINDSOR

C: 20 TO FIND THE MINIMUM RADIUS OF GYRATION FOR DIFFERENT SPECIMENS
C: LINES ADDED 5,6,7,8,9,10,11,12,15,17,19,21,25
REAL IV,IV1,IV2,IV11,IV12,IV13,IVS
OPEN(IV11=1,IV12=1,IV13=1/10000 DATA A1)

1=1

100 READ(9,*) W,A,T
   B=A-A
   C=0.5*T
   IU1=(A*T**3/16.0)+(B**3)*T)/48.+0.
   IU2=(IA-T/2.0)**3/24.+0.(A-T/2.0)**3)/6.0
   IV1=(B**3/4)+((A-T/2.)**3)/24.+0.(A-T/2.)**3)/6.
   IU=(IU1+IU2)
   IV=(IV1+IV2)
   U1=(2.0-(A-T/2.)**3)/24.+0.(A-T/2.)**3)/6.
   US=2.*(U1-IU1)/0.707*UI
   U1=(U1-IU1)*US
   RV=SQRT(U1)
   RU=SQRT(U2)
   J=(2.0/3.0)*(A+B-T/2.0)*T**3
   IU=IU/1000000
   IV=IV/1000000
   IPS=IPS/1000000
C: WRITE(6,920) UI,IV1,IV2,IV13,IV1,IVS
C: STOP
20 FORMAT(4*(I5,F6.2),3*(I5,F6.1),3*(I5,F7.5),2*(I5,F4.1),1X,F8.1)
   IF(I*LE.20 ) GO TO 100
STOP
END

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Appendix E

FORTRAN PROGRAM

TO CALCULATE THE COMPRESSIVE RESISTANCE OF 60° ANGLES

ACCORDING TO VARIOUS SPECIFICATIONS
FILE: LOAD

1. COMPREHENSIVE STRENGTH OF A PIPELINE ACCORDING TO CSA-915.1

2. C T T

3. WRITE(99,20) CR, W, LAMDA, FY1

4. WRITE(6,20) CR, W

5. FORMAT (IX, 2(5X, F6.2))

6. END

7. IF (I .LE. 38) GOTO 100

8. STOP

9. END

10. ...

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FILE: CANCSA.FOR

C COMPUTATIVE RESISTANCE OF AN ANGLE ACCORDING TO CAN/CSA-S84.1

EXTERNAL VALUE
V, K, L, W
UNIT=9, FILE="/CANCSA.DATA"

READ(*) T, C, T1, I
READ(*) T2, T3, T4
READ(*) T5
READ(*) T6
READ(*) T7
READ(*) T8
READ(*) T9
READ(*) T10
READ(*) T11
READ(*) T12
READ(*) T13
READ(*) T14
READ(*) T15
READ(*) T16
READ(*) T17
READ(*) T18
READ(*) T19
READ(*) T20
READ(*) T21
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READ(*) T82
READ(*) T83
READ(*) T84
READ(*) T85
READ(*) T86
READ(*) T87
READ(*) T88
READ(*) T89
READ(*) T90
READ(*) T91
READ(*) T92
READ(*) T93
READ(*) T94
READ(*) T95
READ(*) T96
READ(*) T97
READ(*) T98
READ(*) T99
READ(*) T100

END
C  CALCULATION OF FAILURE LOAD BY ASECE MANUAL NO. 52

C

OPEN(UNIT=9,FILE='PLOAD DATA A')
G=1
WRITE(*,*) 'ASECE MANUAL NO. 52'
WRITE(*,*) 'F=0.71'
100 F=0.7
E=8/25.4
T=1/25.4
P=8/25.4
RA=RA-160.5
IF(I.LE.10)THEN
IFRA=RA+PI/6.0
ELSE
RA=RA+PI/3.0
ENDF
ENDIF
A=2/25.4
L=L/25.4
=144.0/SORT(FY)
F=(X+G1=25.0)GOTO 300
FCR=(1.677-0.677*X/Y)FY
IF(X.LT.Y)THEN
FY=FCR
ELSE
FY=FCR1
ENDIF
FA=(8S600C.0)/((SL)**2)
ENDIF
CR=A1*FA
CR=CP*4.450
WRITE(*,*) 'W/T GT 25.0'
GOTO 300
I = I-1
IC(I.LE.38)GOTO 100
STOP
END

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```
C  CALCULATION OF SHEAR LOAD BY DISC LOAD AND RESISTANCE FACTOR
C
C++=DISC RADIUS
EE=1
OPEN(1,UNIT=3,FILE='RMDAL')
WRITE(5,*) 'DISC LOAD AND RESISTANCE FACTOR'
WRITE(6,*) 'R=2.4'
IT=1
X=0.4
F=2.510.05
DI=3.166
DIH=0.886
I100=READ(9,*) S=1,10,1,11,1,2,1,3,1,4,1
R=2.4
T=7/25.4
A=5/25.4
PRE=2.5/25.4
CA=1/4=70.6
I=0=0.6
IF IT.LE.10) THEN
W=P=PA=1/3.6
END 
C++
W=P=PA=1/3.6
END 
D=
W=PF=1/25.4
A=5/25.4
B=1/25.4
C=1/25.4
D=1/25.4
E=1/25.4
F=1/25.4
G=1/25.4
H=1/25.4
I=1/25.4
J=1/25.4
K=1/25.4
L=1/25.4
M=1/25.4
N=1/25.4
O=1/25.4
P=1/25.4
Q=1/25.4
R=1/25.4
S=1/25.4
T=1/25.4
U=1/25.4
V=1/25.4
W=1/25.4
X=1/25.4
Y=1/25.4
Z=1/25.4
IF X.GE.Y .AND. X.LT. 2) THEN
FY=0.144970
X=0.0144970
Y=7/25.4/SQRT(FY)
GY=FY*0.00647*(X)*SQRT(FY)
G2=15.050.000/((FY**(X**2))
IF X=LT. 1 THEN
FY=0.144970
ELSE
FY=0.02*FY
ENDIF
END 
ELSE
LAMDA=((K*L)/(R**PI))*SQRT(FY/E)
IF(LAMDA.LE. 1.5) THEN
F=1.65858*(LAMDA**2)*FY
ELSE
F=0.87/(LAMDA**2)*FY
ENDIF
C=0.877/(LAMDA**2)*FY
C*FY=FY*0.144970
X=0.0144970
Y=7/25.4/SQRT(FY)
FY=FY*0.00647*(X)*SQRT(FY)
G=15.050.000/((FY**(X**2))
IF X=LT. 1 THEN
FY=0.144970
ELSE
FY=0.02*FY
ENDIF
END 
ELSE
LAMDA=((K*L)/(R**PI))*SQRT(FY/E)
IF(LAMDA.LE. 1.5) THEN
F=1.65858*(LAMDA**2)*FY
ELSE
F=0.87/(LAMDA**2)*FY
ENDIF
C=0.877/(LAMDA**2)*FY
C*FY=FY*0.144970
X=0.0144970
Y=7/25.4/SQRT(FY)
FY=FY*0.00647*(X)*SQRT(FY)
G=15.050.000/((FY**(X**2))
IF X=LT. 1 THEN
FY=0.144970
ELSE
FY=0.02*FY
ENDIF
END 
END
STOP
END

```

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C. **CALCULATION OF THE CALLUS FORM ON AN ANGLE BY SPECTROPHOTOMETER**

```plaintext
C**

REAL L,L(0.0),L(0.0)
OPEN(UNIT=9,FILE='REL72D.DAT',A*)
WRITE(6,*),C,
WRITE(6,*),C
100 READ(9,*),X,Y,Z
K=0.7
R=3.1416
SA=3.0
R=3.0
C=0.0
IF(I.LT.19)THEN
    W=0
ELSE
    W=2.1
ENDF
Y=1.5*2.5/(2.5+2.5)
Z=(1.5)/((1.5)*2.5)
F=2.1
IF(X.LT.W) THEN
    C=0.5
ENDIF
C
END
```

```plaintext
**FORMAT (A,E3)**
```

```
**Y=1.5*2.5/(2.5+2.5)
Z=(1.5)/((1.5)*2.5)
F=2.1
IF(X.LT.W) THEN
    C=0.5
ENDIF
C
END
```
### VITA AUCTORIS

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<thead>
<tr>
<th>Name</th>
<th>Krishna Kishore Sankisa</th>
</tr>
</thead>
<tbody>
<tr>
<td>Place Of Birth</td>
<td>Palakol, Andhra Pradesh, INDIA</td>
</tr>
<tr>
<td>Year Of Birth</td>
<td>March 5, 1970</td>
</tr>
<tr>
<td>Education</td>
<td>Bharatiya Vidya Bhavan’s Public School, Andhra Pradesh, INDIA 1985-1987</td>
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<td></td>
<td>Osmania University, Hyderabad, INDIA</td>
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<tr>
<td></td>
<td>1987-1991 B.E.</td>
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<tr>
<td></td>
<td>University of Windsor, Windsor, ON</td>
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</tbody>
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