PGandE Letter No.: DCL-84-219

ENCLOSURE

LICENSE CONDITION 2.C.(11)

FINAL REPORT

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ATTACHMENTS

SECTION 7

TECHNICAL TOPICS

LICENSE CONDITION 2.C(11), Item 7

"PGandE shall conduct a program to demonstrate that the following technical topics have been adequately addressed in the design of small and large bore piping supports:

(a) Inclusion of warping normal and shear stresses due to torsion in those open sections where warping effects are significant.

(b) Resolution of differences between the AISC Code and Bechtel criteria with regard to allowable lengths of unbraced angle sections in bending.

(c) Consideration of lateral/torsional buckling under axial loading of angle members.

(d) Inclusion of axial and torsional loads due to load eccentricity where appropriate.

(e) Correct calculation of pipe support fundamental frequency by Rayleigh's method.

(f) Consideration of flare bevel weld effective throat thickness as used on structural steel tubing with an outside radius of less than 2T.

PGandE shall submit a report to the NRC Staff documenting the results of the program."

7.0 Description of Program, Development, and Followup Information

7.1 Program Scope

7.1.1 Small Bore

A discussion of the small bore program to address the issues of Item 1, as well as Item 7 of the License Condition, is presented in Section 1 of this report.

7.1.2 Large Bore

To provide the necessary confidence that, with the above technical issues considered, the piping meets all design criteria, a subset of the calculations will be sampled and checked. The calculations used in the sample were selected using a random selection approach. A total of 200 calculations was selected from a total population of 3668 calculations (approximately 5.4% of the total). In establishing the total population support designs containing variable springs and constant supports were excluded since these devices do not carry seismic loads.

All Diablo Canyon Unit 1 pipe supports designated as Design Class I and code class J, D, A, B, C, G, @, or Z as listed in the GPSS (General Pipe Support Status) were sorted and listed by sequence number and were assigned index numbers from 1 through 3668 which were later matched with random numbers. The random numbers were generated using the International Math Statistic Library (IMSL) on Bechtel's Univac Computer system. The random numbers were generated and assigned index numbers from 1 through 200. Pipe support selection was obtained by matching the random numbers (as listed from 1 to 200) with the pipe support numbers (as listed from 1 to 3668).

Further review of the original 200 supports identified by random selection determined that 40 of these supports were:

1. Nonframe type supports
2. Deleted supports
3. Downgraded to seismic Class II

In order to provide an appropriate sample of 200 supports, additional supports were selected by random methods, as previously described, until an appropriate review population of 200 frame-type supports was obtained.

From the sample, the probability of a support not meeting the criteria, p, is determined. This probability will then be uses to estimate the probability P in the total population. The standard error of proportion will be estimated using the following relationship:

$$s_p = \sqrt{\frac{p(1-p)}{n}} \cdot \sqrt{\frac{N-n}{N-1}} \qquad (1)$$

where:

N is the population size: 3668

n is the sample size: 200

p is the probability of a support not meeting the criteria found in the sample (based on a binomial distribution)

Using s_p found in equation 1, the error of estimation, E, is found using:

$$E = |z| \, s_p \qquad (2)$$

where z is a multiple of the number of standard deviation units

Therefore, the probability P interval in the population is estimated as:

$$(p - E) < P < (p + E) \qquad (3)$$

If the randomly selected sample of 200 are all found to meet the project criteria, it can be shown statistically that there is greater than a 99% confidence level that the remainder of the total population will meet the criteria.

If any of the 200 are found not acceptable for each issue, the sample will be increased to achieve an appropriate confidence level, or a screening criteria will be applied to review the remainder of the total population. The screening criteria will be developed based on the results of the review of the 200 calculations. Such screening criteria may involve a field walkdown to identify particular concerns.

A checklist has been designed to guide the analyst/reviewers in performing the large bore reviews for the technical issues in Item 7 of the Licensing Condition. The checklist is included as Attachment 7-1.

7.2 Future Work

In order to address any future analysis performed on pipe supports, the design criteria contained in M-9 for pipe supports were revised to require consideration of Item 7, parts (a), (c), (d), and (e) of the Licensing Condition. Part (b), concerning unbraced angles, will be used only to qualify existing steel members. A specific restriction will prohibit its use in the design of new supports.

7.3 Technical Issues

7.3.1 Warping Normal and Shear Stresses

Normal and shear stresses due to warping of open sections in torsion is a consideration treated in many technical references and industry publications. In most structural applications where wide flange sections are used, warping effects are small and therefore not considered in the design calculations. Other shapes, including "I" sections, could have a larger contribution from warping and, therefore, should be considered. (1)(2)

There are three considerations in the pipe support design at Diablo Canyon that tend to minimize the significance of the warping phenomena:

- o The predominant use of wide flange sections rather than "I" sections or other cross-sectional shapes having a lesser capacity to restrain torsional loads.

- o The pipe supports are designed to use standard size members and a stiffness criteria that, in most cases, assure that the member stresses will not be the critical factor in the strength of the support.

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(1) The AISC Commentary to Section 1.5.1.4.5 states for warping: "The combination of formulas (1.5-6a) or (1.5-6b) and (1.5-7) provides a reasonable design criterion in convenient form." "Formula (1.5-7) is a convenient approximation which assumes the presence of both lateral bending resistance and St. Venant torsional resistance."

(2) The AISC Manual further states: "Torsional analysis is not required for routine design of most structural steel members." 8th Edition, pg. 1-109.

- o Small bore supports typically use angle or square tube section material that are not subject to warping.

7.3.2 Differences Between AISC Code And Project Criteria

The so called "differences" between AISC and the Project criteria using the Australian data, references 1, 2, and 3, with regard to allowable stresses of angle sections in bending do not really exist. The Project design criteria on this topic are in compliance with the AISC code. The use of the Australian paper is for the applications where specific guidance is lacking in the Code. In fact, AISC not only recognizes the limitations of the Code but also suggests special investigation by the engineer. This position has been addressed in two previous responses to the NRC on the same subject (for convenience excerpts from these submittals are included as Attachment No. 7(b)-1 and 7(b)-2).

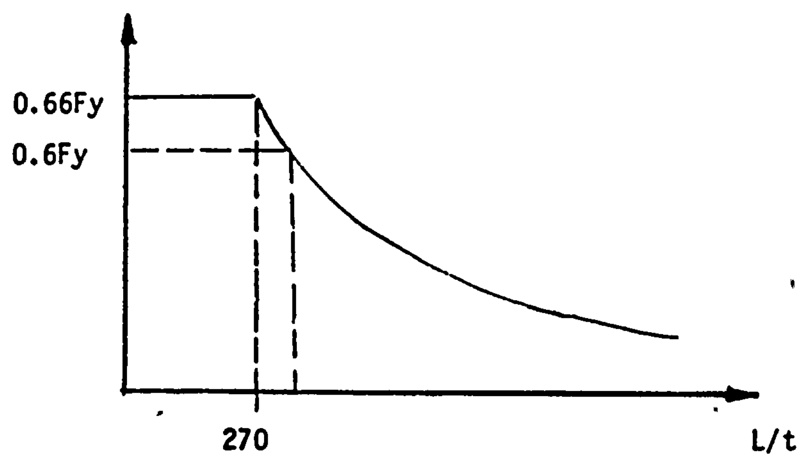
Our understanding of the Staff's concern involves the AISC's position on the Australian data and the appropriateness of its application. To address this, two pieces of additional information are provided as Attachments 7(b)-3 and 7(b)-4. PGandE believes that the AISC's positive and supportive position has been reflected in these two attachments.

Attachment 7(b)-3 is a reprint of the Australian "Safe Load for Laterally Unsupported Angles" published in the official Engineering Journal/AISC, First Quarter, 1984. The AISC's position is summarized in the editor's note to the reprint. The editor stated: "The AISC Specification and Manual offers limited direct design criteria for such members." The reprint of the paper "is in response to the many inquiries AISC has received on the subject." The editor also mentioned the Australian papers "have often been referenced in the past to provide requested design guidance." Thus, it is PGandE's belief that AISC has allowed the use of the Australian paper for

design of angles in bending. Attachment 7(b)-4 is a copy of a handout from an AISC presentation showing further endorsement of the Australian work.

The editor cautions the user about the difference of allowable shear stress between the Australian specification and the AISC specifications, and suggests the allowable bending stress to be 0.6 Fy, instead of 0.66 Fy, where Fy is the yield strength. Project design criteria meet these requirements.

The staff has expressed a concern about using the L/t limits given in the Australian paper if the allowable bending stress is reduced from 0.66 Fy to 0.60 Fy. The sketch below shows that as the allowable bending stress is reduced the L/t limit increases. It should be noted that the theory developed in Attachment 7(b)-4 recommends a L/t limit of 300.



There are additional reasons why the L/t limit of 270 is conservative as explained below:

- The L/t limit of 270 (or 300) is derived from the bending about the major principal axis. This is the loading case in which the angle is most vulnerable to buckling and is generally not the direction of loading in pipe support applications. In pipe support design, the main loading is generally applied in the direction parallel to one of the angle legs. For such cases, the L/t limit may be as high as 990 and 690 for a B/t ratio equal to 6 and 16, respectively where B is the length of leg. However, the L/t limit of 270 is applied to all angle bending load cases in this project. This allows an additional implied conservatism.

- All of the L/t limits derived in the Australian papers and used in the Australian testing are for uniform bending moment along the entire beam. Again, this is the most critical lateral buckling condition and generally does not exist in pipe support application.

Based on the foregoing, it is concluded that the AISC supports and recommends use of the Australian paper. Project design criteria meet or exceed the requirements set forth in the Australian paper and the AISC. Therefore, PGandE believes the Project criteria on bending for angle sections are satisfactory and acceptable.

References

1. B. F. Thomas and J. M. Leigh, "The Behaviour of Latarally Unsupported Angles," BHP Melb. Res. Lab. Rep. MRL 22/4, December 1970.

2. J. M. Leigh and M. G. Lay, "Laterally Unsupported Angles with Equal and Unequal Legs," BHP Melb. Res. Lab. Rep./MRL 22/2, July 1970.

3. "Safe Load Tables for Laterally Unsupported Angles," Australian Institute of Steel Construction, September, 1971.

7.3.3 Lateral/Torsional Buckling Under Axial Loading of Angle Members

Lateral buckling for axially loaded angles is considered using the requirements of the AISC Code. In the commentary to the AISC Code, torsional/compressional buckling is discussed. AISC acknowledges that for singly-symmetric shapes (e.g. angles) with large width-to-thickness ratios, column buckling could occur by twisting at loads smaller than those associated with general column buckling (i.e., that addressed in Section 1.5.1.3 of the AISC Code). AISC references a paper by A. Chajes and G. Winter (reference 1) for further information.

The Chajes and Winter paper states that axial buckling can occur by twisting in thin wall open sections due to their low torsional stiffness. Structural angles generally do not fall into this category. The Chajes and Winter paper provides a method for evaluating the torsional compression buckling stress of equal leg angles:

$$F_{\phi} = G(t/a)^2$$

where: F_{ϕ} = critical torsional buckling stress
G = shear modulus
t = leg thickness
a = leg length

Using this equation for angle sizes covering the range used at Diablo Canyon, the critical stress is always above the yield stress. Thus, torsional buckling is not a governing mode of column buckling and does not need to be considered in pipe support design.

For reasons discussed above, lateral/torsional buckling under axial conditions is not a concern in the DCP design of pipe supports using angle members.

As a result of the discussions during the NRC Staff and PGandE meeting on May 9, 1984, a better understanding of the Staff's concern for lateral/torsional buckling was obtained. This issue concerns failure of structural angles in local plate buckling rather than the lateral and torsional buckling discussed in the Winter et.al.[1] paper.

A review of the test results included in a paper by Kennedy and Murty[2] indicates that structural angles can fail in local plate buckling when the b/t ratio is in excess of 15. The theoretical results in this paper indicate that the buckling load is less than 5% below the yield stress for standard structural angles with a $b/t \leq 12$. In order to explicitly address the NRC Staff's concern regarding local buckling of angles, the design drawings will be reviewed during the present effort and occurrences where $b/t > 12$ will be documented. For angle sizes used on the Diablo Canyon Project, the b/t ratio is usually 9 or less. It should be noted that this screening criterion is more stringent than the AISC criterion of $76/\sqrt{fy}$.

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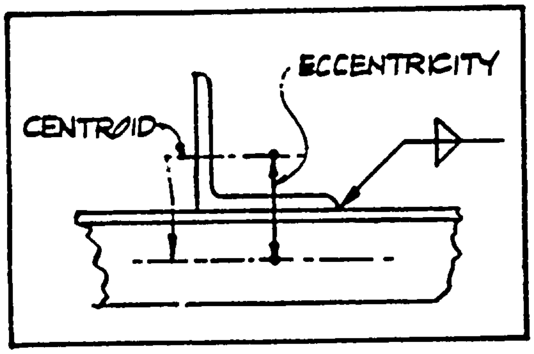
(1) Torsional-Flexural Buckling of the Thin-Walled Members, A. Chajes and G. Winter, ASCE, Structural Division Journal, Aug. 1965

(2) Buckling of Steel Angle and Tee Struts, J. B. Kennedy and M. D. S. Murty, ASCE, Structural Division Journal, Nov. 1972

7.3.4 Inclusion of Axial and Torsional Loads Due to Eccentricity

In order to facilitate the modeling of a pipe support structure in a computer program, in many cases the centroidal axis of two overlapping members are assumed to be intersecting. The joint at the overlapping members (angles for example) is accomplished by welding the intersecting legs together:



This welding locally stiffens the angles and, as a result, they act more in unison, thereby reducing the effect of not explicitly including the eccentricity.

The generation of axial loads due to lack of consideration of eccentricity is, in general, inconsequential. This will be considered, however, along with the torsionally induced loads.

7.3.5 Fundamental Frequency By The Rayleigh Method

The review will identify cases where the displacement method of determining the pipe support frequency may not have approximated the

dominant frequency relative to the piping with sufficient accuracy. In general, the approach used provides sufficient accuracy with only a few exceptions. The most notable exception is a simply supported beam with an overhang. This, and other cases where the adequacy of the displacement method may be questioned (such as cases where the deflected shape due to gravity has significant reverse curvature), will be identified in the review. Where necessary, reanalysis will be performed.

7.3.6 Flare Bevel Weld Effective Throat Thickness

The Project criterion for design of pipe support flare-bevel welds to tube steel used 2.0 t as the tube steel corner radius when determining the effective throat thickness of the weld. The adequacy of the DCP criterion was addressed in PGandE letters DCL-84-083, DCL-84-141, and DCL-84-153.

As described in these letters, site inspections confirmed that the tube steel corner radii are, in fact, 2.0 t (or slightly larger). The technical issue, the size of the weld effective throat, was resolved by performing weld tests which showed the effective throats to be larger than those required by AWS, i.e., 5/16R.

In summary, the DCP flare-bevel weld designs have been shown to be appropriate and conservative.

7.4 Results

7.4.1 Small Bore Support Review Results

A discussion of the results of the small bore support reviews for the Item 7 technical issues is presented in Section 1.3 of this report.

7.4.2 Large Bore Support Review Results

A thorough review of each of the 200 supports for consideration of the technical issues identified in Item 7 of the License Condition was performed and documented. Results are presented as follows:

1. Warping - A thorough evaluation of warping was performed for each support where it is a consideration, i.e., open flanged sections with torsional loads. The following referenced attachments illustrate the warping stress ratios:

   (a) Warping Normal to Allowable Bending Stress Ratio (Attachment 7-2). Note that in 100% of the cases the warping normal stresses are less than 40% of the bending allowable.

   (b) Warping Normal Interaction Value to the Overall Interaction Value Ratio (Attachment 7-3). Note that this graph may be somewhat misleading in that a very low interaction value (i.e., low overall stress) can result in a shift to the right of the curve. In reality, a warping ratio of .4 and a total interaction value of .4 would result in a "1.0" point on the curve, but is inconsequential due to the low total stress values. This curve is being supplied at the NRC Staff's specific request and should be reveiwed with the above considered.

   (c) Warping Shear to the Shear Allowable Stress Ratio (Attachment 7-4). In 100% of the cases, the increased shear stress caused by warping is less than 50% of the shear allowable.

   (d) Total Shear to the Allowable Shear Stress Ratio (Attachment 7-5).

   (e) Total Interaction Value (Attachment 7-6). In 90% of the supports reviewed, the interaction value was less than 60%.

The review for warping indicates that both normal and shear stresses are relatively small in a predominate number of the cases, and in all cases the Project criteria are satisfied.

2. Allowable lengths of unbraced angle sections in bending were addressed generically in Section 7.3.2

3. Consideration of torsional buckling has been addressed generically in Section 7.3.3. Reviews for the effects of lateral buckling on angles were performed where necessary. It should be noted that large bore supports do not in general utilize angle shapes as structural members. However, to the extent they were used, each case was evaluated and shown to be acceptable.

4. Eccentricity was considered for both: (1) load eccentricity and (2) member connectivity eccentricity. All cases were shown to comply with the Project criteria.

5. A review was completed of the methods employed to evaluate natural frequencies of each individual support. This review evaluated mass distribution and multiple pipe arrangement on each support. The review demonstrated that the natural frequencies had been properly determined or that sufficient conservatism existed in the calculated frequency such that no further reanalysis was required.

6. Consideration of flare bevel weld effective throat thickness as used on structural steel tubing was addressed generically in Section 7.3.6 of this report.

The overall review of the sample of large bore supports shows that all supports remain qualified to the Project criteria with one minor exception. In this unique case, a 3-in. cantilevered member of a small bore support is attached to the large bore support frame. Loads from the small bore support were typically included in the

large bore analysis. The support is a three-way restraint and, as such, three loads were provided. The large bore calculation considered two of the loads but did not include the third. When this third load was considered, the large bore support was shown to be qualified for the design basis Hosgri event but not for the smaller seismic event, the DDE conditions. In this case, lower damping, lower stress allowables, and a shift in different response spectra are the primary reasons for the large bore support not meeting the DDE criteria. Fourteen other large bore supports included in the 200 have small bore attachments. In all other cases, all loads from the small bore attachment were properly considered in the calculations.

Based on the foregoing, we believe that the review of the 200 random sample of large bore supports pursuant to Item 7 of the License Condition has demonstrated that the large bore supports have sufficient margin in their design to accommodate the additional technical considerations of this condition.

DATE: 5/15/84
FILE NO.: 146.08
Sheet 1 of 1

DIABLO CANYON PLANT
Lare Bore Pipe Support Review Checklist

NRC LICENSING CONDITION #7 CONCERNS

Pipe Support No. Rev.
Reviewed By: Date:
Checked By: Date:
Approved By: Date:

REVIEW ITEMS	Considerations Acceptable?		REMARKS
	YES	NO	
a) Member centerline eccentricities:			
b) Loads and points of application? eccentricities of load with respect to shear center and centroid of cross section?			
c) Unbraced lengths for angles $L/t \leq 270$?			
d) St. Venant torsional shear stress? (See Attachment #8 of I-55)			
e) Warping normal and shear stresses due to torsion? (See Attachment #6 & 7 of I-55)			
f) Normal stress combined with warping normal stress (absolute summation)?			
g) Lateral, St. Venant, and warping shear stresses combined (absolute summation)?			
h) Lateral/torsional buckling for angles, is $b/t \leq 12$?			
i) Fundamental frequency method. Are points of contraflexure insignificant? (See Guidelines for Reviewing Frequency Calculations of Pipe Supports.			

012kb

NRC Question: The NRC has raised a question about angle-shaped structural members (Allegation No. 95 from SSER 21).

Response

In this response, the following symbols are used.

List of Symbols

B =	Length of angle leg
t =	Thickness of angle leg
L =	Length of span
Fy=	Minimum Yield Strength
b_f=	Width of Compression Flange

In small bore pipe support design, angle-sectioned beams are frequently used for structural members because of the small loads typically encountered in small bore piping.

Angle sections were used at Diablo Canyon prior to the verification program. Where modifications to existing supports were made during the verification program, structural tubing was often substituted for the original angle section.

The criteria for the use of angles as laterally unsupported beams subjected to bending forces were based upon evaluations initiated in 1977. Project-specific criteria were required because the AISC Manual of Steel Construction (Ref. 1) does not provide guidance for angles with laterally unsupported spans greater than 76.0 b_f/Fy. The term 76.0 b_f/Fy is the allowable span for an unbraced length of a member not meeting the requirements of Section 1.5.1.4.6a of Reference 1. However, these criteria were developed for I beams and not specifically for angles. Reference 1 does not provide criteria for laterally unbraced members greater than 76.0 b_f/Fy. The lack of specific guidance in this area has been recognized in the literature (see Reference 2). However, AISC recognizes that special investigations are necessary for angles with laterally unsupported spans greater than 76.0 b_f / Fy. This is indicated on page 2-21 of Reference 1 where a statement is provided which explains the use of angle load tables. The statement is as follows:

*Excerpt from PGandE Letter No. DCL-84-046, dated February 7, 1984.

"The tables are not applicable for angles laterally unsupported or subjected to torsion; for such members a special investigation is necessary."

Because the AISC did not completely address the design of laterally unsupported angles, PGandE performed a literature search in 1977 to determine if other information was available which would be adequate to set criteria. In late 1977 it was found that extensive testing of laterally unsupported angles loaded in bending had been performed in Australia. Literature which describes the testing, findings, and recommendations has been previously provided to the NRC staff (References 3, 4, and 5).

In the Australian tests, various sizes of angles were characterized by different B/t ratios. Angle sections with B/t ratios between 6 and 16 (Reference 5) have been tested. The majority of angles at Diablo Canyon fall within this range. The only angles at Diablo Canyon not falling into this range have B/t values less than 6. However, at this end of the range (beams with B/t less than 6 are less slender) the data can be used conservatively since the net effect is to allow an increase in acceptable unbraced lengths. Based on the tests and comparison to structural theory, simple formulas were developed in Reference 5 for use in the design of laterally unsupported angles in bending using several different methods of load application.

For all the various angle sections and load cases investigated, Reference 4 recommends that an allowable bending stress of 0.66 Fy may be used if L/t is less than 300. The Diablo Canyon Project Design Criteria M-9 limits the maximum bending stress to 0.6 Fy and a maximum L/t ratio of 270. These limits used at Diablo Canyon fall within the recommendation of Reference 4 and are therefore acceptable.

References

1. American Institute of Steel Construction (AISC) Manual of Steel Construction, Seventh Edition, AISC, New York.

2. B. F. Thomas, J. M. Leigh, M. G. Lay, Civil Engineering Transactions, 1973, The Institution of Engineers, Australia.

3. B. F. Thomas and J. M. Leigh, The Behaviour of Laterally Unsupported Angles BHP Melb. Res. Lab. Rep. MRL 22/4, December 1970.

4. J. M. Leigh and M. G. Lay, Laterally Unsupported Angles with Equal and Unequal Legs. BHP Melb. Res. Lab. Rep./ MRL 22/2, July 1970.

5. Safe Load Tables for Laterally Unsupported Angles, Australian Institute of Steel Construction, September, 1971.

dihedral angles less that 60°, calculations are performed to ensure that the weld qualifies as a partial penetration weld with the proper throat reduction. This reduction is in accordance with the requirements of AISC and AWS.

69. Pullman Power Products procedures reference the PGandE specification to which pipe supports are to be installed and the codes to which the weld procedures specifications (WPS) are qualified. For the WPS which are qualified, it is not necessary, and inappropriate for Pullman QC to inspect the welds to the AWS D1.1 prequalified joints. The weld procedure specification, ESD-223, and the design drawings contain everything needed to inspect the welded joint. Flare groove welds are inspected in accordance with the requirements of ESD-223.

70. It is not necessary for Attachment I of ESD-223 to provide limitations for the minimum dihedral angle for intersecting structural shapes. The limitations on the dihedral angle would be governed by the design drawings used. Throat adjustments are reflected in the weld design calculations. The calculation adjustments have taken into account the effect of skewed dihedral angle rather than perpendicular connections, and have considered that acute angle connections will not have complete fusion to the weld root, due to possible slag inclusions.

XV. It is alleged that:

> The second Stokes DR stated that angle members were two-to-three times too long for the allowable bending stress standard used under the AISC code. The angles could buckle under pressure. One hundred frames of 300 checked contained violations. (Stokes, 11/17/83, pp. 17 and 18)

*Excerpt from PGandE's answer in opposition to Joint Intervenor's Motion to augment or, in the alternative, to reopen the record, dated March 6, 1984.

The M-9 computer analysis for angles omitted the relevant provisions of the American Institute of Steel Construction (AISC) code for allowable bending stress, contrary to licensing commitments. (Stokes, 1/25/84, Tr. 15-21)

71. In paragraphs 71 thru 78, the following symbols are used.

List of Symbols

B = Length of angle leg

t = Thickness of angle leg

L = Length of span

Fy = Minimum Yield Strength

b_f = Width of Compression Flange

72. The criteria for the use of angles as laterally unsupported beams subjected to bending forces were based upon evaluations initiated in 1977. Project-specific criteria were required because the AISC Manual of Steel Construction (Ref. 1) does not provide guidance for angles with laterally unsupported spans greater than $76.0\ b_f/\sqrt{Fy}$. The term $76.0\ b_f/\sqrt{Fy}$ is the allowable span for an unbraced length of a member not meeting the requirements of Section 1.5.1.4.6a of Reference 1. However, these criteria were developed for I beams and not specifically for angles. Reference 1 does not provide criteria for laterally unbraced members greater than $76.0\ b_f/\sqrt{Fy}$. The lack of specific guidance in this area has been recognized in the literature (see Reference 2). However, AISC recognizes that special investigations are necessary for angles with laterally unsupported spans greater than $76.0\ b_f/\sqrt{Fy}$. This is indicated on page 2-21 of Reference 1 where a statement is

provided which explains the use of angle load tables. The statement is as follows:

> "The tables are not applicable for angles laterally unsupported or subjected to torsion; for such members a special investigation is necessary."

73. Because the AISC did not completely address the design of laterally unsupported angles, PGandE performed a literature search in 1977 to determine if other information was available which would be adequate to develop criteria. In late 1977 it was found that a theoretical solution to the design of laterally unsupported angle beams was available. The theory had also been verified with extensive testing. The theory and the testing were completed in Australia (Reference 3, 4, and 5).

74. In the Australian tests, various sizes of angles were characterized by different B/t ratios. Angle sections with B/t ratios between 6 and 16 (Reference 5) have been tested. The majority of angles at Diablo Canyon fall within this range. The only angles at Diablo Canyon not falling into this range have B/t values less than 6. However, at this end of the range (beams with B/t less than 6 are less slender) the data can be used conservatively since the net effect is to allow an increase in acceptable unbraced lengths. Based on the tests and comparison to structural theory, simple formulas were developed in Reference 5 for use in the design of laterally unsupported angles in bending using several different methods of load application.

75. For all the various angle sections and load cases investigated, Reference 4 recommends that an allowable bending stress of 0.66 Fy may

be used if L/t is less than 300. The Diablo Canyon Project Design Criteria M-9 limits the maximum bending stress to 0.6 Fy and a maximum L/t ratio of 270. These limits used at Diablo Canyon fall within the recommendation of Reference 4 and are therefore acceptable.

76. DR 83-042-S, written by Mr. Stokes, questioned the acceptability of certain unbraced angle members because the unsupported spans of those members are greater then 76.0 $b_f/\sqrt{F_y}$ per section 1.5.1. 4.6b of Reference 1.

77. It should also be pointed out that the 18 pipe supports identified in the DR 83-042-S as discrepant have been reviewed. All of the angle beam spans are found within the Project Design Criteria.

78. It is concluded that the Project Design Criteria on the design of laterally unsupported angle beams has adequately covered the length greater than 76.0 $b_f/\sqrt{F_y}$.

References

1. American Institute of Steel Construction (AISC) Manual of Steel Construction, Seventh Edition, AISC, New York.

2. B. F. Thomas, J. M. Leigh, M. G. Lay, Civil Engineering Transactions, 1973, The Institution of Engineers, Australia.

3. B.F. Thomas and J. M. Leigh, The Behaviour of Laterally Unsupported Angles BHP Melb. Res. Lab. Rep. MRL 22/4, December 1970.

4. J. M. Leigh and M. G. Lay, Laterally Unsupported Angles with Equal and Unequal Legs. BHP Melb. Res. Lab. Rep./ MRL 22/2, July 1970.

5. Safe Load Tables for Laterally Unsupported Angles, Australian Institute of Steel Construction, September, 1971.

XVI. It is alleged that:

> The third Stokes DR stated the distance between the center of Hilti bolt holes was not verified as the same length required and specified on the drawing. QC had measured the distance between the centers of plates attached to the bolts whereas location of the bolts is supposed to be control for the location of the plates. As a result, whole packages could be in the wrong location. (Stokes, 11/17/83, pp. 18 and 19)

79. The capacity of a concrete anchor bolt is a function of the bolt length (embedment), bolt material, and concrete strength. Anchor bolt capacity relates to a shear cone of concrete originating at the end of the anchor bolt embedment. This cone projects at a 45^{o} angle to the surface. If two anchor bolts are placed close enough together that their shear cones overlap, some of the strength of the anchor bolts may be lost. The 10d (bolt diameter) criterion between anchor bolts was established to assure this would not occur.

80. All shell type anchor bolts on Diablo Canyon have an embedment of less than five bolt diameters. Since the anchor bolt center lines are ten bolt diameters apart, the shear cones can never overlap. Hence the anchor bolts retain their full capacity. The capacity of an anchor bolt is determined by test. The test for a shell anchor is normally

Safe Load for Laterally Unsupported Angles

Editor's Note: *This article is reprinted in its entirety, without revisions, with the gracious permission of the Australian Institute of Steel Construction. It is in response to the many inquiries AISC has received on the subject of laterally unsupported angles. The AISC Specification and Manual offers limited direct design criteria for such members. Relevant Australian research reports and publications, primarily this article, have often been referenced in the past to provide requested design guidance.*

The reader should note that the load tables were prepared in conformance with the Australian Specification which does not necessarily correspond to the AISC Specification (i.e., see shear requirement). In addition, some of the angle sizes shown may not be readily available in the U.S. However, the tables provide a quick rational estimate of angle bending capacity. References on the theoretical and experimental behavior and recommended design of laterally unsupported angles are listed at the end of the article and may be obtained from AISC headquarters.

In addition, a convenient rule of thumb based on the Australian research that may be applied in angle design for flexure is to simply use an allowable bending stress, $F_b = 0.6F_y$, with appropriate serviceability deflection limits. Available evidence indicates that laterally unsupported practical angle sections in bending experience excessive deflections prior to any lateral buckling and, therefore, will be governed by deflection limitations rather than buckling.

An Explanation of the Tables

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INTRODUCTION

The tables are based theoretically on the constant moment case and use a maximum permissible stress of $0.66F_y$,[1] where F_y is the material yield stress. However, for short spans the loads are reduced where necessary to ensure that the maximum permissible shear stresses given in Ref. 1 are not exceeded. The safe loads are applicable for applied loads within half a leg length on either side of the shear centre (Fig. 1). The method by which this load is obtained is described under "Calculation of Safe Loads." The safe load shown in the tables is the uniformly distributed load which causes a maximum bending moment equal to the critical constant moment. This conversion has been made to correspond with the AISC (Australian) Safe Load Tables.[2] Safe loads are given for

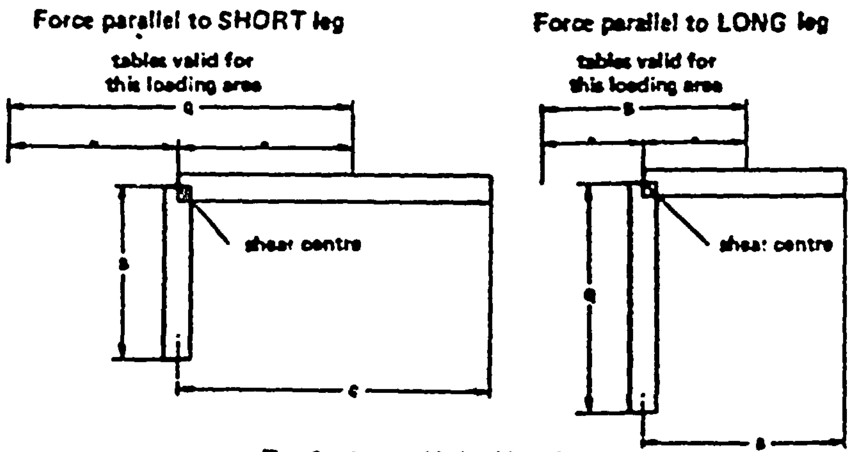

Fig. 1. Acceptable load locations

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steels with nominal yield stresses of 36 and 52 ksi.

In addition to loads, the tables give the associated loading plane deflection of the beam. The deflection is indicated in smaller type directly beneath the corresponding load value.

For cases where the moment on a beam is not constant across the span, the tables give conservative estimates of the load carrying capacity as the constant moment case produces the most critical lateral buckling situation.[3] The same constant moment basis is used for the lateral buckling rules of AS CA1.[1,4]

NOMENCLATURE

B	length of the shorter angle leg as defined in Fig. 1 (actual leg length $-\frac{t}{2}$)
C	centroid location
F_v	maximum shear stress
F_y	yield stress
G	modulus of rigidity
K_T	St. Venant torsional constant
L	length of span
M	moment
M_a	total moment, i.e. applied moment plus moment component due to the dead weight of the beam, calculated about the appropriate leg
M_v	component of the total moment (M_a) about the VV axis
P	applied load
P'	uniformly distributed load
Q	length of longer angle leg as defined in Fig. 1 (actual leg length $-\frac{t}{2}$)
S	shear centre location
T	applied torque
UU	major principal axis
U'U'	major principal axis of the twisted cross-section
VV	minor principal axis
V'V'	minor principal axis of the twisted cross-section
X, Y	axes through the centroid parallel to an angle leg of the twisted cross-section
Z_a	section modulus about the same axis as M_a
Z_x	section modulus about the XX axis
e	load eccentricity
m	weight of beam in lb/inch length
t	thickness of angle leg
u	deflection of the shear centre in the U direction
v	deflection of the shear centre in the V direction
x	deflection of the shear centre in the X direction
y	deflection of the shear centre in the Y direction
σ_a	nominal stress found from $\sigma_a = \frac{M_a}{Z_a}$
σ_{max}	actual maximum section stress
δ	deflection
λ_1, λ_2	coefficients used in equation 8
ϕ_T	the algebraic sum of component twists
θ	angle between XX and UU axis

CALCULATION OF SAFE LOADS

The design relationships for angle beams obtained in Ref. 5 have been used to determine the value of the equivalent uniformly distributed load which produces a maximum section stress of $0.66F_y$. Iterative methods have been used to locate the value of this load from an initial approximation of

$$M_a = \sigma_a Z_a \tag{1}$$

where σ_a is the nominal applied stress, Z_a the section modulus and M_a the total applied moment about some common axis, aa.

For small angles of twist, the nominal applied stress, σ_a, and the actual maximum section stress, σ_{max}, are related by:[5]

$$\sigma_a = 0.80\,\sigma_{max} \tag{2}$$

Hence for

$$\sigma_{max} = 0.66\,F_y \tag{3}$$

$$\sigma_a = 0.528\,F_y \tag{4}$$

Thus, the initial approximations, using the relationships for Z_a from Ref. 5, are:

Unequal Angles—Force direction parallel to short leg

$$M_a = 0.528F_y\,\frac{B^2t(B + 4Q)}{6\,(B + 2Q)} \tag{5}$$

Force direction parallel to long leg

$$M_a = 0.528F_y\,\frac{Q^2t\,(Q + 4B)}{6\,(2B + Q)} \tag{6}$$

Equal Angles

$$M_a = 0.528F_y\,\frac{B^2t}{3.6} \tag{7}$$

For small angles of twist these expressions give the actual value of the safe load directly. However, large angles of twist modify the value of the maximum stress in the section with consequent changes in its load carrying capacity.

Studies in Ref. 6 have shown that the effects of loads located within half a leg length of the shear centre (Fig. 1) are negligible for the load ranges considered. For loads outside these limits the designer should use the procedures on Refs. 5 and 7.

The maximum stress in the section can be found from:

$$\sigma_{max} = \frac{18\,M_a}{(B + Q)^2t}(\lambda_1 + \lambda_2\,\phi_T) \tag{8}$$

where values of λ_1 and λ_2 for each unequal angle section, regardless of t, are given in Table 1.

For all equal angles, $\lambda_1 = 1.0$ $\quad$ $\lambda_2 = ⅓$

The value of ϕ_T is the algebraic sum of the twist due to nonprincipal axis loading,[5] the initial twist and all applied torques. The first two are usually negligible for the situations covered by the tables.[5] The initial twist value assumed from studies in Ref. 5 is

$$\phi_{initial} = 0.436 \times 10^{-4}\,L \text{ radian} \tag{9}$$

Table 1

Section Dimensions	Force parallel to short leg ϕ_T positive		Force parallel to long leg ϕ_T positive	
in. x in.	λ_1	λ_2	λ_1	λ_2
6 x 4	1.49	−0.276	0.731	0.422
6 x 3½	1.73	−0.261	0.672	0.462
5 x 3½	1.42	−0.280	0.763	0.406
5 x 3	1.67	−0.263	0.682	0.450
4 x 3	1.32	−0.290	0.792	0.392
3½ x 3	1.15	−0.308	0.878	0.363
3½ x 2½	1.38	−0.283	0.786	0.404
3 x 2½	1.19	−0.303	0.858	0.368
3 x 2	1.50	−0.273	0.726	0.422
2½ x 2	1.23	−0.298	0.831	0.379

For all equal angles $\lambda_1 = 1.0$ $\quad \lambda_2 = -\frac{1}{3}$

The twist due to the applied torque T is given by TL/GK_T where G is the shear modulus and K_T the torsion constant. In assessing the value of T the tables include the effect of eccentricity of the beam self-weight and of applied load P relative to the shear centre.

A force parallel to one of the angle legs may be oriented to produce either tension or compression at the leg tip. The influence on the load carrying capacity of the orientation of loading varies with the loading condition. For angles subjected to loads in the short leg direction, Ref. 5 shows that the effect of reversing the load orientation is negligible. For loads in the long leg direction the tabulated values are chosen for the worst case and the maximum variation between load orientations is 6%.

AS CA1[1] states that the maximum shear stress F_v in a member shall not exceed $0.45\,F_y$ and hence the shear stress limits[8,9] are:

Force parallel to short leg

$$F_v = \frac{3P}{4Bt} + \frac{PQt}{4K_T} \leq 0.45\,F_y \qquad (10)$$

Force parallel to long leg

$$F_v = \frac{3P}{4Qt} + \frac{PBt}{4K_T} \leq 0.45\,F_y \qquad (11)$$

Where it is the shear stress limitation that governs, the tabulated loads are based on this and are indicated as those to the left of the heavy broken line in the tables.

The theoretical predictions given above and used in formulating the tables have been confirmed by an extensive test series on laterally unsupported angles.[10]

DEFLECTION EQUATIONS

The exact approach to this problem would involve the solution of a set of coupled partial differential equations for a variety of boundary conditions. The problem does not warrant the time involved in utilizing such a solution. A simplified analysis based on an extension of first order theory is used to find the maximum loading plane deflection

The total angle of twist (ϕ_T) is computed and the section rotated through this angle while the applied moment M_o retains its original direction (Fig. 2). The applied moment is then resolved into components about the closest rotated axes and the principal axes deflections due to each component determined using the appropriate equations from Fig. 2 and Ref. 5. Thus, loading plane deflections are calculated from:

Force parallel to short leg

$$x_{max} = \sqrt{u^2_{max} + v^2_{max}} \times \cos\left(\theta + \phi_T + \arctan\frac{v_{max}}{u_{max}}\right) \qquad (12)$$

Force parallel to long leg

$$y_{max} = \sqrt{u^2_{max} + v^2_{max}} \cdot \sin\left(\theta + \phi_T + \arctan\frac{v_{max}}{u_{max}}\right) \qquad (13)$$

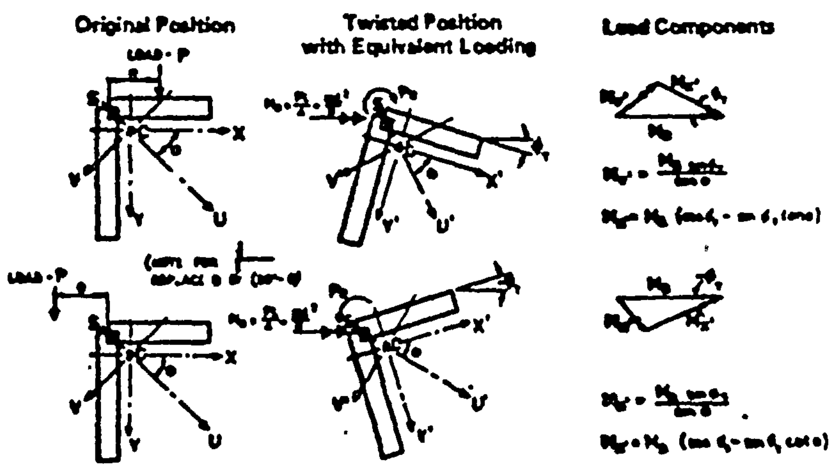


Fig. 2. Method of determination of load components for deflection equations

where u_{max} and v_{max} are respectively the summation of component deflections along the U and V axes. The tables use equations involving ϕ_T, however, reasonably accurate answers to cases where deflections are restricted can be obtained by calculating u_{max} and v_{max} from simple beam theory. If this simplification is used, then the applied moment, M_a, can be resolved directly into components about $U'U'$ and $V'V'$ (Fig. 2).

AS CA1, Appendix A, Sec. A2.2, recommends a deflection limit of:

$$\Delta = \frac{L}{180} \tag{14}$$

for structural applications where angle sections could be used. This limit is shown in the safe load tables as a heavy line, dividing the tables into regions above and below this limit. Deflection values to the left of this line are less than those recommended by AS CA1. Where possible, beam selection should be confined to this area of the tables.

The deflections for other loads may be estimated by proportions from the tables. However, as these include deadweight effects, a small adjustment must be made. For a load P'' which is less than the tabulated safe load P', the relevant deflection Δ'' can be calculated from the tabulated deflection Δ' as

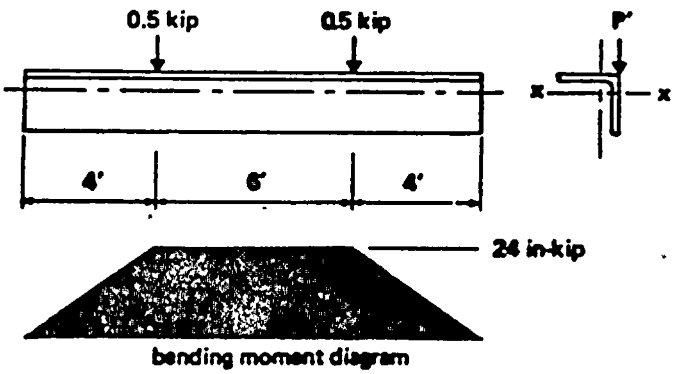
$$\Delta'' = \frac{P'' + mL}{P' + mL} \times \Delta \tag{15}$$

where mL is the beam weight.

Note that AS CA1 actually only requires live load deflections to be considered. However, the tables include dead load deflections as many angles otherwise included would suffer large visible deformations under their own weight and cause concern during fixing.

Example

The following simple example illustrates the use of the tables.



Data:

Steel	$F_y = 36$ ksi
Permissible Bending Stress	$0.66F_y$
Allowable maximum deflection	$\frac{L}{180} = 0.93''$
Loading Case:	For loads parallel to the long leg.

Maximum Bending Moment due to Applied Load:

$$M = 24 \text{ in.-kips}$$

Equivalent Uniformly Distributed Load:

$$P = \frac{8M}{L}$$

$$= 1.15 \text{ kips.}$$

The appropriate safe load tables are found on page 39 et seq for this loading and yield stress.

For a 14-ft span the smallest section capable of sustaining its maximum load whilst remaining within the permitted deflection limit is the 6 × 3½ × 5/16 angle.

However, use of Equation 15 will permit the use of many lighter sections. The lightest section that can be used is the 5 × 3 × 5/16. The table values for this 8.1 lb/ft section are $P' = 1.47$ kip and $\delta = 1.13$ inch.

Equation 15 gives

$$\Delta'' = \frac{1.15 + 0.0081 \times 14}{1.47 + 0.0081 \times 14} \times 1.13 = 0.90''$$

which satisfies the limit of 0.93".

REFERENCES

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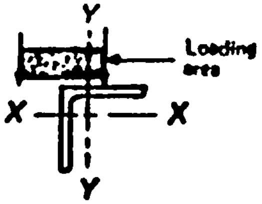
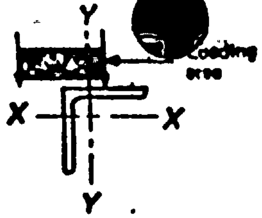


EQUAL ANGLES

HIGH STRENGTH STEEL (52 ksi)

BASED ON AS CA1 1968



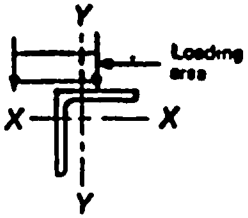
Nominal size	Thickness t nominal	Weight per foot	Safe distributed loads (kips) / max deflection (in) — Spans in feet: 2	4	6	8	10	12	14	20	25
in	in	lb	2	4	6	8	10	12	14	20	25
8 x 8	1	51.0	49.3 / 0.01	49.3 / 0.05	47.0 / 0.12	35.8 / 0.20	27.9 / 0.40	23.0 / 0.57	19.8 / 0.81	13.2 / 1.66	10.1 / 2.9
	7/8	45.0	38.8 / 0.01	38.8 / 0.04	38.8 / 0.16	31.1 / 0.29	24.7 / 0.46	20.4 / 0.66	17.4 / 0.91	11.7 / 1.66	8.96 / 2.9
	3/4	38.9	29.3 / 0.01	29.3 / 0.04	29.3 / 0.13	27.1 / 0.29	21.5 / 0.46	17.9 / 0.66	15.1 / 0.90	10.2 / 1.66	7.79 / 2.9
	5/8	32.7	21.0 / 0.00	21.0 / 0.03	21.0 / 0.11	21.0 / 0.27	18.2 / 0.46	15.0 / 0.67	12.7 / 0.91	8.65 / 1.87	6.51 / 2.9
	1/2	26.6	13.8 / 0.00	13.8 / 0.03	13.8 / 0.09	13.8 / 0.22	13.8 / 0.44	12.0 / 0.60	10.2 / 0.81	6.92 / 1.67	5.14 / 3.1
6 x 6	1	37.5	46.0 / 0.02	38.5 / 0.10	25.6 / 0.23	19.0 / 0.41	15.1 / 0.63	12.4 / 0.91	10.5 / 1.25	6.90 / 2.6	5.10 / 4.0
	7/8	33.1	39.3 / 0.02	34.6 / 0.10	22.9 / 0.23	17.0 / 0.40	13.6 / 0.63	11.1 / 0.91	9.42 / 1.24	6.07 / 2.6	4.66 / 4.0
	3/4	28.7	27.7 / 0.01	27.7 / 0.06	19.0 / 0.22	14.0 / 0.39	11.0 / 0.61	9.52 / 0.88	7.99 / 1.20	5.30 / 2.5	3.98 / 3.9
	5/8	24.2	20.0 / 0.01	20.0 / 0.06	16.0 / 0.22	12.4 / 0.39	9.81 / 0.61	8.09 / 0.88	6.84 / 1.20	4.40 / 2.6	3.38 / 3.9
	9/16	21.9	19.2 / 0.01	19.2 / 0.06	15.0 / 0.22	11.1 / 0.39	8.86 / 0.61	7.30 / 0.88	6.00 / 1.20	4.08 / 2.5	3.10 / 3.9
	1/2	19.6	13.3 / 0.01	13.3 / 0.07	13.3 / 0.22	10.0 / 0.39	7.97 / 0.61	6.57 / 0.88	5.49 / 1.20	3.64 / 2.5	2.74 / 3.9
	7/16	17.2	10.2 / 0.01	10.2 / 0.06	10.2 / 0.19	8.70 / 0.39	6.90 / 0.61	5.66 / 0.88	4.79 / 1.19	3.18 / 2.5	2.30 / 4.0
	3/8	14.8	7.77 / 0.01	7.77 / 0.06	7.77 / 0.17	7.00 / 0.39	5.56 / 0.61	4.90 / 0.88	4.15 / 1.20	2.75 / 2.5	2.08 / 4.0
5 x 5	3/4	23.6	20.5 / 0.02	20.4 / 0.12	13.8 / 0.27	10.1 / 0.48	7.90 / 0.70	6.56 / 1.00	5.53 / 1.40	3.83 / 3.0	2.81 / 4.8
	5/8	19.9	19.2 / 0.02	17.0 / 0.12	11.3 / 0.27	8.38 / 0.47	6.63 / 0.74	5.46 / 1.07	4.60 / 1.46	3.02 / 3.0	2.23 / 4.7
	1/2	16.1	12.9 / 0.01	12.9 / 0.11	9.20 / 0.28	6.84 / 0.47	5.41 / 0.73	4.46 / 1.05	3.78 / 1.45	2.47 / 3.0	1.79 / 4.7
	3/8	12.3	7.90 / 0.01	7.90 / 0.09	7.01 / 0.28	5.21 / 0.49	4.13 / 0.71	3.39 / 1.05	2.86 / 1.45	1.83 / 3.0	1.38 / 4.9
	5/16	10.4	5.41 / 0.01	5.41 / 0.07	5.41 / 0.24	4.37 / 0.47	3.46 / 0.74	2.86 / 1.07	2.40 / 1.49	1.53 / 3.1	1.09 / 5.1

Deflection values to the right of the heavy line are greater than 1/180 of the span.
Load values to the left of the broken line are based on shear capacity, and are less than the permissible flexural load.

Nominal size	Thickness t nominal	Weight per foot	Safe distributed loads (kips) / max deflection (in) — Spans in feet: 2	4	6	8	10	12	14	20	25
in	in	lb	2	4	6	8	10	12	14	20	25
4 x 4	5/8	15.7	19.2 / 0.03	10.8 / 0.15	7.14 / 0.34	5.30 / 0.61	4.19 / 0.95	3.43 / 1.37	2.82 / 1.86	1.81 / 3.9	1.29 / 6.0
	1/2	12.7	12.3 / 0.03	8.00 / 0.15	5.75 / 0.33	4.27 / 0.59	3.37 / 0.93	2.78 / 1.34	2.33 / 1.84	1.49 / 3.9	1.06 / 6.0
	3/8	9.7	7.33 / 0.02	6.66 / 0.14	4.43 / 0.33	3.29 / 0.59	2.60 / 0.92	2.13 / 1.34	1.79 / 1.87	1.15 / 3.9	0.80 / 6.0
	5/16	8.1	5.10 / 0.02	5.10 / 0.13	3.67 / 0.33	2.73 / 0.60	2.15 / 0.92	1.78 / 1.34	1.49 / 1.85	0.94 / 3.9	0.66 / 6.3
	1/4	6.6	3.46 / 0.02	3.46 / 0.11	3.00 / 0.33	2.22 / 0.59	1.78 / 0.94	1.44 / 1.37	1.19 / 1.87	0.73 / 4.0	0.51 / 6.7
3½ x 3½	5/8	13.6	16.2 / 0.04	8.08 / 0.17	5.33 / 0.39	3.96 / 0.70	3.11 / 1.10	2.54 / 1.60	2.13 / 2.2	1.30 / 4.4	0.82 / 7.0
	1/2	11.0	11.9 / 0.04	6.70 / 0.17	4.43 / 0.39	3.29 / 0.69	2.66 / 1.07	2.08 / 1.56	1.75 / 2.1	1.06 / 4.4	0.70 / 7.0
	3/8	8.4	7.16 / 0.03	5.13 / 0.17	3.39 / 0.39	2.51 / 0.68	1.98 / 1.06	1.62 / 1.54	1.36 / 2.1	0.84 / 4.3	0.59 / 7.1
	5/16	7.1	5.60 / 0.02	4.38 / 0.17	2.83 / 0.38	2.10 / 0.68	1.66 / 1.07	1.37 / 1.52	1.11 / 2.1	0.69 / 4.4	0.49 / 7.2
	1/4	5.7	3.40 / 0.02	3.40 / 0.16	2.33 / 0.38	1.69 / 0.67	1.32 / 1.05	1.09 / 1.54	0.89 / 2.1	0.56 / 4.0	0.38 / 7.0
3 x 3	5/8	11.4	11.6 / 0.06	5.72 / 0.21	3.77 / 0.47	2.79 / 0.83	2.19 / 1.30	1.79 / 1.87	1.49 / 2.6	0.91 / 5.3	0.61 / 8.2
	1/2	9.3	9.04 / 0.05	4.79 / 0.20	3.16 / 0.46	2.34 / 0.81	1.84 / 1.28	1.47 / 1.83	1.22 / 2.6	0.75 / 5.2	0.51 / 8.2
	3/8	7.1	6.93 / 0.05	3.67 / 0.20	2.42 / 0.44	1.79 / 0.79	1.41 / 1.28	1.16 / 1.81	0.96 / 2.5	0.59 / 5.2	0.40 / 8.2
	5/16	6.0	4.62 / 0.04	3.07 / 0.19	2.03 / 0.44	1.50 / 0.79	1.18 / 1.24	0.96 / 1.80	0.80 / 2.6	0.49 / 5.1	0.32 / 8.3
	1/4	4.8	3.37 / 0.03	2.53 / 0.19	1.67 / 0.44	1.23 / 0.79	0.97 / 1.24	0.79 / 1.82	0.66 / 2.5	0.39 / 5.4	0.28 / 8.7
	3/16	3.7	1.96 / 0.03	1.92 / 0.19	1.27 / 0.44	0.94 / 0.80	0.73 / 1.27	0.59 / 1.84	0.47 / 2.6	0.27 / 5.8	0.17 / 9.0

The tables are calculated for loads applied anywhere within half a leg length on either side of the shear centre—see fig 1

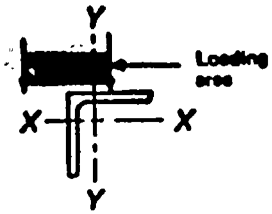
Safe Load Tables for Laterally Unsupported Angles
Normal-Strength Steel—36 ksi yield stress



EQUAL ANGLES

HIGH STRENGTH STEEL (52 ksi)

BASED ON AS CA1 1968

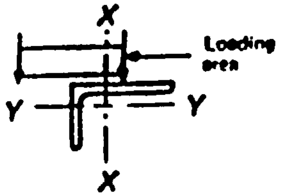


NOMINAL SIZE	THICKNESS t NOMINAL	WEIGHT PER FOOT	SAFE DISTRIBUTED LOADS (KIPS) / MAX DEFLECTION (in) SPANS IN FEET								
in	in	lb	2	4	6	8	10	12	14	20	25
3½ x 3½	[illegible]	7.8	6.46 / 0.09	3.30 / 0.35	2.11 / 0.80	1.66 / 0.90	1.22 / 1.66	0.88 / 2.3	0.79 / 3.1	0.47 / 6.3	
	[illegible]	5.9	5.11 / 0.09	2.84 / 0.34	1.87 / 0.56	1.20 / 0.90	0.94 / 1.52	0.78 / 2.2	0.63 / 3.0	0.37 / 6.1	
	[illegible]	5.0	4.20 / 0.08	2.00 / 0.24	1.37 / 0.53	1.01 / 0.90	0.79 / 1.61	0.64 / 2.2	0.53 / 3.0	0.31 / 6.2	
	[illegible]	4.0	3.22 / 0.06	1.73 / 0.23	1.14 / 0.53	0.83 / 0.86	0.88 / 1.50	0.53 / 2.2	0.43 / 3.0	0.24 / 6.4	
	[illegible]	3.1	1.81 / 0.04	1.32 / 0.23	0.88 / 0.53	0.64 / 0.84	0.48 / 1.40	0.40 / 2.2	0.32 / 3.0	0.19 / 6.5	
2½ x 2½	[illegible]	6.2	4.88 / 0.07	2.82 / 0.27	1.33 / 0.61	0.98 / 1.10	0.78 / 1.70	0.61 / 2.5	0.48 / 3.4	0.29 / 7.0	
	[illegible]	4.4	3.46 / 0.07	1.71 / 0.27	1.11 / 0.60	0.82 / 1.07	0.64 / 1.66	0.52 / 2.5	0.42 / 3.3	0.24 / 6.9	
	[illegible]	3.6	2.81 / 0.07	1.38 / 0.28	0.91 / 0.58	0.67 / 1.07	0.52 / 1.66	0.42 / 2.4	0.34 / 3.3	0.20 / 6.9	
	[illegible]	2.7	1.88 / 0.06	1.06 / 0.26	0.70 / 0.58	0.51 / 1.06	0.40 / 1.67	0.32 / 2.4	0.26 / 3.3	0.14 / 7.1	
2 x 2	[illegible]	4.8	3.14 / 0.08	1.66 / 0.31	1.02 / 0.66	0.78 / 1.24	0.67 / 1.93	0.48 / 2.8	0.37 / 3.8	0.21 / 7.9	
	[illegible]	3.9	2.67 / 0.08	1.33 / 0.30	0.87 / 0.66	0.63 / 1.22	0.49 / 1.92	0.38 / 2.8	0.32 / 3.8	0.18 / 8.0	
	[illegible]	3.2	2.19 / 0.07	1.08 / 0.30	0.71 / 0.67	0.52 / 1.21	0.41 / 1.90	0.32 / 2.8	0.26 / 3.8	0.15 / 8.1	
	[illegible]	2.4	1.60 / 0.07	0.84 / 0.29	0.54 / 0.67	0.40 / 1.19	0.31 / 1.90	0.25 / 2.8	0.20 / 3.8	0.11 / 8.1	
	[illegible]	1.7	0.86 / 0.05	0.56 / 0.29	0.37 / 0.67	0.28 / 1.22	0.20 / 2.0	0.16 / 2.8	0.13 / 4.0		

Deflection values to the right of the heavy line are greater than 1/180 of the span.

NOMINAL SIZE	THICKNESS t NOMINAL	WEIGHT PER FOOT	SAFE DISTRIBUTED LOADS (KIPS) / MAX DEFLECTION (in) SPANS IN FEET								
in	in	lb	2	4	6	8	10	12	14	20	25
1¾ x 1¾	[illegible]	4.0	2.33 / 0.08	1.15 / 0.38	0.78 / 0.80	0.66 / 1.43	0.43 / 2.3	0.33 / 3.3	0.27 / 4.4	0.14 / 8.1	
	[illegible]	3.4	2.00 / 0.08	0.98 / 0.36	0.66 / 0.79	0.48 / 1.41	0.38 / 2.2	0.29 / 3.2	0.23 / 4.3	0.12 / 8.0	
	[illegible]	2.7	1.66 / 0.09	0.83 / 0.33	0.53 / 0.77	0.38 / 1.38	0.30 / 2.2	0.24 / 3.1	0.19 / 4.3	0.10 / 8.2	
	[illegible]	2.1	1.28 / 0.08	0.64 / 0.34	0.41 / 0.77	0.30 / 1.37	0.23 / 2.2	0.18 / 3.2	0.15 / 4.4		
	[illegible]	1.6	0.86 / 0.08	0.43 / 0.33	0.28 / 0.77	0.20 / 1.37	0.15 / 2.3	0.12 / 3.3			
1½ x 1½	[illegible]	2.86	1.41 / 0.10	0.70 / 0.41	0.48 / 0.94	0.33 / 1.67	0.26 / 2.6	0.20 / 3.8	0.16 / 5.1		
	[illegible]	2.38	1.20 / 0.10	0.60 / 0.41	0.38 / 0.92	0.28 / 1.64	0.21 / 2.6	0.17 / 3.7	0.13 / 5.1		
	[illegible]	1.8	0.82 / 0.10	0.46 / 0.40	0.30 / 0.91	0.22 / 1.62	0.17 / 2.6	0.13 / 3.8	0.10 / 5.2		
	[illegible]	1.2	0.63 / 0.10	0.31 / 0.39	0.20 / 0.91	0.14 / 1.63	0.11 / 2.6				
1¼ x 1¼	[illegible]	2.3	0.96 / 0.13	0.47 / 0.51	0.30 / 1.15	0.22 / 2.1	0.16 / 3.2	0.13 / 4.7	0.10 / 6.3		
	[illegible]	1.9	0.80 / 0.12	0.40 / 0.50	0.26 / 1.12	0.18 / 2.0	0.14 / 3.2	0.11 / 4.5			
	[illegible]	1.46	0.62 / 0.12	0.30 / 0.48	0.20 / 1.10	0.14 / 2.0	0.11 / 3.1				
	[illegible]	1.0	0.44 / 0.12	0.22 / 0.48	0.14 / 1.10	0.10 / 2.0					
1 x 1	[illegible]	1.5	0.49 / 0.18	0.24 / 0.64	0.15 / 1.44	0.11 / 2.6					
	[illegible]	1.15	0.38 / 0.15	0.19 / 0.62	0.12 / 1.40						
	[illegible]	0.8	0.27 / 0.15	0.13 / 0.60							

The tables are calculated for loads applied anywhere within half a leg length on either side of the shear centre—see fig. 1.

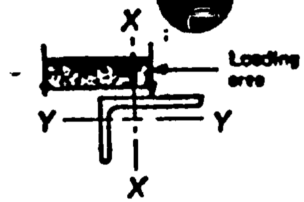


UNEQUAL ANGLES

Force parallel to SHORT leg

HIGH STRENGTH STEEL (52 ksi)

BASED ON AS CA1 1968



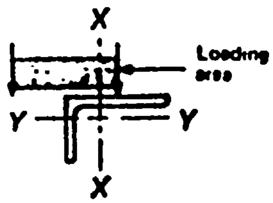
NOMINAL SIZE	THICKNESS t NOMINAL	WEIGHT PER FOOT	SAFE DISTRIBUTED LOADS (KIPS) / MAX DEFLECTION (in) SPANS IN FEET								
in	in	lb	2	4	6	8	10	12	14	20	25
6 x 4	[illegible]	23.0	21.4 / 0.03	13.2 / 0.14	8.89 / 0.12	6.44 / 0.58	5.08 / 0.91	4.13 / 1.31	3.48 / 1.78	2.18 / 3.7	1.83 / 5.3
	[illegible]	19.9	15.6 / 0.03	11.3 / 0.14	7.45 / 0.32	5.52 / 0.58	4.35 / 0.90	3.65 / 1.31	2.87 / 1.78	1.97 / 3.7	1.32 / 5.8
	[illegible]	16.1	10.4 / 0.02	9.28 / 0.15	6.13 / 0.31	4.64 / 0.58	3.87 / 0.92	2.92 / 1.32	2.44 / 1.81	1.66 / 3.8	1.09 / 6.0
	[illegible]	14.2	8.17 / 0.02	8.17 / 0.15	5.46 / 0.33	4.04 / 0.58	3.18 / 0.91	2.60 / 1.35	2.17 / 1.85	1.38 / 3.9	0.98 / 6.2
	[illegible]	12.3	6.19 / 0.02	6.19 / 0.13	4.78 / 0.34	3.53 / 0.61	2.78 / 0.95	2.27 / 1.38	1.90 / 1.92	1.21 / 4.1	0.85 / 6.6
6 x 3½	[illegible]	18.8	14.8 / 0.03	8.84 / 0.18	5.89 / 0.37	4.20 / 0.66	3.29 / 1.01	2.68 / 1.48	2.22 / 2.0	1.38 / 4.2	0.92 / 6.6
	[illegible]	15.3	9.73 / 0.03	7.16 / 0.18	4.72 / 0.37	3.49 / 0.66	2.73 / 1.04	2.22 / 1.51	1.85 / 2.1	1.14 / 4.3	0.77 / 6.8
	[illegible]	13.8	7.78 / 0.02	6.30 / 0.17	4.21 / 0.38	3.11 / 0.67	2.44 / 1.08	1.99 / 1.53	1.66 / 2.1	1.02 / 4.4	0.69 / 7.1
	[illegible]	11.8	5.73 / 0.02	5.52 / 0.17	3.64 / 0.39	2.69 / 0.69	2.11 / 1.09	1.72 / 1.58	1.43 / 2.2	0.88 / 4.6	0.60 / 7.4
	[illegible]	9.7	4.07 / 0.02	4.07 / 0.16	3.07 / 0.41	2.27 / 0.73	1.78 / 1.15	1.45 / 1.67	1.20 / 2.3	0.74 / 4.9	0.51 / 8.1
5 x 3½	[illegible]	16.7	15.2 / 0.04	8.36 / 0.17	5.51 / 0.37	4.08 / 0.65	3.20 / 1.04	2.61 / 1.50	2.17 / 2.0	1.36 / 4.2	0.93 / 6.7
	[illegible]	13.8	10.4 / 0.03	6.95 / 0.17	4.59 / 0.37	3.39 / 0.66	2.67 / 1.04	2.17 / 1.51	1.81 / 2.1	1.13 / 4.3	0.78 / 6.9
	[illegible]	10.3	8.10 / 0.02	5.38 / 0.17	3.55 / 0.38	2.61 / 0.68	2.08 / 1.07	1.68 / 1.56	1.40 / 2.1	0.88 / 4.5	0.61 / 7.4
	[illegible]	8.7	4.38 / 0.02	4.38 / 0.18	3.02 / 0.38	2.24 / 0.71	1.78 / 1.11	1.43 / 1.62	1.20 / 2.2	0.75 / 4.8	0.52 / 7.9

Deflection values to the right of the heavy line are greater than 1/180 of the span.
Load values to the left of the broken line are based on shear capacity, and are less than the permissible flexural load.

NOMINAL SIZE	THICKNESS t NOMINAL	WEIGHT PER FOOT	SAFE DISTRIBUTED LOADS (KIPS) / MAX DEFLECTION (in) SPANS IN FEET								
in	in	lb	2	4	6	8	10	12	14	20	25
6 x 3	[illegible]	12.7	8.44 / 0.05	6.10 / 0.19	3.38 / 0.41	2.47 / 0.77	1.93 / 1.21	1.58 / 1.74	1.29 / 2.4	0.77 / 5.0	0.50 / 7.9
	[illegible]	11.2	7.40 / 0.04	4.05 / 0.19	2.98 / 0.43	2.21 / 0.77	1.72 / 1.21	1.39 / 1.76	1.16 / 2.4	0.69 / 5.0	0.46 / 8.1
	[illegible]	9.7	5.66 / 0.04	3.98 / 0.20	2.62 / 0.44	1.93 / 0.79	1.51 / 1.24	1.22 / 1.79	1.01 / 2.5	0.61 / 5.2	0.40 / 8.4
	[illegible]	8.1	4.01 / 0.03	3.38 / 0.20	2.21 / 0.45	1.63 / 0.81	1.28 / 1.28	1.03 / 1.88	0.86 / 2.6	0.51 / 5.5	0.34 / 9.0
	[illegible]	6.6	2.72 / 0.02	2.72 / 0.21	1.82 / 0.48	1.34 / 0.86	1.06 / 1.38	0.86 / 2.0	0.71 / 2.8	0.43 / 6.1	0.28 / 10.2
4 x 3	[illegible]	13.6	12.3 / 0.05	6.10 / 0.20	4.02 / 0.44	2.97 / 0.79	2.32 / 1.24	1.88 / 1.78	1.58 / 2.4	0.96 / 5.0	0.63 / 8.0
	[illegible]	11.8	9.89 / 0.05	4.91 / 0.19	3.24 / 0.44	2.38 / 0.78	1.87 / 1.22	1.52 / 1.77	1.26 / 2.4	0.77 / 5.1	0.52 / 8.1
	[illegible]	9.4	6.67 / 0.04	3.84 / 0.20	2.53 / 0.44	1.87 / 0.79	1.47 / 1.24	1.19 / 1.80	0.99 / 2.5	0.61 / 5.2	0.41 / 8.6
	[illegible]	7.1	4.41 / 0.03	3.29 / 0.20	2.17 / 0.45	1.60 / 0.80	1.26 / 1.26	1.02 / 1.84	0.85 / 2.5	0.52 / 5.4	0.36 / 8.8
	[illegible]	5.8	2.96 / 0.01	2.70 / 0.20	1.78 / 0.46	1.32 / 0.83	1.03 / 1.32	0.84 / 1.93	0.70 / 2.7	0.41 / 5.6	0.27 / 9.3
3½ x 3	[illegible]	10.2	8.78 / 0.05	4.88 / 0.20	3.21 / 0.45	2.37 / 0.78	1.88 / 1.26	1.51 / 1.80	1.26 / 2.5	0.78 / 5.1	0.53 / 8.7
	[illegible]	7.8	6.46 / 0.04	3.80 / 0.20	2.51 / 0.44	1.86 / 0.79	1.46 / 1.25	1.18 / 1.81	0.99 / 2.5	0.60 / 5.1	0.40 / 8.7
	[illegible]	6.5	4.50 / 0.04	3.21 / 0.20	2.12 / 0.45	1.57 / 0.80	1.23 / 1.26	1.00 / 1.84	0.81 / 2.5	0.50 / 5.2	0.33 / 8.5
	[illegible]	5.3	3.08 / 0.01	2.64 / 0.20	1.69 / 0.44	1.25 / 0.80	0.98 / 1.28	0.79 / 1.83	0.66 / 2.5	0.40 / 5.4	0.26 / 8.9

The tables are calculated for loads applied anywhere within half a leg length on either side of the shear centre–see fig 1.

Safe Load Tables for Laterally Unsupported Angles



UNEQUAL ANGLES

Force parallel to SHORT leg

HIGH STRENGTH STEEL (52 ksi)

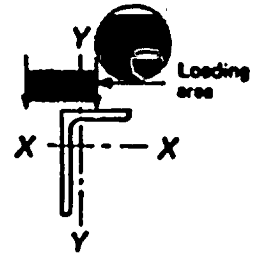
BASED ON AS CA1 1968

NOMINAL SIZE (in)	THICKNESS t NOMINAL (in)	WEIGHT PER FOOT (lb)	2	4	6	8	10	12	14	20	26
			SAFE DISTRIBUTED LOADS (KIPS) / MAX DEFLECTION (in) — SPANS IN FEET								
3½ x 2½	[illegible]	9.4	6.73 0.08	3.33 0.23	2.19 0.53	1.61 0.94	1.28 1.47	1.00 2.1	0.83 2.9	0.40 6.1	0.30 9.7
	[illegible]	7.1	5.26 0.08	2.88 0.23	1.71 0.52	1.26 0.94	0.98 1.47	0.79 2.1	0.66 2.9	0.38 6.2	0.24 10.1
	[illegible]	8.8	4.14 0.08	2.23 0.23	1.47 0.53	1.08 0.95	0.84 1.48	0.68 2.2	0.58 3.0	0.33 6.4	0.21 10.8
	[illegible]	4.8	2.73 0.06	1.82 0.24	1.20 0.56	0.88 0.98	0.69 1.56	0.58 2.3	0.46 3.2	0.26 6.7	0.18 11.2
	[illegible]	3.7	1.86 0.03	1.43 0.25	0.94 0.58	0.68 1.05	0.54 1.68	0.44 2.5	0.36 3.4	0.20 7.5	0.12 12.0
3 x 2½	[illegible]	8.5	6.88 0.08	3.28 0.24	2.15 0.54	1.58 0.96	1.23 1.51	1.00 2.2	0.82 3.0	0.40 6.2	0.32 9.9
	[illegible]	6.5	5.17 0.08	2.67 0.24	1.88 0.53	1.26 0.95	0.97 1.50	0.78 2.2	0.66 3.0	0.30 6.3	0.24 9.9
	[illegible]	6.6	4.43 0.08	2.20 0.24	1.46 0.53	1.07 0.95	0.84 1.51	0.68 2.2	0.56 3.0	0.32 6.1	0.21 10.2
	[illegible]	4.4	2.90 0.06	1.70 0.24	1.10 0.54	0.87 0.97	0.68 1.54	0.53 2.2	0.44 3.0	0.26 6.6	0.18 10.9
	[illegible]	3.4	1.78 0.04	1.38 0.24	0.90 0.54	0.66 0.99	0.51 1.56	0.41 2.3	0.34 3.2	0.20 7.1	0.12 11.7
3 x 2	[illegible]	7.7	4.13 0.07	2.04 0.29	1.33 0.66	0.97 1.19	0.75 1.86	0.60 2.7	0.48 3.7	0.26 7.7	0.14 12.2
	[illegible]	5.9	3.38 0.07	1.63 0.29	1.07 0.66	0.78 1.17	0.60 1.83	0.48 2.7	0.39 3.7	0.21 7.7	0.12 12.4
	[illegible]	5.0	2.84 0.07	1.40 0.29	0.92 0.66	0.67 1.17	0.52 1.85	0.42 2.7	0.34 3.7	0.18 7.9	0.10 13.0
	[illegible]	4.0	2.32 0.07	1.15 0.29	0.75 0.66	0.56 1.19	0.42 1.86	0.34 2.8	0.27 3.8	0.15 8.4	
	[illegible]	3.1	1.56 0.07	0.90 0.31	0.58 0.70	0.43 1.26	0.33 2.02	0.27 3.0	0.22 4.2	0.12 9.3	
2½ x 2	[illegible]	5.2	3.10 0.07	1.57 0.30	1.03 0.67	0.76 1.19	0.58 1.87	0.47 2.7	0.38 3.7	0.21 7.8	0.12 12.9
	[illegible]	4.4	2.72 0.07	1.36 0.29	0.88 0.67	0.66 1.19	0.50 1.88	0.40 2.7	0.33 3.8	0.19 8.1	0.10 12.9
	[illegible]	3.6	2.26 0.07	1.12 0.29	0.73 0.67	0.54 1.20	0.42 1.90	0.34 2.8	0.27 3.9	0.15 8.3	
	[illegible]	2.7	1.57 0.07	0.86 0.30	0.56 0.69	0.40 1.21	0.31 1.96	0.25 2.9	0.20 4.0	0.11 9.1	

Deflection values to the right of the heavy line are greater than 1/180 of the span
Load values to the left of the broken line are based on shear capacity, and are less than the permissible flexural load

HIGH STRENGTH STEEL (52 ksi)

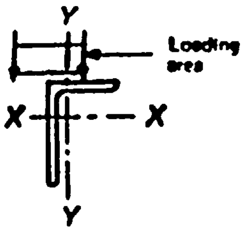
BASED ON AS CA1 1968



NOMINAL SIZE (in)	THICKNESS t NOMINAL (in)	WEIGHT PER FOOT (lb)	2	4	6	8	10	12	14	20	26
			SAFE DISTRIBUTED LOADS (KIPS) / MAX DEFLECTION (in) — SPANS IN FEET								
8 x 4	[illegible]	23.6	33.3 0.02	27.3 0.10	18.1 0.23	13.6 0.42	10.7 0.66	8.84 0.96	7.40 1.30	6.00 2.7	3.71 4.3
	[illegible]	19.9	24.0 0.02	23.1 0.10	18.3 0.23	11.4 0.41	9.07 0.66	7.40 0.94	6.34 1.30	4.10 2.7	3.12 4.4
	[illegible]	16.1	16.0 0.01	16.0 0.08	12.4 0.23	9.26 0.41	7.38 0.66	6.08 0.96	5.14 1.32	3.36 2.7	2.49 4.4
	[illegible]	14.2	12.6 0.01	12.6 0.08	10.9 0.23	8.14 0.42	6.48 0.66	5.34 0.97	4.38 1.32	2.82 2.8	2.09 4.7
	[illegible]	12.3	9.43 0.01	9.43 0.07	9.42 0.23	7.62 0.42	6.41 0.66	4.44 0.97	3.32 1.31	2.37 2.9	1.71 4.8
8 x 3½	[illegible]	18.8	20.4 0.02	22.0 0.10	15.0 0.23	11.1 0.42	8.88 0.66	7.34 0.97	6.10 1.34	3.07 2.9	2.98 4.8
	[illegible]	15.3	17.2 0.01	17.2 0.08	12.1 0.23	8.98 0.42	7.11 0.66	5.87 0.97	4.98 1.35	3.14 2.8	2.28 4.8
	[illegible]	13.6	13.6 0.01	13.6 0.08	10.6 0.23	7.90 0.42	6.27 0.67	5.17 0.98	4.24 1.34	2.71 2.9	1.93 5.1
	[illegible]	11.8	10.1 0.01	10.1 0.07	9.03 0.24	6.73 0.43	5.34 0.68	4.28 0.98	3.86 1.38	2.21 3.1	1.64 5.2
	[illegible]	9.7	7.10 0.01	7.10 0.07	7.10 0.23	6.40 0.43	4.23 0.68	3.42 1.03	2.82 1.47	1.88 3.3	1.14 5.2
6 x 3½	[illegible]	16.7	22.4 0.02	15.8 0.12	10.5 0.20	7.88 0.50	6.90 0.79	5.08 1.12	4.27 1.64	2.79 3.2	2.07 5.2
	[illegible]	13.8	15.1 0.02	13.8 0.12	8.67 0.20	6.38 0.50	5.03 0.77	4.17 1.11	3.48 1.63	2.30 3.3	1.63 5.2
	[illegible]	10.3	8.86 0.01	8.86 0.11	6.48 0.20	4.79 0.40	3.78 0.77	3.08 1.13	2.00 1.57	1.64 3.3	1.16 5.5
	[illegible]	8.7	6.36 0.01	6.36 0.09	5.30 0.20	3.97 0.40	3.13 0.79	2.57 1.10	2.17 1.63	1.32 3.6	0.81 5.8

The tables are calculated for loads applied anywhere within half a leg length on either side of the shear centre—see fig. 1.

Safe Load Tables for Laterally Unsupported Angles



UNEQUAL ANGLES

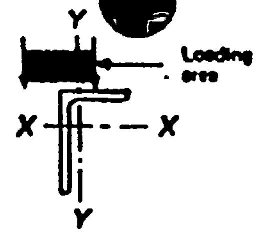
Force parallel to LONG leg

NOMINAL SIZE	THICKNESS / NOMINAL	WEIGHT PER FOOT	SAFE DISTRIBUTED LOADS (KIPS) / MAX DEFLECTION (in) — SPANS IN FEET								
			2	4	6	8	10	12	14	20	25
in	in	lb									
5 x 3	1/2	12.7	10.3 / 0.02	12.8 / 0.12	8.31 / 0.29	6.10 / 0.51	4.87 / 0.79	3.96 / 1.15	3.34 / 1.56	2.17 / 3.8	1.53 / 5.8
	7/16	11.2	12.8 / 0.02	11.1 / 0.12	7.33 / 0.29	5.44 / 0.51	4.29 / 0.79	3.50 / 1.18	2.93 / 1.61	1.86 / 3.5	1.31 / 5.8
	3/8	9.7	9.68 / 0.02	9.84 / 0.12	6.30 / 0.29	4.66 / 0.51	3.66 / 0.80	2.98 / 1.19	2.48 / 1.66	1.54 / 3.7	1.07 / 6.1
	5/16	8.1	8.83 / 0.01	8.83 / 0.11	5.20 / 0.29	3.82 / 0.51	2.99 / 0.82	2.41 / 1.20	2.04 / 1.74	1.20 / 3.9	0.80 / 6.2
	1/4	6.6	4.82 / 0.01	4.82 / 0.10	4.14 / 0.29	3.01 / 0.52	2.34 / 0.86	1.87 / 1.32	1.57 / 1.86	0.96 / 4.0	0.66 / 6.4
4 x 3	5/8	13.6	16.1 / 0.04	10.6 / 0.18	7.04 / 0.36	5.00 / 0.63	4.01 / 0.99	3.29 / 1.43	2.78 / 1.95	1.78 / 4.0	1.27 / 6.4
	1/2	11.0	13.8 / 0.03	8.29 / 0.18	5.41 / 0.35	4.02 / 0.62	3.18 / 0.98	2.60 / 1.39	2.17 / 1.93	1.38 / 4.0	0.90 / 6.6
	3/8	8.4	8.23 / 0.03	6.29 / 0.15	4.14 / 0.34	3.06 / 0.62	2.42 / 0.96	1.97 / 1.40	1.66 / 1.93	1.04 / 4.2	0.73 / 6.7
	5/16	7.1	6.66 / 0.02	5.29 / 0.15	3.56 / 0.34	2.56 / 0.61	2.03 / 0.97	1.66 / 1.41	1.37 / 1.94	0.86 / 4.2	0.58 / 7.0
	1/4	5.8	3.96 / 0.02	3.96 / 0.14	2.81 / 0.34	2.07 / 0.62	1.62 / 0.93	1.31 / 1.44	1.08 / 2.0	0.64 / 4.5	0.42 / 7.0
3½ x 3	1/2	10.2	12.7 / 0.04	6.90 / 0.18	4.30 / 0.40	3.11 / 0.70	2.46 / 1.08	1.98 / 1.56	1.67 / 2.2	1.05 / 4.4	0.74 / 7.1
	3/8	7.8	7.90 / 0.03	4.88 / 0.17	3.23 / 0.38	2.40 / 0.68	1.88 / 1.09	1.54 / 1.56	1.28 / 2.2	0.80 / 4.5	0.56 / 7.4
	5/16	6.6	5.37 / 0.03	4.08 / 0.17	2.70 / 0.38	2.00 / 0.68	1.57 / 1.07	1.27 / 1.56	1.06 / 2.2	0.66 / 4.6	0.45 / 7.4
	1/4	5.3	3.80 / 0.02	3.32 / 0.17	2.20 / 0.38	1.61 / 0.68	1.28 / 1.08	1.02 / 1.56	0.86 / 2.2	0.51 / 4.6	0.34 / 7.8

Deflection values to the right of the heavy line are greater than 1/180 of the span.
Load values to the left of the broken line are based on shear capacity, and are less than the permissible flexural load.

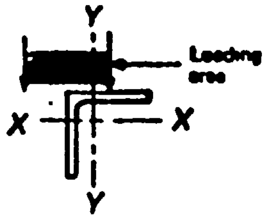
HIGH STRENGTH STEEL (52 ksi)

BASED ON AS CA1 1968



NOMINAL SIZE	THICKNESS / NOMINAL	WEIGHT PER FOOT	SAFE DISTRIBUTED LOADS (KIPS) / MAX DEFLECTION (in) — SPANS IN FEET								
			2	4	6	8	10	12	14	20	25
in	in	lb									
3½ x 2½	1/2	9.4	12.7 / 0.04	6.17 / 0.18	4.06 / 0.40	3.01 / 0.72	2.38 / 1.14	1.93 / 1.64	1.61 / 2.3	1.02 / 4.7	0.71 / 7.6
	3/8	7.1	8.17 / 0.03	4.73 / 0.18	3.19 / 0.40	2.30 / 0.71	1.80 / 1.11	1.46 / 1.64	1.22 / 2.2	0.76 / 4.8	0.51 / 7.6
	5/16	6.0	5.90 / 0.03	3.97 / 0.18	2.62 / 0.39	1.93 / 0.71	1.51 / 1.12	1.22 / 1.63	1.01 / 2.3	0.61 / 5.0	0.41 / 8.0
	1/4	4.8	3.80 / 0.02	3.19 / 0.17	2.00 / 0.40	1.53 / 0.71	1.19 / 1.14	0.96 / 1.60	0.79 / 2.4	0.48 / 5.3	0.29 / 8.2
	3/16	3.7	2.36 / 0.02	2.38 / 0.17	1.66 / 0.40	1.14 / 0.74	0.89 / 1.22	0.70 / 1.69	0.57 / 2.6	0.30 / 5.2	0.17 / 8.2
3 x 2½	1/2	8.5	9.33 / 0.05	4.64 / 0.21	3.06 / 0.47	2.27 / 0.84	1.74 / 1.31	1.42 / 1.90	1.17 / 2.6	0.73 / 5.3	0.50 / 8.4
	3/8	6.6	7.00 / 0.04	3.83 / 0.20	2.33 / 0.46	1.71 / 0.82	1.34 / 1.29	1.06 / 1.86	0.91 / 2.6	0.56 / 5.4	0.37 / 8.6
	5/16	5.5	5.30 / 0.04	3.00 / 0.20	1.96 / 0.46	1.46 / 0.82	1.10 / 1.27	0.92 / 1.80	0.76 / 2.6	0.47 / 5.4	0.31 / 8.8
	1/4	4.4	3.81 / 0.04	2.41 / 0.20	1.57 / 0.46	1.17 / 0.82	0.90 / 1.26	0.73 / 1.80	0.60 / 2.6	0.38 / 5.6	0.23 / 9.1
	3/16	3.4	2.13 / 0.03	1.83 / 0.20	1.21 / 0.40	0.88 / 0.82	0.68 / 1.33	0.56 / 1.93	0.46 / 2.7	0.26 / 5.8	0.16 / 9.1
3 x 2	1/2	7.7	8.86 / 0.06	4.47 / 0.22	2.87 / 0.48	2.12 / 0.86	1.67 / 1.36	1.36 / 1.94	1.13 / 2.7	0.70 / 5.7	0.47 / 9.2
	3/8	5.9	6.84 / 0.06	3.46 / 0.21	2.22 / 0.47	1.66 / 0.84	1.29 / 1.35	1.04 / 1.95	0.86 / 2.7	0.52 / 5.8	0.34 / 9.9
	5/16	5.0	5.03 / 0.05	2.86 / 0.21	1.86 / 0.47	1.40 / 0.84	1.06 / 1.33	0.87 / 1.96	0.72 / 2.7	0.43 / 6.1	0.27 / 10.1
	1/4	4.0	3.80 / 0.04	2.29 / 0.21	1.51 / 0.47	1.10 / 0.86	0.86 / 1.30	0.69 / 2.0	0.56 / 2.9	0.31 / 6.3	0.19 / 10.0
	3/16	3.1	2.37 / 0.03	1.78 / 0.21	1.13 / 0.46	0.82 / 0.88	0.63 / 1.46	0.50 / 2.2	0.40 / 3.3	0.20 / 6.5	0.11 / 10.2
2½ x 2	3/8	5.2	4.83 / 0.05	2.46 / 0.25	1.64 / 0.56	1.14 / 1.00	0.88 / 1.57	0.71 / 2.3	0.58 / 3.1	0.35 / 6.6	0.23 / 10.9
	5/16	4.4	4.01 / 0.05	1.98 / 0.24	1.31 / 0.56	0.97 / 0.99	0.76 / 1.57	0.61 / 2.3	0.50 / 3.2	0.29 / 6.8	0.18 / 10.9
	1/4	3.6	3.70 / 0.04	1.63 / 0.24	1.08 / 0.55	0.78 / 0.99	0.61 / 1.58	0.50 / 2.3	0.40 / 3.2	0.23 / 7.1	0.14 / 11.0
	3/16	2.7	1.86 / 0.04	1.22 / 0.24	0.80 / 0.56	0.60 / 1.01	0.46 / 1.64	0.36 / 2.5	0.29 / 3.3	0.15 / 7.0	

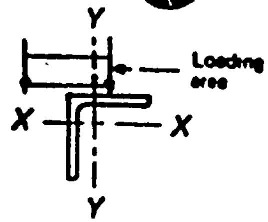
The tables are calculated for loads applied anywhere within half a leg length on either side of the shear centre—see fig. 1.



EQUAL ANGLES

NORMAL STRENGTH STEEL (36 ksi)

BASED ON AS CA1 1968

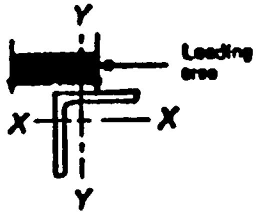


NOMINAL SIZE	THICKNESS t NOMINAL	WEIGHT PER FOOT	SAFE DISTRIBUTED LOADS (KIPS) / MAX DEFLECTION (in) — SPANS IN FEET								
in	in	lb	2	4	6	8	10	12	14	20	25
8 x 8	1	51.0	34.1 / 0.00	34.1 / 0.00	28.9 / 0.12	24.6 / 0.20	19.4 / 0.32	16.0 / 0.46	13.6 / 0.63	8.03 / 1.30	6.00 / 2.0
	[illegible]	45.0	36.9 / 0.00	36.9 / 0.00	28.9 / 0.11	21.6 / 0.20	17.3 / 0.32	14.2 / 0.40	12.0 / 0.63	7.96 / 1.30	6.06 / 2.0
	[illegible]	38.9	30.3 / 0.00	30.3 / 0.02	20.3 / 0.10	19.0 / 0.20	16.0 / 0.32	12.4 / 0.46	10.6 / 0.63	6.94 / 1.30	5.06 / 1.90
	[illegible]	32.7	14.8 / 0.00	14.8 / 0.00	14.8 / 0.00	14.8 / 0.19	12.4 / 0.31	10.2 / 0.46	8.61 / 0.61	6.00 / 1.30	4.20 / 1.90
	[illegible]	26.6	9.57 / 0.00	9.57 / 0.02	9.57 / 0.08	9.57 / 0.15	9.57 / 0.30	8.22 / 0.46	6.96 / 0.61	4.00 / 1.27	3.44 / 2.0
6 x 6	1	37.6	31.7 / 0.01	26.0 / 0.07	17.8 / 0.16	13.1 / 0.20	10.3 / 0.44	8.47 / 0.63	7.12 / 0.86	4.00 / 1.70	3.36 / 2.6
	[illegible]	33.1	26.2 / 0.01	23.8 / 0.07	15.8 / 0.16	11.7 / 0.20	9.25 / 0.43	7.60 / 0.62	6.30 / 0.86	4.13 / 1.74	3.00 / 2.7
	[illegible]	28.7	19.2 / 0.01	19.2 / 0.00	13.8 / 0.15	10.3 / 0.20	8.11 / 0.43	6.66 / 0.62	5.60 / 0.86	3.62 / 1.73	2.64 / 2.7
	[illegible]	24.2	13.8 / 0.01	13.8 / 0.06	11.8 / 0.15	8.75 / 0.27	6.91 / 0.43	5.67 / 0.62	4.77 / 0.84	3.09 / 1.73	2.14 / 2.7
	[illegible]	21.9	11.4 / 0.01	11.4 / 0.06	10.8 / 0.16	7.83 / 0.27	6.36 / 0.42	5.00 / 0.60	4.23 / 0.83	2.66 / 1.72	1.90 / 2.7
	[illegible]	19.6	9.20 / 0.01	9.20 / 0.05	9.20 / 0.16	6.92 / 0.27	5.46 / 0.42	4.48 / 0.60	3.77 / 0.82	2.44 / 1.70	1.74 / 2.7
	[illegible]	17.2	7.00 / 0.01	7.00 / 0.04	7.00 / 0.13	6.06 / 0.27	4.78 / 0.42	3.92 / 0.60	3.20 / 0.82	2.13 / 1.70	1.61 / 2.7
	[illegible]	14.8	5.30 / 0.01	5.30 / 0.04	5.30 / 0.11	5.26 / 0.28	4.16 / 0.42	3.40 / 0.60	2.86 / 0.82	1.86 / 1.71	1.31 / 2.6
5 x 5	[illegible]	23.6	16.4 / 0.01	14.1 / 0.06	9.32 / 0.19	6.91 / 0.33	5.44 / 0.52	4.46 / 0.76	3.72 / 1.02	2.37 / 2.1	1.66 / 3.3
	[illegible]	19.9	13.3 / 0.01	12.1 / 0.08	7.96 / 0.19	5.91 / 0.33	4.66 / 0.52	3.81 / 0.76	3.19 / 1.02	2.03 / 2.1	1.46 / 3.3
	[illegible]	16.1	8.92 / 0.01	8.92 / 0.07	6.56 / 0.18	4.86 / 0.33	3.83 / 0.51	3.13 / 0.74	2.63 / 1.01	1.63 / 2.1	1.10 / 3.2
	[illegible]	12.3	5.26 / 0.01	5.26 / 0.06	4.80 / 0.18	3.62 / 0.32	2.86 / 0.50	2.33 / 0.72	1.96 / 1.00	1.24 / 2.1	0.87 / 3.3
	[illegible]	10.4	3.76 / 0.01	3.76 / 0.05	3.76 / 0.16	3.06 / 0.32	2.40 / 0.50	1.96 / 0.73	1.64 / 1.00	1.04 / 2.1	0.71 / 3.2

Deflection values to the right of the heavy line are greater than 1/180 of the span.
Load values to the left of the broken line are based on shear capacity, and are less than the permissible flexural load.

NOMINAL SIZE	THICKNESS t NOMINAL	WEIGHT PER FOOT	SAFE DISTRIBUTED LOADS (KIPS) / MAX DEFLECTION (in) — SPANS IN FEET								
in	in	lb	2	4	6	8	10	12	14	20	25
4 x 4	[illegible]	15.7	12.6 / 0.03	7.46 / 0.10	4.61 / 0.24	3.63 / 0.42	2.86 / 0.66	2.31 / 0.94	1.93 / 1.30	1.19 / 2.6	0.81 / 4.2
	[illegible]	12.7	9.63 / 0.02	6.17 / 0.10	4.07 / 0.23	3.01 / 0.41	2.38 / 0.66	1.92 / 0.93	1.60 / 1.27	0.99 / 2.6	0.66 / 4.1
	[illegible]	9.7	5.00 / 0.02	4.70 / 0.10	3.16 / 0.23	2.33 / 0.41	1.83 / 0.64	1.46 / 0.91	1.21 / 1.25	0.75 / 2.6	0.49 / 4.1
	[illegible]	8.1	3.63 / 0.01	3.53 / 0.09	2.66 / 0.22	1.90 / 0.40	1.48 / 0.63	1.21 / 0.91	1.00 / 1.26	0.62 / 2.6	0.41 / 4.2
	[illegible]	6.6	2.30 / 0.01	2.30 / 0.07	2.11 / 0.22	1.56 / 0.40	1.22 / 0.63	0.90 / 0.92	0.83 / 1.26	0.49 / 2.6	0.33 / 4.1
3½ x 3½	[illegible]	13.6	11.2 / 0.03	5.56 / 0.12	3.66 / 0.27	2.70 / 0.46	2.11 / 0.76	1.71 / 1.09	1.42 / 1.49	0.86 / 3.1	0.56 / 4.8
	[illegible]	11.0	8.26 / 0.03	4.63 / 0.12	3.04 / 0.27	2.25 / 0.46	1.76 / 0.74	1.42 / 1.07	1.19 / 1.47	0.71 / 3.0	0.47 / 4.8
	[illegible]	8.4	4.96 / 0.02	3.61 / 0.12	2.38 / 0.26	1.76 / 0.47	1.37 / 0.74	1.11 / 1.07	0.92 / 1.40	0.54 / 3.0	0.36 / 4.7
	[illegible]	7.1	3.46 / 0.02	3.02 / 0.12	1.99 / 0.26	1.41 / 0.46	1.10 / 0.72	0.89 / 1.06	0.74 / 1.44	0.46 / 2.9	0.29 / 4.8
	[illegible]	5.7	2.36 / 0.01	2.30 / 0.12	1.57 / 0.26	1.16 / 0.46	0.90 / 0.72	0.73 / 1.06	0.61 / 1.44	0.37 / 3.0	0.24 / 4.8
3 x 3	[illegible]	11.4	7.96 / 0.04	3.96 / 0.14	2.60 / 0.32	1.91 / 0.57	1.49 / 0.90	1.19 / 1.30	0.99 / 1.76	0.57 / 3.8	0.36 / 5.7
	[illegible]	9.3	6.67 / 0.04	3.31 / 0.14	2.17 / 0.32	1.60 / 0.56	1.24 / 0.88	1.00 / 1.27	0.82 / 1.73	0.49 / 3.8	0.30 / 5.8
	[illegible]	7.1	4.80 / 0.03	2.60 / 0.14	1.71 / 0.31	1.26 / 0.56	0.98 / 0.87	0.79 / 1.26	0.66 / 1.71	0.38 / 3.5	0.23 / 5.5
	[illegible]	6.0	3.41 / 0.02	2.18 / 0.14	1.44 / 0.31	1.06 / 0.55	0.82 / 0.86	0.66 / 1.24	0.54 / 1.70	0.30 / 3.5	0.19 / 5.5
	[illegible]	4.9	2.30 / 0.02	1.74 / 0.13	1.14 / 0.30	0.84 / 0.54	0.66 / 0.85	0.52 / 1.23	0.43 / 1.69	0.25 / 3.5	0.16 / 5.6
	[illegible]	3.7	1.35 / 0.02	1.32 / 0.13	0.87 / 0.30	0.64 / 0.54	0.49 / 0.85	0.40 / 1.21	0.32 / 1.66	0.19 / 3.7	0.11 / 5.9

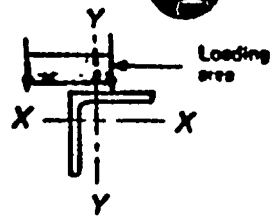
The tables are calculated for loads applied anywhere within half a leg length on either side of the shear centre — see fig 1



EQUAL ANGLES

NORMAL STRENGTH STEEL (36 ksi)

BASED ON AS CA1 1968



NOMINAL SIZE	THICKNESS t NOMINAL	WEIGHT PER FOOT	SAFE DISTRIBUTED LOADS (KIPS) / MAX DEFLECTION (in) — SPANS IN FEET								
in	in	lb	2	4	6	8	10	12	14	20	26
2½ × 2½	[illegible]	7.0	4.40 / 0.04	2.20 / 0.17	1.44 / 0.38	1.[illegible] / 0.[illegible]	0.82 / 1.07	0.[illegible] / 1.54	0.53 / 2.1	0.29 / 4.3	0.17 / [illegible]
	[illegible]	5.9	3.54 / 0.04	1.76 / 0.17	1.15 / 0.38	0.84 / 0.87	0.66 / 1.06	0.52 / 1.57	0.42 / 2.1	0.23 / 4.3	0.13 / [illegible]
	[illegible]	5.0	2.[illegible] / 0.04	1.40 / 0.17	0.97 / 0.37	0.71 / 0.[illegible]	0.56 / 1.04	0.44 / 1.51	0.38 / 2.1	0.19 / 4.2	0.11 / 6.7
	[illegible]	4.0	2.42 / 0.04	1.19 / 0.17	0.78 / 0.38	0.[illegible] / 0.65	0.44 / 1.03	0.36 / 1.40	0.28 / 2.0	0.14 / 4.3	
	[illegible]	3.1	1.32 / 0.04	0.91 / 0.10	0.[illegible] / 0.38	0.43 / 0.66	0.34 / 1.03	0.27 / 1.40	0.22 / 2.0	0.12 / 4.4	
2¼ × 2¼	[illegible]	[illegible]	2.81 / 0.0[illegible]	1.30 / 0.19	0.91 / 0.42	0.[illegible] / 0.75	0.51 / 1.17	0.41 / 1.70	0.33 / 2.3	0.18 / 4.8	
	[illegible]	4.4	2.30 / 0.0[illegible]	1.10 / 0.19	0.77 / 0.42	0.[illegible] / 0.74	0.43 / 1.10	0.38 / 1.[illegible]	0.29 / 2.3	0.14 / 4.8	
	[illegible]	3.8	1.[illegible] / 0.0[illegible]	0.[illegible] / 0.19	0.64 / 0.41	0.47 / 0.76	0.38 / 1.18	0.30 / 1.[illegible]	0.22 / 2.3	0.12 / 4.7	
	[illegible]	2.7	1.30 / 0.0[illegible]	0.74 / 0.18	0.48 / 0.40	0.38 / 0.72	0.27 / 1.18	0.21 / 1.64	0.17 / 2.3		
2 × 2	[illegible]	4.8	2.17 / 0.05	1.07 / 0.21	0.70 / 0.48	0.51 / 0.86	0.39 / 1.33	0.31 / 1.92	0.26 / 2.8	0.13 / 5.8	
	[illegible]	3.9	2.06 / 0.05	0.91 / 0.21	0.60 / 0.47	0.42 / 0.84	0.33 / 1.32	0.26 / 1.91	0.21 / 2.6	0.10 / 5.4	
	[illegible]	3.2	1.[illegible] / 0.05	0.78 / 0.21	0.50 / 0.47	0.38 / 0.83	0.28 / 1.31	0.22 / 1.90	0.17 / 2.8		
	[illegible]	2.4	1.20 / 0.05	0.[illegible] / 0.21	0.37 / 0.46	0.27 / 0.82	0.21 / 1.30	0.18 / 1.88	0.13 / 2.8		
	[illegible]	1.7	0.[illegible] / 0.04	0.40 / 0.20	0.25 / 0.46	0.18 / 0.81	0.14 / 1.29	0.11 / 1.90			

Deflection values to the right of the heavy line are greater than 1/180 of the span.

NOMINAL SIZE	THICKNESS t NOMINAL	WEIGHT PER FOOT	SAFE DISTRIBUTED LOADS (KIPS) / MAX DEFLECTION (in) — SPANS IN FEET								
in	in	lb	2	4	6	8	10	12	14	20	26
1¾ × 1¾	[illegible]	4.0	1.61 / 0.06	0.79 / 0.25	0.52 / 0.55	0.37 / 0.90	0.28 / 1.55	0.22 / 2.2	0.18 / 3.1		
	[illegible]	3.4	1.38 / 0.06	0.[illegible] / 0.24	0.44 / 0.54	0.32 / 0.97	0.24 / 1.52	0.19 / 2.2	0.15 / 3.0		
	[illegible]	2.7	1.16 / 0.06	0.57 / 0.24	0.37 / 0.54	0.27 / 0.96	0.21 / 1.51	0.16 / 2.2	0.12 / 3.0		
	[illegible]	2.1	0.[illegible] / 0.06	0.44 / 0.24	0.29 / 0.53	0.21 / 0.94	0.16 / 1.40	0.12 / 2.2			
	[illegible]	1.5	0.[illegible] / 0.06	0.30 / 0.21	0.19 / 0.52	0.14 / 0.94	0.11 / 1.90				
1½ × 1½	[illegible]	2.86	0.[illegible] / 0.07	0.48 / 0.29	0.31 / 0.64	0.22 / 1.15	0.17 / 1.90	0.13 / 2.0	0.10 / 3.0		
	[illegible]	2.35	0.83 / 0.07	0.41 / 0.28	0.26 / 0.63	0.19 / 1.13	0.14 / 1.80	0.11 / 2.0			
	[illegible]	1.8	0.66 / 0.07	0.32 / 0.28	0.21 / 0.63	0.16 / 1.12	0.11 / 1.75				
	[illegible]	1.2	0.44 / 0.07	0.22 / 0.27	0.14 / 0.62	0.10 / 1.10					
1¼ × 1¼	[illegible]	2.3	0.[illegible] / 0.0[illegible]	0.32 / 0.36	0.21 / 0.79	0.16 / 1.41	0.11 / 2.2				
	[illegible]	1.9	0.[illegible] / 0.09	0.27 / 0.36	0.18 / 0.77	0.12 / 1.38					
	[illegible]	1.46	0.43 / 0.08	0.21 / 0.33	0.14 / 0.76						
	[illegible]	1.0	0.31 / 0.08	0.15 / 0.33							
1 × 1	[illegible]	1.5	0.34 / 0.11	0.16 / 0.44	0.10 / 0.99						
	[illegible]	1.15	0.27 / 0.11	0.13 / 0.43							
	[illegible]	0.8	0.19 / 0.10								

The tables are calculated for loads applied anywhere within half a leg length on either side of the shear centre—see fig. 1.

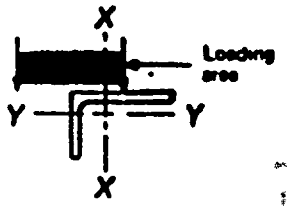
Safe Load Tables for Laterally Unsupported Angles

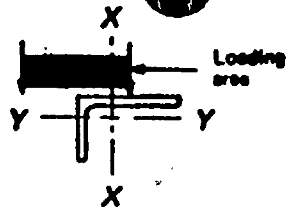
UNEQUAL ANGLES

Force parallel to SHORT leg

NORMAL STRENGTH STEEL (36 ksi)

BASED ON AS CA1 1968



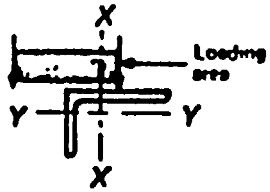


NOMINAL SIZE	THICKNESS t, NOMINAL	WEIGHT PER FOOT	SAFE DISTRIBUTED LOADS (KIPS) / MAX DEFLECTION (in) SPANS IN FEET								
in	in	lb	2	4	6	8	10	12	14	20	26
6 x 4	[illegible]	23.6	14.6 / [illegible]	[illegible] / 0.10	[illegible] / 0.22	4.30 / 0.39	3.42 / 0.61	2.77 / [illegible]	2.70 / 1.20	1.36 / 2.4	0.87 / 3.8
	[illegible]	19.9	10.8 / 0.01	7.74 / 0.10	5.10 / 0.21	3.76 / 0.39	2.83 / [illegible]	2.37 / [illegible]	1.96 / 1.17	1.17 / 2.4	0.78 / 3.8
	[illegible]	16.1	7.21 / 0.01	6.41 / 0.10	4.22 / 0.21	3.11 / 0.39	2.43 / [illegible]	1.99 / [illegible]	1.62 / 1.17	0.97 / 2.4	0.63 / 3.8
	[illegible]	14.2	[illegible] / 0.01	[illegible] / 0.10	2.75 / 0.21	2.78 / 0.39	2.16 / 0.60	1.76 / [illegible]	1.44 / 1.17	0.87 / 2.4	0.57 / 3.8
	[illegible]	12.3	4.30 / 0.01	4.30 / 0.10	2.27 / 0.21	2.41 / 0.39	1.90 / 0.60	1.53 / [illegible]	1.26 / 1.17	0.76 / 2.4	0.50 / 3.8
6 x 3½	[illegible]	16.8	10.2 / 0.03	6.94 / 0.11	3.00 / 0.24	2.96 / 0.43	2.21 / 0.67	1.77 / 0.97	1.46 / 1.32	0.82 / 2.7	0.40 / 4.3
	[illegible]	15.3	8.74 / 0.02	4.83 / 0.11	3.23 / 0.26	2.37 / 0.44	1.84 / [illegible]	1.48 / [illegible]	1.21 / 1.36	0.80 / 2.8	0.42 / 4.3
	[illegible]	13.6	6.33 / 0.02	4.41 / 0.10	2.89 / 0.24	2.12 / 0.43	1.66 / [illegible]	1.32 / [illegible]	1.08 / 1.33	0.62 / 2.7	0.38 / 4.3
	[illegible]	11.6	3.87 / 0.02	3.82 / 0.10	2.51 / 0.24	1.84 / 0.42	1.43 / [illegible]	1.15 / 0.96	0.94 / 1.30	0.64 / 2.7	0.33 / 4.3
	[illegible]	9.7	2.81 / 0.01	2.81 / 0.10	2.12 / 0.24	1.56 / 0.43	1.21 / 0.67	0.97 / [illegible]	0.80 / 1.31	0.46 / 2.7	0.28 / 4.5
5 x 3½	[illegible]	16.7	10.5 / 0.03	5.78 / 0.11	3.78 / 0.25	2.78 / 0.45	2.18 / 0.70	1.74 / 1.01	1.43 / 1.38	0.83 / 2.8	0.52 / 4.4
	[illegible]	13.6	7.17 / 0.02	4.70 / 0.11	3.16 / 0.25	2.32 / 0.45	1.80 / 0.70	1.46 / 1.01	1.20 / 1.38	0.69 / 2.8	0.43 / 4.4
	[illegible]	10.3	4.27 / 0.02	3.71 / 0.11	2.44 / 0.25	1.78 / 0.44	1.40 / 0.67	1.13 / 1.00	0.92 / 1.34	0.54 / 2.8	0.34 / 4.5
	[illegible]	8.7	3.03 / 0.01	3.03 / 0.11	2.06 / 0.24	1.61 / 0.43	1.18 / 0.67	0.95 / 0.97	0.78 / 1.34	0.46 / 2.8	0.29 / 4.6

Deflection values to the right of the heavy line are greater than 1/180 of the span.
Load values to the left of the broken line are based on shear capacity, and are less than the permissible flexural load.

NOMINAL SIZE	THICKNESS t, NOMINAL	WEIGHT PER FOOT	SAFE DISTRIBUTED LOADS (KIPS) / MAX DEFLECTION (in) SPANS IN FEET								
in	in	lb	2	4	6	8	10	12	14	20	26
5 x 3 *	[illegible]	12.7	6.53 / 0.03	3.90 / 0.12	3.30 / [illegible]	1.67 / [illegible]	1.30 / 0.70	1.08 * / 1.12	[illegible] / 1.56	[illegible] / 3.3	[illegible] / 5.1
	[illegible]	11.2	5.16 / 0.02	3.13 / 0.12	2.66 / [illegible]	1.50 / [illegible]	1.16 / 0.70	[illegible] / 1.12	0.76 / 1.53	0.41 / 3.3	[illegible] / 5.1
	[illegible]	9.7	3.92 / 0.02	2.76 / 0.12	1.80 / [illegible]	1.32 / 0.60	1.02 / 0.77	0.81 / 1.12	[illegible] / 1.53	[illegible] / 3.3	0.30 / 5.1
	[illegible]	8.1	2.78 / 0.02	2.22 / 0.12	1.52 / [illegible]	1.11 / [illegible]	[illegible] / 0.70	[illegible] / 1.12	[illegible] / 1.53	0.21 / 3.3	0.17 / 5.2
	[illegible]	6.6	1.90 / 0.01	1.90 / 0.12	1.30 / 0.27	[illegible] / 0.48	0.71 / 0.77	0.57 / 1.12	[illegible] / 1.54	[illegible] / 3.3	0.14 / 5.4
4 x 3	[illegible]	13.6	8.90 / 0.03	4.21 / 0.13	2.76 / 0.31	2.02 / [illegible]	1.57 / [illegible]	1.28 / 1.23	1.02 / 1.60	0.57 / 3.4	0.33 / 5.4
	[illegible]	11.0	6.82 / 0.03	3.38 / 0.13	2.20 / 0.30	1.62 / 0.52	1.26 / [illegible]	1.01 / 1.10	0.82 / 1.61	0.46 / 3.3	0.27 / 5.3
	[illegible]	8.4	4.29 / 0.02	2.62 / 0.13	1.73 / 0.30	1.27 / 0.51	[illegible] / 0.81	0.79 / 1.16	0.64 / 1.50	0.37 / 3.3	0.22 / 5.3
	[illegible]	7.1	3.05 / 0.02	2.26 / 0.13	1.48 / 0.30	1.09 / 0.51	[illegible] / [illegible]	[illegible] / 1.16	[illegible] / 1.50	0.32 / 3.3	0.19 / 5.4
	[illegible]	5.8	3.76 / 0.03	1.86 / 0.13	1.22 / 0.29	[illegible] / 0.52	0.70 / 0.81	[illegible] / 1.10	0.46 / 1.60	0.26 / 3.4	0.16 / 5.6
3½ x 3	[illegible]	10.2	[illegible] / 0.03	3.40 / 0.14	2.23 / 0.31	1.64 / [illegible]	1.27 / [illegible]	1.02 / 1.20	[illegible] / 1.60	0.48 / 3.4	0.30 / 5.4
	[illegible]	7.9	5.33 / 0.03	2.64 / 0.13	1.74 / 0.30	1.27 / [illegible]	[illegible] / [illegible]	[illegible] / 1.21	[illegible] / 1.64	0.38 / 3.4	0.23 / 5.4
	[illegible]	6.6	4.52 / 0.03	2.24 / 0.13	1.47 / 0.30	1.08 / 0.54	[illegible] / [illegible]	[illegible] / 1.21	0.57 / 1.66	0.31 / 3.3	0.19 / 5.3
	[illegible]	5.3	3.70 / 0.03	1.83 / 0.13	1.20 / 0.30	[illegible] / 0.53	0.67 / [illegible]	0.56 / 1.21	0.44 / 1.61	0.26 / 3.3	0.15 / 5.2

The tables are calculated for loads applied anywhere within half a leg length on either side of the shear centre — see Fig. 1.

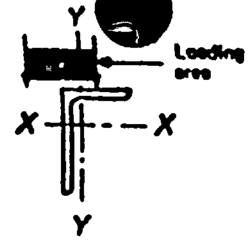


UNEQUAL ANGLES

Force parallel to LONG leg

NORMAL STRENGTH STEEL (36 ksi)

BASED ON AS CA1 1968



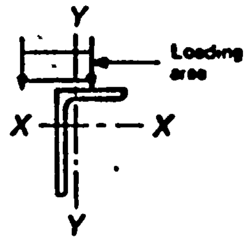
NOMINAL SIZE	THICKNESS t NOMINAL	WEIGHT PER FOOT	SAFE DISTRIBUTED LOADS (KIPS) / MAX DEFLECTION (in) SPANS IN FEET								
in	in	lb	2	4	6	8	10	12	14	20	25
3½ x 2½	½	9.4	4.81 / 0.04	2.38 / 0.10	1.06 / 0.37	1.13 / 0.66	0.87 / 1.02	0.66 / 1.42	0.53 / 1.93	0.28 / 4.0	0.14 / 8.2
	⅜	7.1	3.71 / 0.06	1.84 / 0.18	1.20 / 0.38	0.88 / 0.63	0.67 / 0.99	0.54 / 1.42	0.43 / 1.94	0.23 / 4.0	0.12 / 8.1
	5/16	6.0	2.87 / 0.04	1.46 / 0.16	1.02 / 0.35	0.76 / 0.62	0.57 / 0.97	0.46 / 1.41	0.37 / 1.92	0.20 / 3.9	0.10 / 8.2
	¼	4.9	1.89 / 0.03	1.29 / 0.16	0.84 / 0.35	0.59 / 0.60	0.46 / 0.95	0.36 / 1.37	0.29 / 1.87	0.16 / 3.9	
	3/16	3.7	1.16 / 0.02	0.97 / 0.15	0.63 / 0.34	0.46 / 0.60	0.38 / 0.94	0.28 / 1.36	0.23 / 1.87	0.12 / 3.9	
3 x 2½	½	8.5	4.06 / 0.04	2.28 / 0.17	1.40 / 0.37	1.08 / 0.66	0.84 / 1.04	0.67 / 1.50	0.54 / 2.0	0.29 / 4.2	0.18 / 6.6
	⅜	6.5	3.61 / 0.06	1.79 / 0.18	1.17 / 0.36	0.86 / 0.65	0.66 / 1.01	0.53 / 1.46	0.43 / 2.0	0.23 / 4.1	0.13 / 6.5
	5/16	5.5	3.07 / 0.04	1.54 / 0.16	1.01 / 0.36	0.74 / 0.64	0.57 / 1.00	0.46 / 1.40	0.37 / 2.0	0.20 / 4.0	0.11 / 6.3
	¼	4.4	2.01 / 0.03	1.26 / 0.16	0.82 / 0.36	0.60 / 0.63	0.46 / 1.00	0.37 / 1.47	0.30 / 2.0	0.16 / 4.1	
	3/16	3.4	1.22 / 0.03	0.96 / 0.15	0.62 / 0.35	0.46 / 0.62	0.36 / 0.97	0.28 / 1.41	0.24 / 2.0	0.12 / 4.2	
3 x 2	½	7.7	2.88 / 0.05	1.42 / 0.20	0.92 / 0.40	0.68 / 0.81	0.50 / 1.27	0.38 / 1.83	0.30 / 2.5	0.14 / 5.1	
	⅜	5.9	2.38 / 0.06	1.18 / 0.20	0.78 / 0.46	0.52 / 0.77	0.38 / 1.21	0.31 / 1.74	0.24 / 2.4	0.11 / 5.0	
	5/16	5.0	1.99 / 0.05	0.98 / 0.20	0.64 / 0.44	0.46 / 0.78	0.36 / 1.21	0.27 / 1.75	0.22 / 2.4	0.10 / 4.9	
	¼	4.0	1.63 / 0.05	0.81 / 0.20	0.52 / 0.43	0.38 / 0.77	0.29 / 1.20	0.23 / 1.70	0.18 / 2.4		
	3/16	3.1	1.08 / 0.04	0.76 / 0.20	0.41 / 0.43	0.30 / 0.76	0.23 / 1.20	0.18 / 1.73	0.14 / 2.4		
2½ x 2	⅜	5.3	2.24 / 0.06	1.19 / 0.20	0.72 / 0.46	0.52 / 0.81	0.40 / 1.20	0.31 / 1.87	0.25 / 2.6	0.11 / 5.1	
	5/16	4.4	1.86 / 0.05	0.92 / 0.20	0.60 / 0.44	0.43 / 0.79	0.33 / 1.21	0.26 / 1.80	0.20 / 2.4	0.10 / 5.2	
	¼	3.6	1.57 / 0.05	0.77 / 0.20	0.50 / 0.44	0.37 / 0.80	0.28 / 1.22	0.22 / 1.79	0.18 / 2.4		
	3/16	2.7	1.08 / 0.05	0.98 / 0.19	0.38 / 0.41	0.28 / 0.80	0.21 / 1.22	0.17 / 1.78	0.13 / 2.4		

Deflection values to the right of the heavy line are greater than 1/180 of the span.
Load values to the left of the broken line are based on shear capacity, and are less than the permissible flexural load.

NOMINAL SIZE	THICKNESS t NOMINAL	WEIGHT PER FOOT	SAFE DISTRIBUTED LOADS (KIPS) / MAX DEFLECTION (in) SPANS IN FEET								
in	in	lb	2	4	6	8	10	12	14	20	25
6 x 4	¾	23.6	23.0 / 0.01	19.2 / 0.07	12.7 / 0.16	9.23 / 0.29	7.39 / 0.44	6.06 / 0.64	5.06 / 0.89	3.30 / 1.81	2.43 / 2.8
	⅝	19.6	16.6 / 0.01	16.1 / 0.07	10.7 / 0.16	7.94 / 0.29	6.28 / 0.44	5.16 / 0.64	4.35 / 0.88	2.84 / 1.83	2.03 / 2.8
	½	16.1	11.0 / 0.01	11.0 / 0.06	8.68 / 0.16	6.40 / 0.29	5.11 / 0.44	4.20 / 0.64	3.04 / 0.89	2.23 / 1.81	1.66 / 2.8
	7/16	14.2	9.06 / 0.02	9.06 / 0.05	7.57 / 0.16	5.63 / 0.29	4.46 / 0.44	3.66 / 0.64	3.08 / 0.89	2.01 / 1.07	1.42 / 2.8
	⅜	12.3	6.53 / 0.01	6.53 / 0.06	6.40 / 0.16	4.80 / 0.29	3.80 / 0.44	3.12 / 0.64	2.63 / 0.88	1.72 / 1.82	1.20 / 2.8
6 x 3½	⅝	18.8	19.2 / 0.01	15.9 / 0.07	10.5 / 0.16	7.83 / 0.29	6.29 / 0.46	5.09 / 0.66	4.39 / 0.91	2.71 / 1.87	2.00 / 2.9
	½	15.3	11.9 / 0.01	11.9 / 0.07	8.40 / 0.16	6.28 / 0.29	4.98 / 0.45	4.07 / 0.66	3.43 / 0.91	2.24 / 1.90	1.60 / 2.1
	7/16	13.5	9.38 / 0.01	9.38 / 0.06	7.42 / 0.19	5.52 / 0.29	4.32 / 0.46	3.56 / 0.66	2.99 / 0.91	1.96 / 1.93	1.38 / 2.9
	⅜	11.8	6.96 / 0.01	6.96 / 0.06	6.31 / 0.16	4.70 / 0.29	3.71 / 0.46	3.06 / 0.67	2.57 / 0.92	1.63 / 1.92	1.19 / 2.2
	5/16	9.7	4.92 / 0.01	4.92 / 0.04	4.92 / 0.15	3.92 / 0.29	3.19 / 0.47	2.46 / 0.67	2.04 / 0.92	1.31 / 2.1	0.98 / 2.9
5 x 3½	⅝	16.7	19.6 / 0.01	11.2 / 0.08	7.38 / 0.16	5.48 / 0.34	4.38 / 0.56	3.56 / 0.76	2.97 / 1.07	1.91 / 2.2	1.32 / 3.0
	½	13.6	16.6 / 0.01	9.16 / 0.09	6.06 / 0.19	4.88 / 0.34	3.58 / 0.54	2.91 / 0.76	2.44 / 1.00	1.57 / 2.2	1.07 / 3.5
	⅜	10.3	8.13 / 0.01	8.13 / 0.07	4.51 / 0.19	3.34 / 0.34	2.64 / 0.53	2.16 / 0.77	1.81 / 1.05	1.17 / 2.3	0.79 / 3.8
	5/16	8.7	4.30 / 0.01	4.30 / 0.06	3.66 / 0.19	2.77 / 0.34	2.19 / 0.54	1.79 / 0.77	1.50 / 1.06	0.98 / 2.2	0.68 / 3.7

The tables are calculated for loads applied anywhere within half a leg length on either side of the shear centre— see fig. 1

Safe Load Tables for Laterally Unsupported Angles

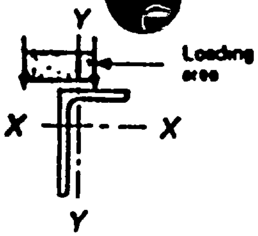


UNEQUAL ANGLES

Force parallel to LONG leg

NORMAL STRENGTH STEEL (36 ksi)

BASED ON AS CA1 1968



NOMINAL SIZE	THICKNESS t, NOMINAL	WEIGHT PER FOOT	SAFE DISTRIBUTED LOADS (KIPS) / MAX DEFLECTION (in) SPANS IN FEET								
in	in	lb	2	4	6	8	10	12	14	20	25
5 x 3	[illegible]	12.7	11.8 / 0.01	8.70 / [illegible]	6.81 / 0.19	4.30 / 0.34	3.41 / 0.54	2.70 / 0.76	2.30 / 1.03	1.81 / 2.3	1.00 / 3.7
	[illegible]	11.2	9.87 / 0.01	7.70 / [illegible]	5.10 / 0.19	3.83 / 0.36	3.02 / 0.54	2.40 / 0.79	2.08 / 1.09	1.29 / 2.3	0.92 / 3.7
	[illegible]	9.7	8.70 / 0.01	8.75 / [illegible]	4.47 / 0.19	3.32 / 0.36	2.82 / 0.56	2.18 / 0.81	1.77 / 1.08	1.12 / 2.4	0.77 / 3.8
	[illegible]	8.1	4.73 / 0.01	4.73 / 0.07	3.04 / 0.19	2.90 / 0.36	2.13 / 0.56	1.76 / 0.81	1.47 / 1.13	0.90 / 2.4	0.66 / 4.0
	[illegible]	6.6	3.30 / 0.01	3.30 / [illegible]	2.83 / 0.19	2.16 / 0.36	1.88 / 0.56	1.37 / 0.82	1.16 / 1.16	0.67 / 2.6	0.44 / 4.2
4 x 3	[illegible]	13.6	14.7 / [illegible]	7.31 / 0.11	4.83 / 0.24	3.57 / 0.44	2.81 / [illegible]	2.29 / [illegible]	1.91 / 1.36	1.19 / 2.8	0.83 / 4.4
	[illegible]	11.0	9.87 / 0.02	6.00 / 0.11	3.87 / 0.24	2.88 / 0.43	2.22 / 0.67	1.81 / 0.87	1.51 / 1.33	0.94 / 2.8	0.64 / 4.4
	[illegible]	8.4	6.70 / 0.02	4.44 / 0.11	2.93 / 0.23	2.17 / 0.42	1.71 / 0.67	1.30 / 0.87	1.10 / 1.33	0.70 / 2.8	0.47 / 4.3
	[illegible]	7.1	4.13 / 0.02	3.74 / 0.10	2.47 / 0.24	1.83 / 0.42	1.44 / 0.67	1.17 / 0.87	0.97 / 1.32	0.60 / 2.8	0.40 / 4.5
	[illegible]	5.8	2.75 / 0.01	2.75 / [illegible]	2.00 / 0.24	1.40 / 0.43	1.14 / [illegible]	0.93 / 0.87	0.77 / 1.36	0.48 / 3.0	0.29 / 4.6
3½ x 3	[illegible]	10.2	9.12 / 0.03	4.53 / 0.12	2.90 / 0.28	2.20 / 0.49	1.67 / 0.76	1.36 / 1.10	1.12 / 1.50	0.87 / 3.1	0.44 / 4.7
	[illegible]	7.8	6.28 / 0.03	3.38 / 0.12	2.23 / 0.28	1.64 / 0.47	1.29 / 0.74	1.04 / 1.06	0.87 / 1.45	0.53 / 3.0	0.33 / 4.9
	[illegible]	6.6	3.71 / 0.02	2.98 / 0.12	1.90 / 0.27	1.40 / 0.47	1.07 / 0.75	0.87 / 1.06	0.72 / 1.46	0.44 / 3.1	0.29 / 5.0
	[illegible]	5.3	2.40 / 0.02	2.30 / 0.12	1.57 / 0.27	1.17 / 0.48	0.88 / 0.74	0.72 / 1.06	0.59 / 1.49	0.33 / 3.2	0.20 / 5.0

Deflection values to the right of the heavy line are greater than 1/180 of the span.
Load values to the left of the broken line are based on shear capacity, and are less than the permissible flexural load.

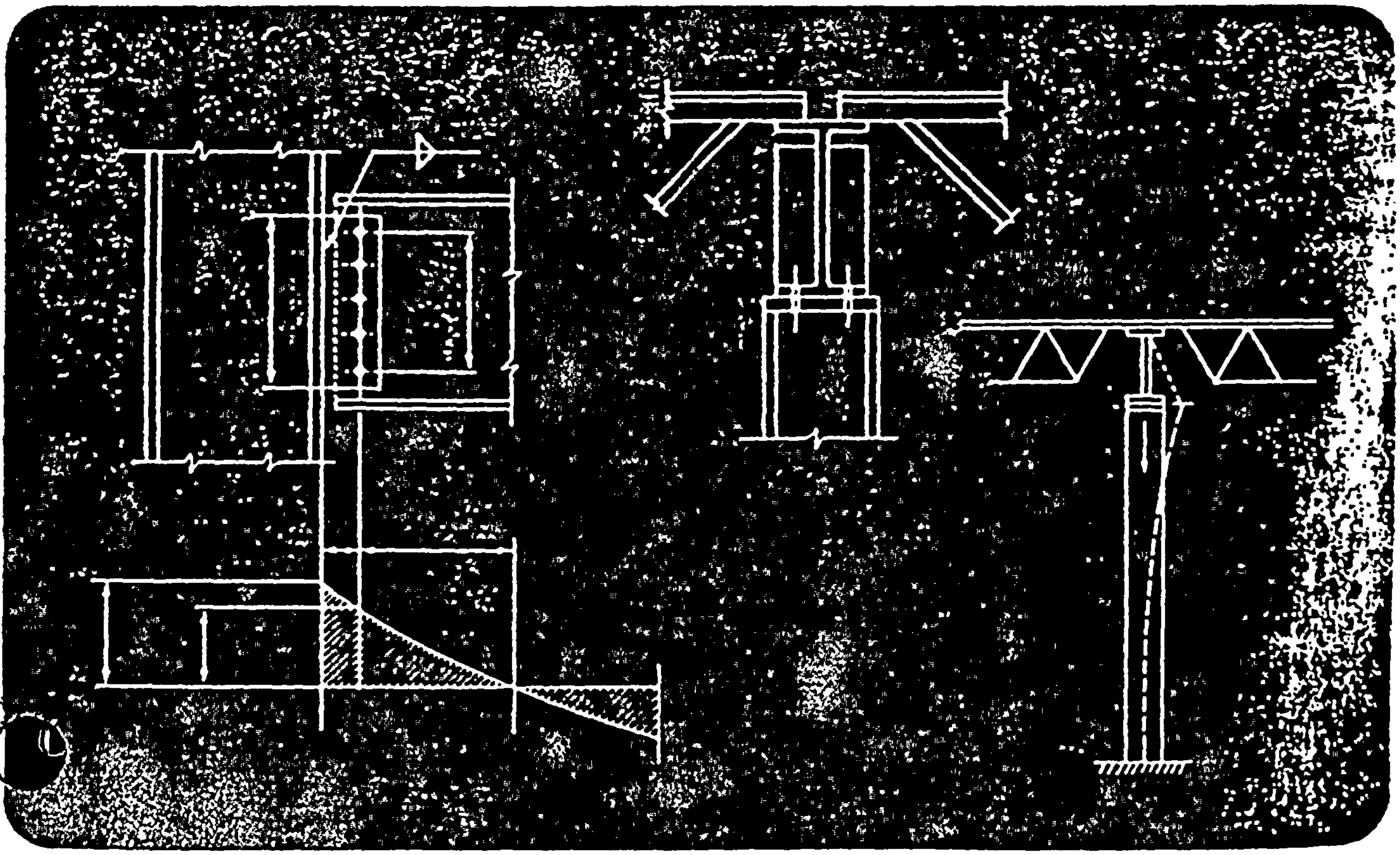
NOMINAL SIZE	THICKNESS t, NOMINAL	WEIGHT PER FOOT	SAFE DISTRIBUTED LOADS (KIPS) / MAX DEFLECTION (in) SPANS IN FEET								
in	in	lb	2	4	6	8	10	12	14	20	25
3½ x 2½	[illegible]	8.4	[illegible] / 0.03	4.30 / 0.13	2.84 / 0.28	2.10 / 0.49	1.84 / 0.77	1.34 / 1.12	1.11 / 1.53	[illegible] / 3.2	[illegible] / 5.0
	[illegible]	7.1	6.66 / 0.03	3.33 / 0.12	2.19 / 0.27	1.62 / 0.49	1.27 / 0.77	1.01 / 1.11	0.84 / 1.52	0.51 / 3.2	0.33 / 5.1
	[illegible]	6.0	4.00 / 0.02	2.86 / 0.12	1.88 / 0.28	1.36 / 0.48	1.08 / 0.77	0.86 / 1.13	0.71 / 1.53	0.42 / 3.3	0.28 / 5.3
	[illegible]	4.8	2.80 / 0.02	2.28 / 0.12	1.44 / 0.27	1.08 / 0.48	0.78 / 0.77	0.66 / 1.15	0.56 / 1.58	0.33 / 3.4	0.20 / 5.5
	[illegible]	3.7	1.83 / 0.01	1.83 / 0.12	1.11 / 0.27	0.81 / 0.48	0.63 / 0.77	0.50 / 1.17	0.41 / 1.61	0.23 / 3.6	0.13 / 5.6
3 x 2½	[illegible]	6.6	6.40 / 0.04	3.10 / 0.15	2.08 / 0.32	1.54 / 0.57	1.20 / 0.88	0.97 / 1.31	0.80 / 1.77	0.47 / 3.7	0.30 / 5.8
	[illegible]	6.5	5.83 / 0.04	2.60 / 0.14	1.64 / 0.32	1.21 / 0.57	0.94 / 0.90	0.76 / 1.28	0.61 / 1.76	0.38 / 3.8	0.23 / 6.0
	[illegible]	5.6	3.72 / 0.03	2.12 / 0.14	1.40 / 0.33	1.03 / 0.56	0.80 / 0.89	0.62 / 1.27	0.52 / 1.76	0.30 / 3.8	0.19 / 6.0
	[illegible]	4.4	2.43 / 0.03	1.88 / 0.14	1.10 / 0.31	0.81 / 0.56	0.63 / 0.88	0.51 / 1.28	0.42 / 1.80	0.24 / 3.7	0.15 / 6.0
	[illegible]	3.4	1.47 / 0.02	1.31 / 0.14	0.86 / 0.31	0.62 / 0.56	0.50 / 0.86	0.38 / 1.30	0.32 / 1.80	0.18 / 3.7	0.10 / 6.3
3 x 2	[illegible]	7.7	6.18 / 0.04	3.08 / 0.15	2.02 / 0.33	1.40 / 0.58	1.18 / 0.83	0.94 / 1.35	0.78 / 1.84	0.44 / 3.9	0.28 / 6.1
	[illegible]	5.9	4.80 / 0.04	2.38 / 0.14	1.57 / 0.33	1.16 / 0.58	0.90 / 0.92	0.73 / 1.34	0.60 / 1.83	0.34 / 3.8	0.22 / 6.3
	[illegible]	5.0	4.14 / 0.04	2.06 / 0.15	1.29 / 0.33	0.98 / 0.60	0.86 / 1.00	0.56 / 1.35	0.40 / 1.84	0.29 / 4.0	0.19 / 6.5
	[illegible]	4.0	2.78 / 0.03	1.63 / 0.14	1.04 / 0.32	0.79 / 0.58	0.60 / 0.82	0.48 / 1.38	0.38 / 1.89	0.22 / 4.1	0.13 / 6.6
	[illegible]	3.1	1.64 / 0.04	1.21 / 0.14	0.80 / 0.32	0.58 / 0.58	0.46 / 0.86	0.38 / 1.38	0.30 / 1.86	0.16 / 4.3	
2½ x 2	[illegible]	5.2	3.31 / 0.04	1.84 / 0.17	1.08 / 0.38	0.79 / 0.68	0.61 / 1.08	0.48 / 1.57	0.40 / 2.2	0.20 / 4.3	0.11 / 7.2
	[illegible]	4.4	2.82 / 0.04	1.40 / 0.17	0.88 / 0.38	0.67 / 0.66	0.52 / 1.08	0.42 / 1.58	0.33 / 2.2	0.17 / 4.4	0.10 / 7.1
	[illegible]	3.6	2.34 / 0.04	1.18 / 0.17	0.78 / 0.38	0.53 / 0.68	0.43 / 1.10	0.36 / 1.60	0.28 / 2.2	0.15 / 4.4	
	[illegible]	2.7	1.37 / 0.03	0.84 / 0.17	0.56 / 0.38	0.40 / 0.67	0.31 / 1.07	0.25 / 1.56	0.20 / 2.2	0.10 / 4.7	

The tables are calculated for loads applied anywhere within half a leg length on either side of the shear centre—see fig 1.

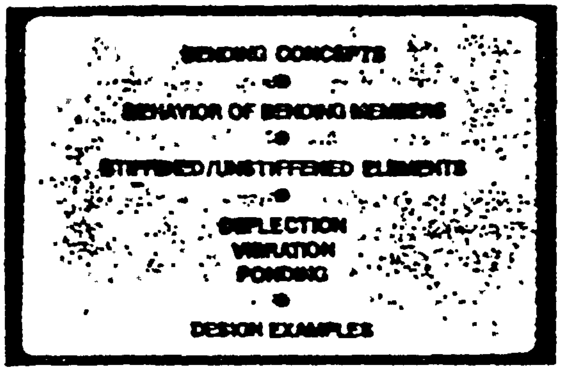
2

BENDING MEMBERS

STEEL DESIGN *CURRENT PRACTICE*



Slide No. 2-3



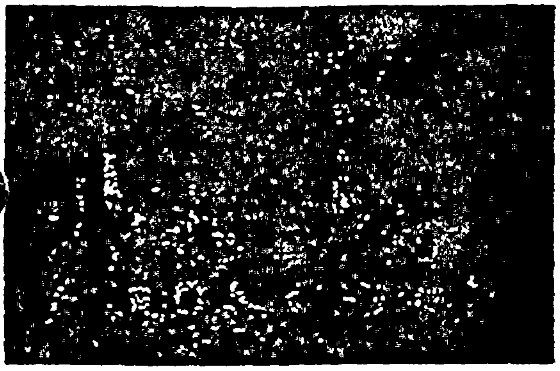
* Preview contents of the lecture.

* Bending concepts - review basic concepts.

* Behavior of Bending Members - related to different failure mode.

* Material includes:
  - compact sections
  - non-compact sections
  - laterally unsupported beams
  - box sections

Slide No. 2-4

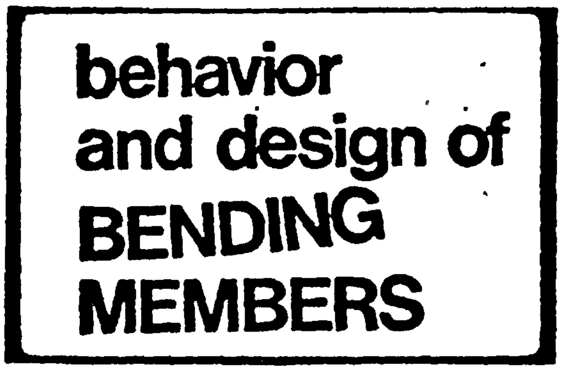


* Section modulus S = maximum moment from the moment diagram at working or design load levels divided by Allowable Bending Stress.

* Plastic modulus Z = maximum moment from moment diagram at some specified overload (usually 70% overload for gravity loads) divided by yield point.

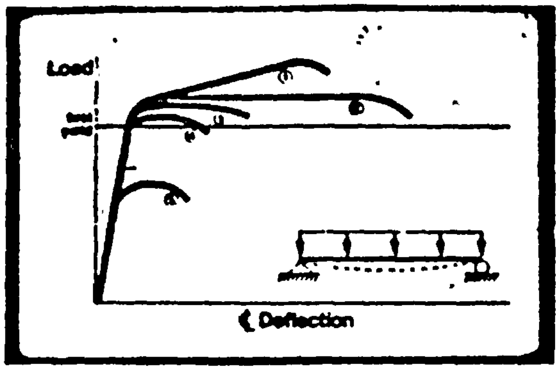
* When are these formulas valid?

Slide No. 2-5



* To understand and properly utilize the design methods and specification provisions for structural steel design, the variables that affect beam behavior will be explored.

Slide No. 2-6



* All beam behavior cannot be represented by a single load-deflection curve because of the number of variables involved.

* Five curves represent potential behavior of a beam or girder in a building.

  - 1 2 Plastic straining without local or lateral buckling.
  - 3 4 Reach first yield without local or lateral buckling.
  - 5 Lateral or local buckling.

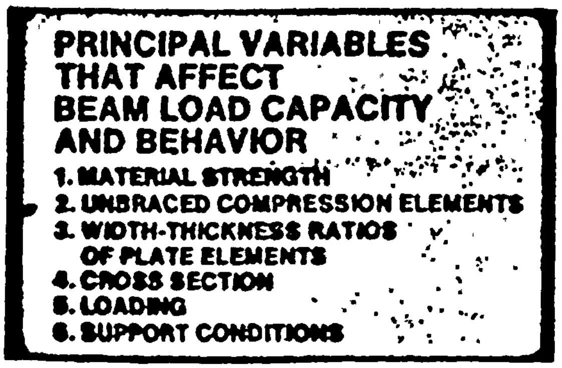
* The various allowable stresses permitted in the latest AISC Spec. are related to the behavior depicted in the 5 curves.

* Plastic Design is based on behavior curves (1) and (2) so provisions are established to prevent types (3) (4) (5).

* Curves 1 and 2 will generally provide the lightest beams but sometimes the fabrication and detail is increased because of the added bracing, stiffeners, etc.

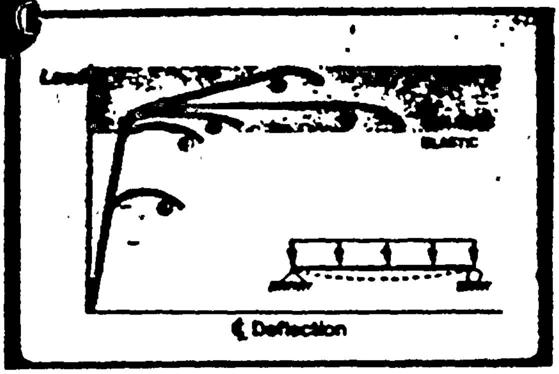
* The proper design is the economical one, not the lightest one.

Slide No. 2-7



* Principal variables that affect beam load capacity and behavior.

Slide No. 2-8



* Safe, economical structures can be designed on the basis of any one of these typical curves.

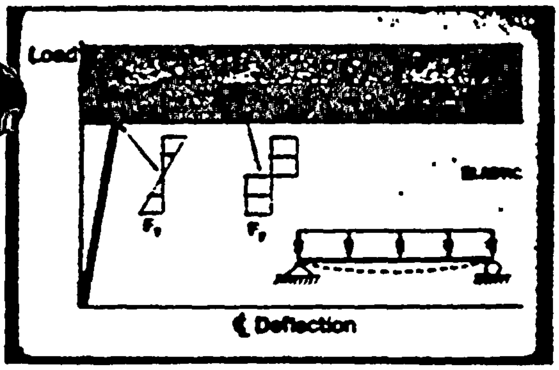
* Curves 1 and 2 will generally provide the lightest beams but sometimes the fabrication and detail cost is increased because of the added bracing, stiffeners, etc.

* The proper design is the economical one, not the lightest one.

* We will look at these curves in more detail and see how they relate to the latest AISC Spec. and Supplements.

* Of course, shear and deflection can also affect the design.

Slide No. 2-9



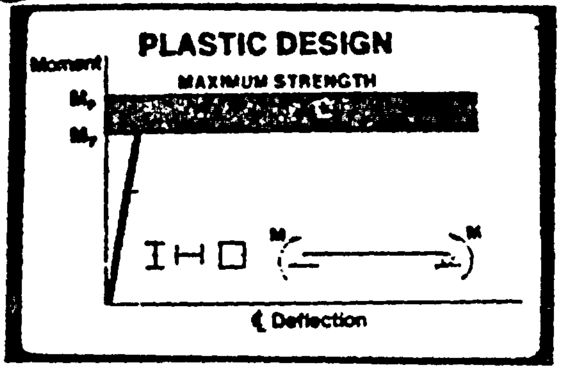
* Curves 1 and 2 treated together because the design provisions are basically the same.

* Local buckling and lateral buckling are controlled until significant yielding takes place.

* ASD - called compact sections.
  PD - when plastic design approaches are desired, this type of behavior must be assured.

* 1 is the most common structural situation. Load increases due to a moment gradient and strain hardening - moment varies along the length. Strain hardening strength is neglected in design 2 for a uniform moment region and is also an idealization of 1.

Slide No. 2-10



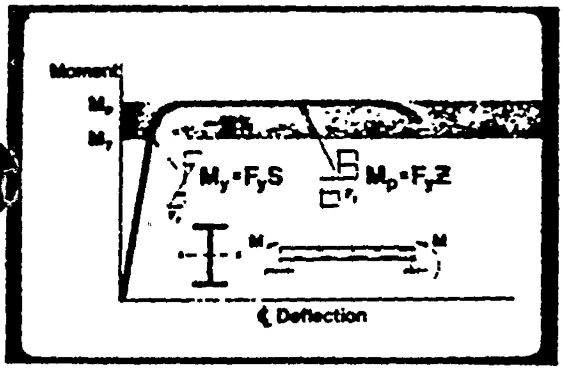
* Bending strength based on full yield of cross section. Takes full use of each type of section.

* Load factor = 1.7 for gravity loads regardless of type of cross section.

* Not concerned with some slight local yielding at working load because yielding will always occur due to residual stress, stress concentration, erection, etc. Also, once one cycle of loading and unloading occurs, further response is elastic.

* Maximum strength without strain hardening.

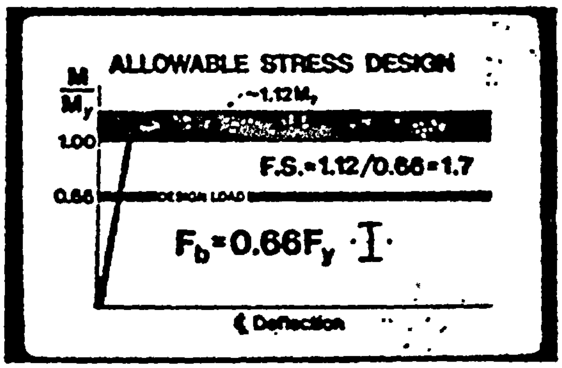
Slide No. 2-11



* H-Shape bent about x-x axis. Spacing of lateral bracing and width-thickness ratios of flange and web small enough to avoid local buckling until the entire cross section has yielded.

* On the average, plastic strength about 12% higher than 1st yield for H-shape sections.

Slide No. 2-12

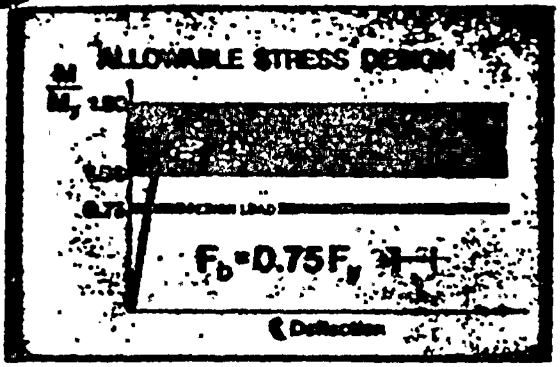


* Basing the factor of safety on full yielding of the cross section, not first yield, F.S. = 1.7.

* Actual design differs from plastic design in that only limited inelastic deformation is counted on.

* However, almost all provisions (compact sections) in ASD are based on this higher strength.

e No. 2-13



* Allowable bending stress is increased to 0.75 F_y when bending occurs about the weak axis because of large reserve strength beyond first yield (50% here).

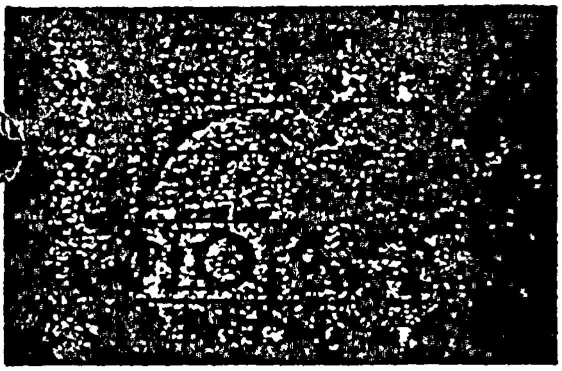
* Still has more than adequate F. S. = 2.0.

* Margin of safety is provided against yielding at work load.

* Use .75F_y for sections with good reserve strength like solid sections.

* Do not use for box or tubular members.

Slide No. 2-14



* Round sections subjected to bending reach their ultimate capacity in one of three basic failure modes;

  1. For very thick sections the compressive capacity of the material is reached, which means that large distortion occurs with no drop-off in the load.

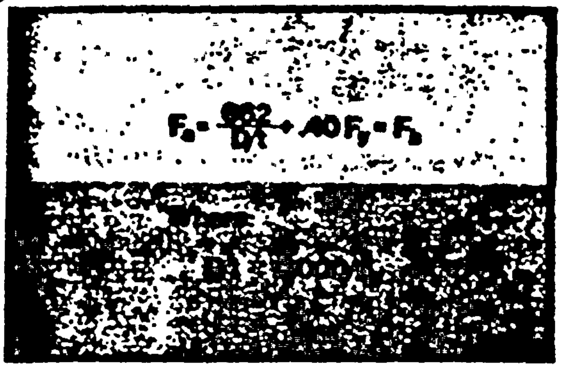
  2. Thinner round sections fail by excessive ovalization of the cross section. This is a type of inelastic instability problem in which the decrease in moment capacity caused by the reduction in the section modulus due to flattening occurs more rapidly than the increase in moment.

  3. Very thin cylinders fail in a diamond shaped local buckling pattern.

* The division between ovalization and local buckling is taken as 3300/F_y which is the D/t limit given in the AISC Specification for compact circular sections.

* Ovalization will not impair the development of the plastic hinge in tubes with D/t less than 1300/F_y. See Sherman, D. R., "Tentative Criteria for Structural Applications of Steel Tubing and Pipe", AISI, Washington, D.C. 1976

Slide No. 2-15



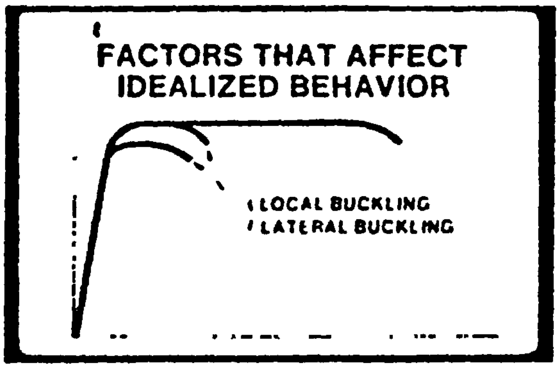
* What the allowable bending stress is for circular sections that exceed the D/t = 3300/F_y limit?

* Appendix "C" of the AISC Spec. gives this formula for allowable axial stress when tubular members do not meet the D/t requirements necessary for stiffened elements subject to axial compression.

* This same formula is applicable to an allowable bending stress as long as the D/t ratio does not exceed 13,000/F_y.

NOTE:
See page 5-95 of Appendix C for background. Also, Page 9 of "Tentative Criteria for Structural Applications of Steel Tubing and Pipe", D. R. Sherman, AISI Publication.

Slide No. 2-16



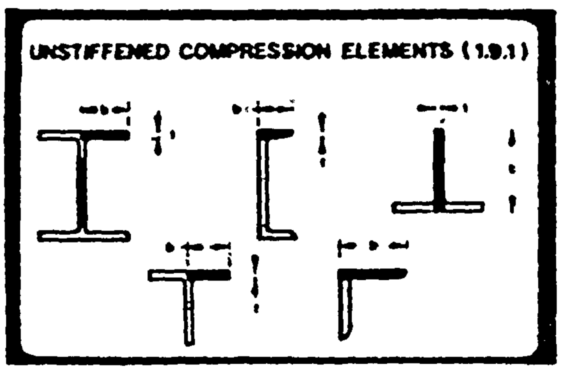
* The two previous solutions are based on the idealized behavior (shown solid).

* To achieve this behavior, lateral buckling and local flange and web buckling must be controlled.

* One or both will always eventually cause failure of the member, but only after the structure becomes useless because of excessive deflection.

* Sections that satisfy the width-thickness and bracing requirements are called compact sections.

Slide No. 2-17

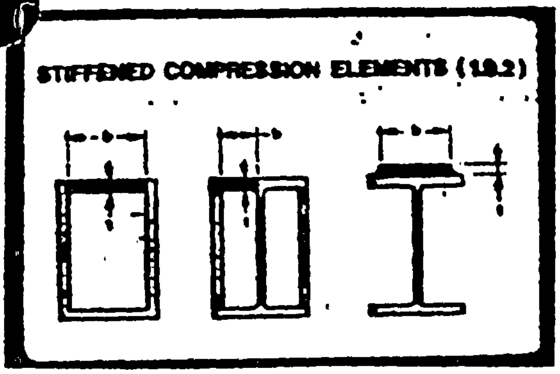


* Width-thickness ratios are defined in Spec. Section 1.9.1.

* Thickness is average for elements like flanges of channels and "I" (S Shapes).

* Appendix C used when values in Section 1.9.1 are exceeded.

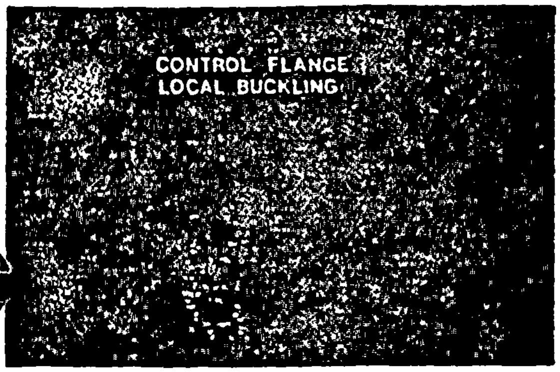
Slide No. 2-18



* Stiffened compression elements are also defined in Section 1.9.2 of Specs.

* Sections shown can be compact.

Slide No. 2-19



* Relationship between width-thickness ratio of unstiffened compression flanges and yield stress.

* Give values for A36 steel. 21.6 for ASD/LRFD and 17 for PD.

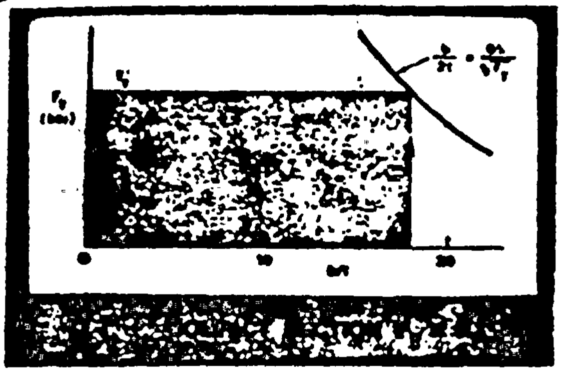
* The differences in ASD and PD requirements is that PD may require large rotation capacity - thus local buckling more critical.

* ASD requirements are based on a compact section that assumes an inelastic rotation capacity of 3. When a higher rotation capacity is required, then the b/t requirements would be tightened to those of plastic design.

* Experimental data are limited for the very high strength steels, so use of compact behavior and plastic design only for steels up to F_y = 65 ksi.

* Combinations of F_y and b/t that fall in the shaded area satisfy the AISC compactness requirements.

Slide No. 2-20

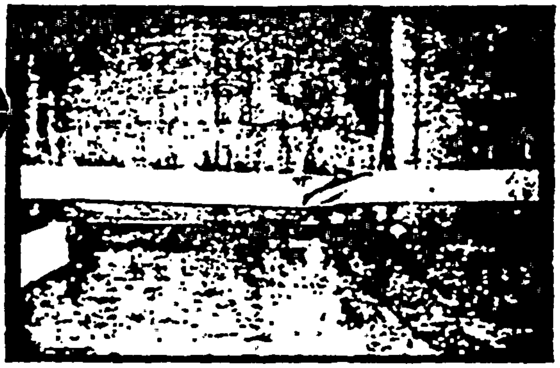


* For a given b/t, the yield stress which just satisfies the equation can be calculated. It is called F'_y.

* For each rolled section, the b/t is known so F'_y can be determined. If the yield stress is greater, you do not satisfy the equation and cannot use $0.66\ F_y$. If the yield stress is less than F'_y, the cross section satisfies the requirements for a compact section to control local flange buckling.

* Values of F'_y are tabulated in the AISC Manual for rolled sections under the PROPERTIES FOR DESIGNING.

Slide No. 2-21



* Web slenderness requirements for compact sections try to ensure web yielding before web buckling starts.

* Web buckling depends on the stress distribution in the web; the presence of axial force in addition to the moment alters the stress in the web, so compact section criteria for webs includes effect of axial stress.

Slide No. 2-22

* If axial load is zero, $d/t = 640/\sqrt{F_y}$. Half of the web is in tension, the other half is in compression.

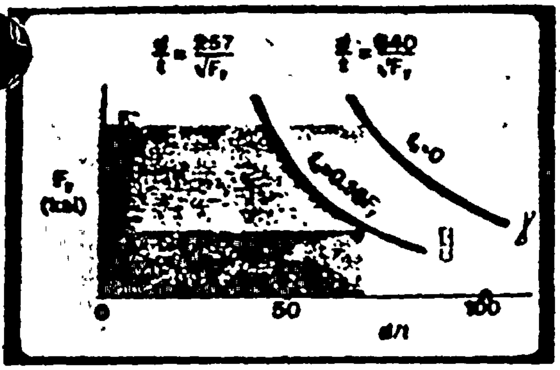
* When $f_a/F_y \geq 0.16$, entire web will have a uniform compressive stress distribution at ultimate load.

* Give values of d/t limits for A36 steel.

  - No axial load, $d/t \leq 107$
  - $f_a/F_y \geq 0.16$, $d/t \leq 43$

* The formula shown is for ASD, and is also applicable to PD when no axial load is present. An inelastic rotation capability of 3 is assured. For a greater rotation capacity, d/t is limited to $412/\sqrt{F_y}$ in PD.

Slide No. 2-23



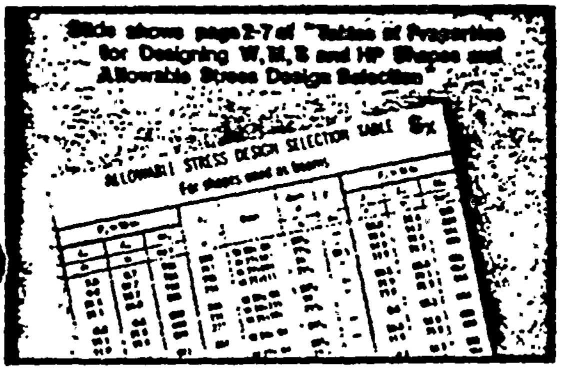
* If no axial load is present $F_y \le F''_y$, the web is compact.

* F''_y is the hypothetical yield stress above which the section is non- compact due to web criteria.

* When axial load is present and $F_y < F'''_y$, the web is compact. If F_y is between F'''_y and F''_y, check the formula for d/t requirements.

* Actually, all shapes now available conform to $d/t \le 640/\sqrt{F_y}$ with available steels. Therefore F''_y is not required.

Slide No. 2-24



* Slide shows page 32 of "Tables of Properties for Designing W.M.S. and HP Shapes and Allowable Stress Design Selection.

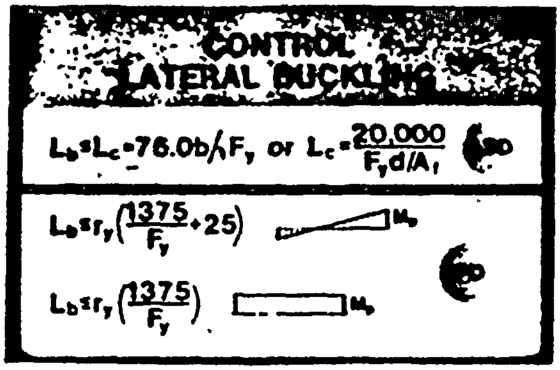
Slide No. 2-25

* Lateral buckling affected by:

  - Steel strength.
  - Unbraced length of compression flange.
  - Moment gradient.

* Bracing must be spaced close enough to prevent lateral buckling from significantly affecting the idealized behavior.

Slide No. 2-26



* Lateral buckling control is not completely understood to date as evidenced by the vast difference in appearance between the formulas for ASD and PD.

* ASD Formulas involve four different cross sectional properties, and the checking of two formulas.

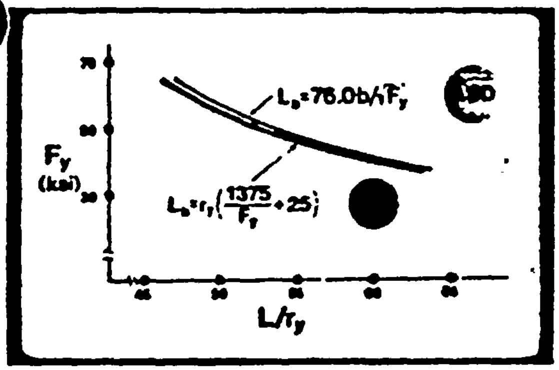
* Governing L_c listed in AISC Manual in Beam Load Tables for beam-type cross sections.

* Formulas are based on thorough tests.

* Uniform Moment if $-0.5 > M/M_p > 1.0$.

* ASD makes no distinction between uniform moment and moment gradient but PD.

Slide No. 2-27



* ASD/PD provisions shown are almost identical.

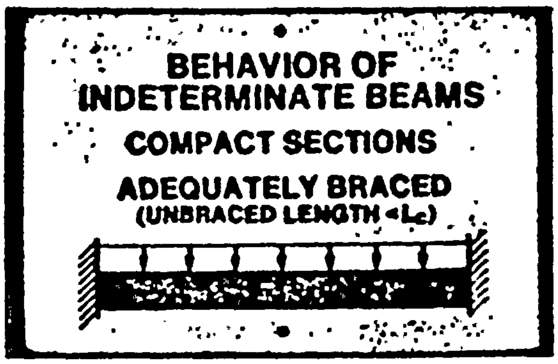
* PD curve is for moment gradient case.

* Only for the case of uniform moment will plastic design require L_c, ASD may require a shorter bracing spacing than that for ASD.

* Since ASD requires checking another formula which could govern L_c, ASD may require a closer bracing spacing than PD.

* Safe region is below curves.

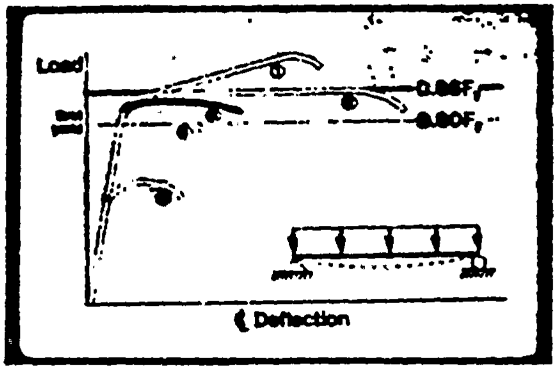
Slide No. 2-28



* ASD - limit of usefulness is based on yielding of the cross section at one point only.

* PD - limit of usefulness = ultimate load.
  - apply load factor to working load. (1.7)

No. 2-29



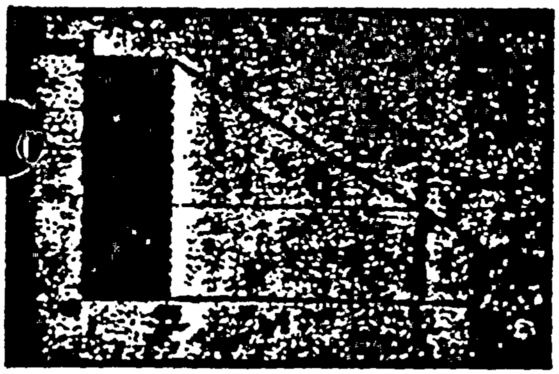
* Behavior illustrated by curve (3) should be expected if lateral buckling is controlled but flange or web slenderness ratios exceed compactness limits.

* PD not permitted. No moment redistribution permitted.

* ASD permits gradual change in allowable stress between $.6F_y < F_b < .66F_y$ when flange compactness limits are exceeded.

* Historically the AISC Spec. does not permit local buckling below 1st yield in hot rolled members.

Slide No. 2-30



* Shows local buckling criteria in AISC Spec.

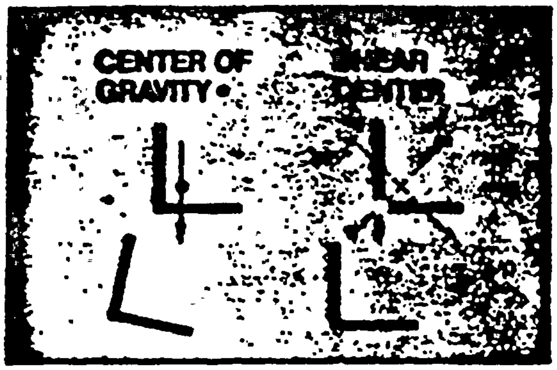
* $F_b = 0.66\ F_y$ for b/t up to $65/\sqrt{F_y}$.

* Straight line transition to $F_b = 0.6F_y$ at b/t $= 95/\sqrt{F_y}$.

* Appendix C for b/t > $95\ \sqrt{F_y}$.

* Here, b is the width of the unstiffened element.

Slide No. 2-31



* Now is a good time to discuss the angle which is a very common member in building construction, but has limited design guidance available to the engineer.

* What design criteria that is available consists mainly of empirical extrapolations of solutions for other shapes and continued misconceptions about non-principal axis loading and shear center eccentricities.

* The design condition which presents the designer with a major information gap occurs when the angle is used as a laterally unsupported beam.

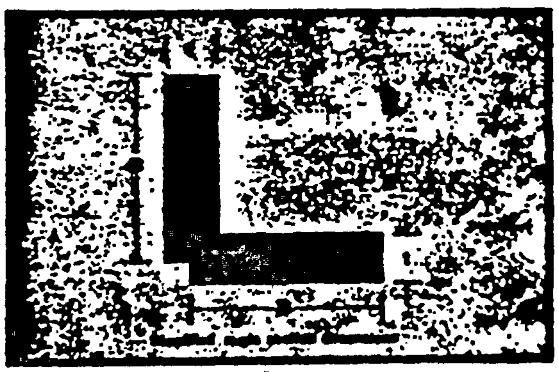
* The angle is a difficult shape for stress analysis.

* The principal axes of the cross-section do not coincide with common loading directions and any routine loading will therefore cause biaxial bending deflections which are not in the same plane as the applied loads.

* To further complicate the problem, the shear center is not at the centroid and is not on the line of most major applied loads. Thus most loads will cause the cross-section to twist and to deflect out of its loading plane.

* Finally, commonly used end connections are usually eccentric because of the lack of symmetry of the cross-section.

Slide No. 2-32



* There was a study made in Australia which developed some rational, simple formulas for the design of laterally unsupported angles in bending. See hand-out material.

* The theoretical study had these parameters:

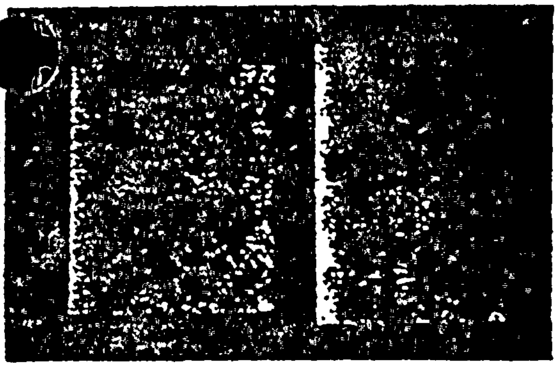
1. The loading resulted in uniform moment along the entire laterally unsupported span, which will produce the most critical lateral buckling situation.

2. The angle lengths were assumed to be completely laterally unsupported.

3. The following slides are applicable to equal leg angles, although similar research results on unequal leg angles are available. See hand-out material.

4. The sections are approximated by the dual rectangle idealization shown. This linearized section ignores fillets and toe radii, but can be made to reproduce actual member properties very precisely by adjusting the idealized leg length, B, to produce an exact similitude for some chosen geometrical property (such as area). The assumption, therefore, is not critical and is necessary in order to obtain a solvable set of equations.

Slide No. 2-33



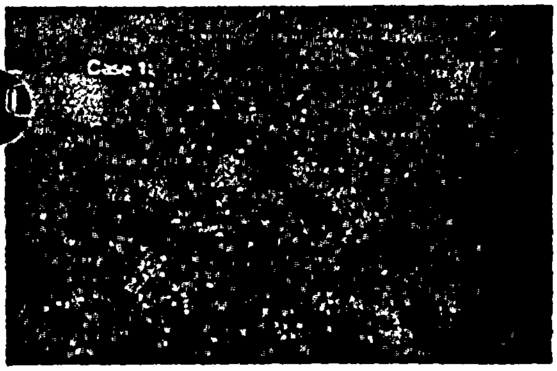
* The following slides will show how practical angle sections are usually governed by stress ($F_b = 0.6F_y$) or be deflection limitations rather than buckling.

* Case I is a common design situation, so let's briefly examine the Australian work for this case.

* Loading is as shown with M_e being the applied moment, which is resolved into components about the major principal axis U, and the minor principal axis V.

* If the maximum stress were calculated without resolving the applied load into U and V components, the result could be unconservative by as much as 50%.

Slide No. 2-34



* The Australian study showed that for laterally unrestrained angle beams the following relationships apply:

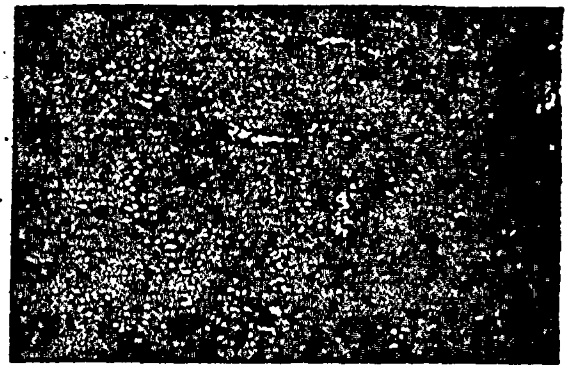
* The stress at any point in the section is
$$\frac{3M}{b^3 t}\left[(V + 4U)\,\phi_1 + V - 4U \right]$$

  where V and U are cordinate points normal to the principal axes.

* Max. Section Stress is as shown.

* Angle of twist ϕ_1 is made up of ;
  ϕ = twist due to applied loads
  ϕ_e = initial angle of twist due to imperfections.

Slide No. 2-35



* The top equation shows that the stress in the section is a linear function of the amount of twisting (ϕ) to which it has been subjected.

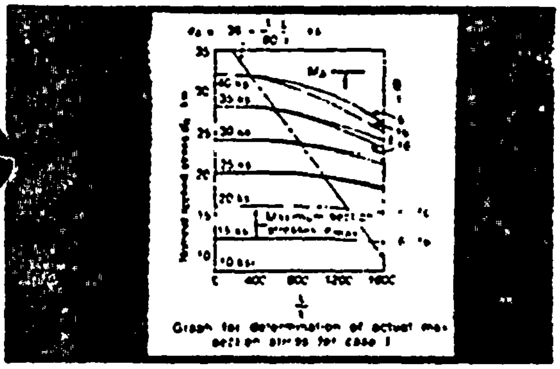
* ϕ also has a direct relation to

$L/_t$, $\frac{\sigma_a}{E}$ and $\frac{B}{t}$

Where: $\sigma_a = \frac{M_a}{Z_a}$ does not include stress due to twist.

Z_a is elastic section modulus (Australian nomenclature).

Slide No. 2-36



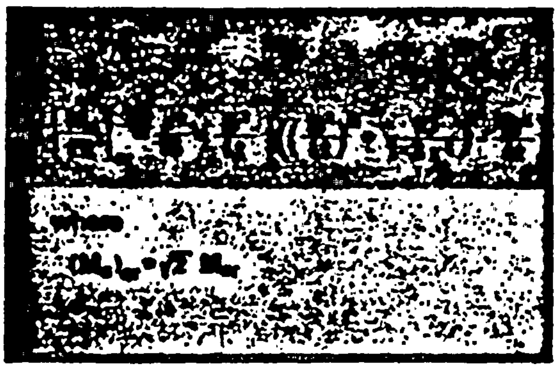
* Thus it is possible to produce curves of σ_a against L/t with contours of σ_{max}, the maximum section stress.

* Twisting may be ignored if,

$$\sigma_a \leq (38 - \frac{1}{60} \times \frac{L}{t})\ \text{ksi}$$

* Later research on unequal leg angles confirms, in general, $\sigma_{max} = 1.25\ \sigma_a$.

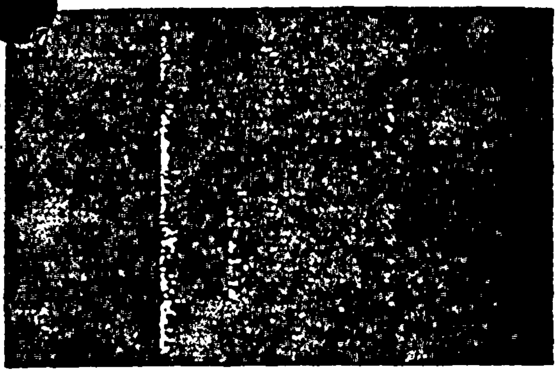
Slide No. 2-37



* An alternative method of angle beam design for Case I is to consider the critical buckling moment given by this formula .... where the critical applied moment is equal to $\sqrt{2}$ times the critical buckling moment.

* The dimensionless parameters $\frac{M_a}{Et^3}$ & $\frac{B^2}{Lt}$ are then used to draw the critical buckling curve.

Slide No. 2-38



* This curve allows the estimation of the critical applied moment for a given length and section.

* The horizontal lines represent the values of $\frac{M_a}{Et^3}$ necessary to produce a stress of $3F_y$, for various $\frac{B}{t}$ ratios.

* It was shown that failure stresses will be unaffected by elastic buckling if the calculated buckling stress is at least three times the material yield stress.

* Shaded area shows design range. For instance, with B/t of 16 and a stress of 3Fy, $\left[\frac{L}{t}\right]\cdot\left[\frac{t}{B}\right]^2 = 2.7$.

  Therefore, $\frac{L}{t} = (16)^2 \times 2.7 = 690$. Similar calculations for other B/t ratios can be made.

Slide No. 2-39

Case	B/t	Range for $F_b = 0.66\,F_y$
F=36 ksi	6	0<L/t<990
	11	0<L/t<850
	16	0<L/t<690

* Therefore, Australian research indicates that allowable bending stress F_b may be taken as $0.66\ F_y$ for these limitations on $B/_t$ and $L/_t$.

* It has been practice in U.S. to use $F_b = 0.6F_y$.

* At these high stresses, deflection may control.

Slide No. 2-40

* The critical stress corresponding to the critical applied moment can be obtained (upper equation), and then converted into a safe bending stress (F_{bc}) thru use of these two formulas from the Australian Steel Structures Code AS CA 1.

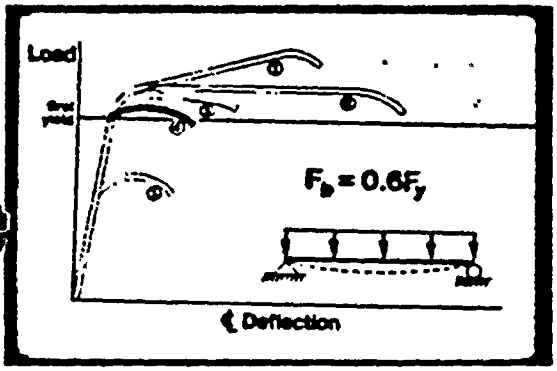
* Again, shaded area represents design range. As before $\left[\frac{L}{t}\right]\cdot\left[\frac{t}{B}\right]^2$ can be seen as approximately 2.7.

Slide No. 2-41

* The result of converting the critical stress into a safe bending stress is shown here in graphic form, which may be used directly for design.

* A copy of the Australian research report is enclosed within the handout you received. Hopefully it will assist you when designing angle beams in the future.

Slide No. 2-42



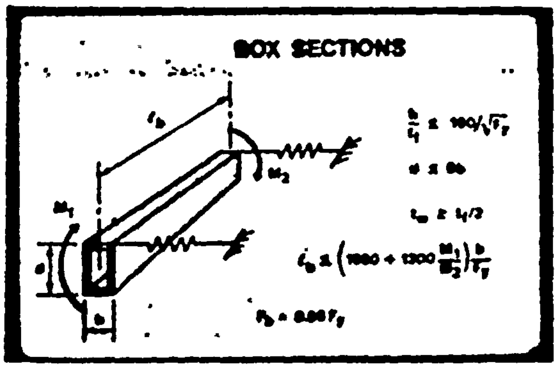
* Curve 4 is typical of sections with non-compact webs - welded girders in general.

* Also typical of box girders that are unbraced laterally.

* $F_b = 0.6\ F_y$.

* Curve 5 is typical of beams which fail by local or Lateral Torsional Buckling (LTB) and will be covered in a later lecture.

Slide No. 2-43



* Box sections may be compact.
  Also less susceptible than W-shapes to lateral torsional buckling.

* Criteria for compactness shown

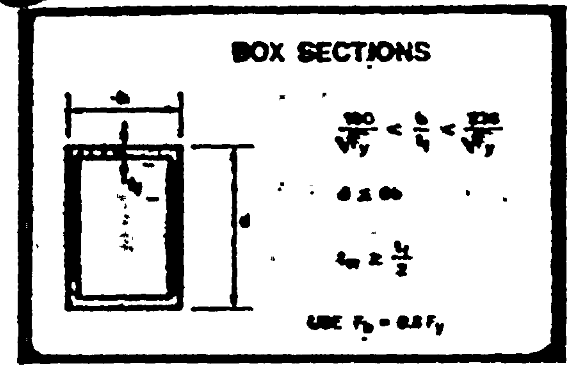
  b/t less than $190/\sqrt{F_y}$
  $d \leq 6b$ and $t_w \geq t_f/2$

* Also a bracing requirement which takes into account moment gradient. Moments shown are in plane of beam.

* M_1/M_2 same as defined in other part of Spec.

* L_b need not be less than $1200(b/F_y)$.

Slide No. 2-44

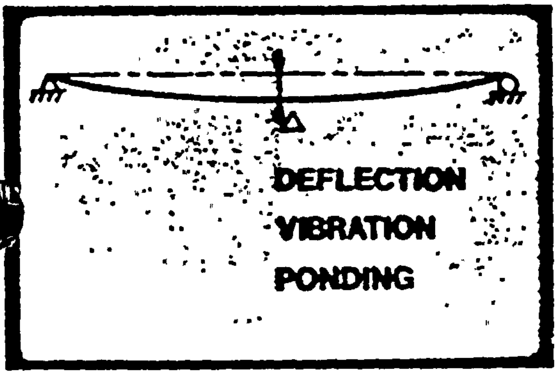


* If box sections do not meet compactness requirements use $F_b = 0.6\ F_y$.

* No lateral torsional buckling consideration if d less than 6b, and $t_w \geq t_f/2$.

* Unbraced length does not affect carrying capacity. Deflection will govern with very long spans.

Slide No. 2-45



* The design of beams in a floor or roof system would not be complete without some attention to Deflection, Vibration and Ponding. Sometimes these are criteria for design rather than stres. .

* While the Specification does require that Deflection, Vibration and Ponding be considered the only precise limits enumerated are the 1/360 of the span live load deflection for beams supporting plaster ceilings and the Ponding Formulas to be checked for flat roofs. We will look at the ponding formulas in detail later.

* Deflection limits must rest on the sound judgment of the designer and the experience of the behavior of similar structures. The Commentary to the AISC Specification gives as a guide the following:

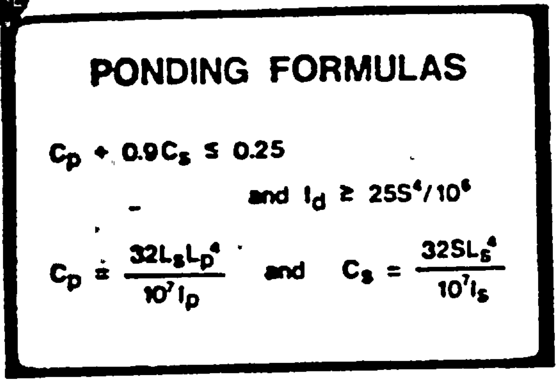
  Fully stressed floor beams and girders; F_y depth not less than $F_y/800$ times the span.

  Fully stressed roof purlins (except in flat roofs) depth not less than $F_y/1000$ times the span.

  For A36 steel these recommendations work out to be approximately 1/22 for floor beams and 1/28 for the roof purlins.

* Large open floor areas free of partitions or other sources of damping may be susceptible to transient vibration due to pedestrian traffic. While there are some design methods available to check a floor system for vibration susceptibility they necessarily involve trying to evaluate the difficult problem of human perception of vibration. The Commentary recommends as a guide the depth of a steel beam be not less than 1/20 of the span where a problem of perceptible transient vibration might be suspected.

Slide No. 2-46

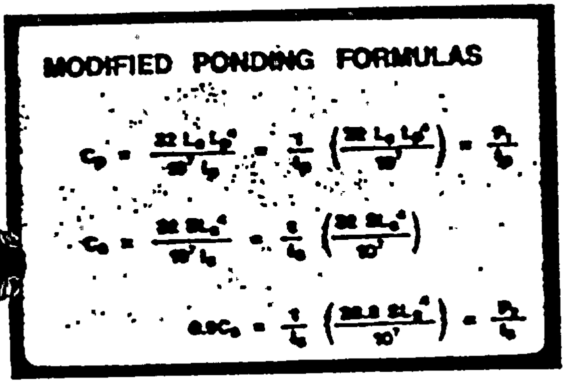


* Spec. Sect. 1.13 gives approximate but conservative formulas for ponding.

* Point out the more exact method in the Commentary.

* C_p and C_s are ponding coefficients.

Slide No. 2-47

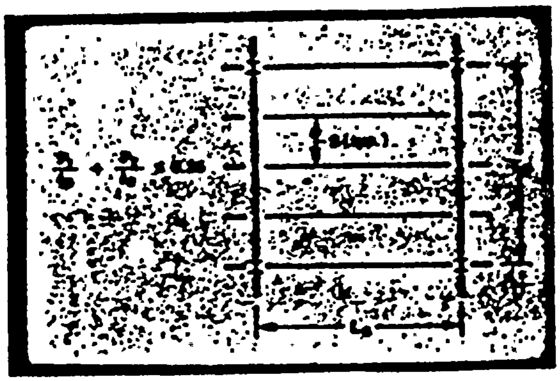


* Discuss the Modified Ponding Formulas.
  Show how they were derived.

  (AISC Engineering Journal - First Quarter, 1973, Page 26)

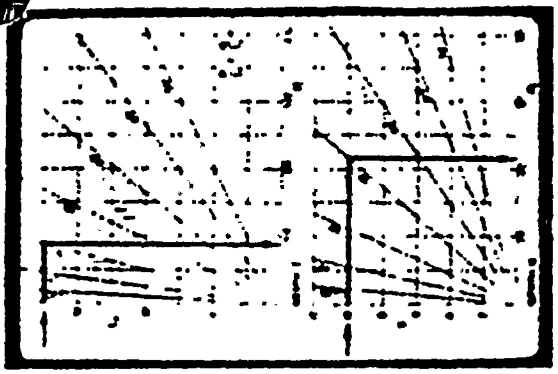
* Modified Ponding Formulas derived by Burgett, "Fast Check for Ponding", Eng. Journal, 1st Quarter, 1973.

Slide No. 2-48



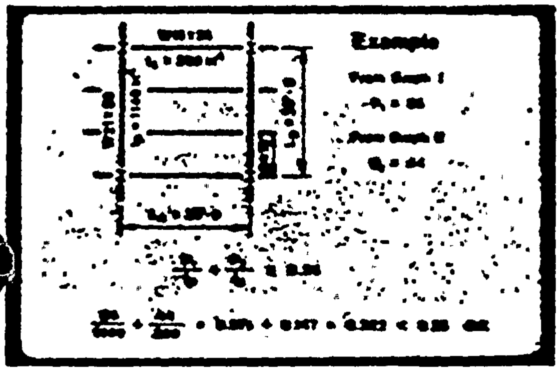
* Definition of symbols shown on typical roof framing plan.

Slide No. 2-49



* Graphs I and II have been developed to determine P_1 and P_2 which are available in Burgett paper.

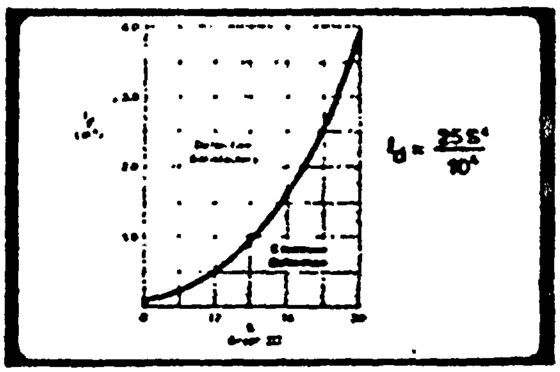
Slide No. 2-50



* Design Example

* Illustrates the use of Graphs I and II.

Slide No. 2-51



* Illustrates the use of graph III and the check for steel deck.

The design of laterally unsupported angles

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1. Introduction

The steel angle is a common and almost traditional member in building construction. Its popularity stems from its relative lightness and compactness and the ease with which it can be connected to other members. In view of its long and widespread use it is surprising to find that little is known of many major aspects of its performance as a structural member. In these areas design guidance is only available to a limited extent and consists mainly of empirical extrapolations of solutions for other sections[1] and continued misconceptions about non-principal axis loading and shear centre eccentricities.

The behaviour of angles as compression members has been studied relatively extensively (e.g. [2, 3, 4]) as a result of their widespread use in such structures as transmission towers. These towers are usually precisely analysed[5] for actual failure under well defined load factors and an accurate knowledge of member load capacity has been essential. Even here, however, the underlying research has frequently been highly empirical with strut load capacities given for each member size under practical field conditions[6].

The case which presents the designer with his current major information gap occurs when the angle is used as a laterally unsupported beam. For example, the S.A.A. Steel Structures Code AS CA1[7] states in Rule 5.4.3:

> 'The Standards Association of Australia is not prepared at this stage to make recommendations for angles which are not supported laterally.'

The British Code permits its standard beam rules to be used for angles, but the technique developed can not be rationally defended[8, 9] and does not lead to consistent design solutions. The U.S. steel design specification[10] does not specifically cover the case.

The logical question to ask at this stage is why the problem of the laterally unsupported angle used as a beam has remained without a practical solution for so long. The answer is, basically, that although the angle is a very simple section to the lay-

man and the producer, it is a difficult one for the stress analyst. The principal axes of the cross-section do not coincide with common loading directions and any routine loading will therefore cause biaxial bending deflections which are not in the same plane as the applied loads. To further complicate the problem, the shear centre is not at the centroid and is not on the line of most major applied loads. Thus most loads will cause the cross-section to twist and to deflect out of its loading plane. Finally, common end connections are usually eccentric because of the lack of symmetry of the cross-section.

2. Current Investigations

The purpose of the current investigation is to develop rational but simple formulas for the design of laterally unsupported angles in bending. This should help fill the present, previously quoted, void in the S.A.A. Steel Structures Code, CA1, and thus permit the more widespread use of angles in building construction.

The loading case to be considered will be a uniform moment along the entire laterally unsupported span. This will produce the most critical lateral buckling situation[11] and will therefore give results which will be safe for any other bending moment distribution. The same uniform moment basis is used for the other lateral buckling rules of CA1[12, 13]. The lengths under consideration are assumed to be completely unsupported and the solutions may therefore be applied to both fully unsupported beams or restrained beams between restraint points.

Later work will include an experimental examination of various aspects of the problem. However, this article will be confined to a theoretical derivation of design rules.

Solutions are only presented for equal angles (leg lengths equal). Similar solutions can be obtained for unequal angles, but the complete asymmetry of these latter sections produces algebraically involved results which tend to obscure the basic underlying principles.

The range of equal angles produced by BHP are given in [14]. The sections are approximated by the dual rectangle idealisation shown in Fig.1. This linearised section ignores fillets and toe radii, but can be made to reproduce actual member properties very precisely by adjusting the idealised leg length, B, to produce an exact similitude for some chosen geometrical property (such as area). The assumption, therefore, is not critical and is necessary in order to obtain a solvable set of equations.

3. Notation and sign convention

The notation to be used is:

- B = Width of angle leg.
- A, C, D = Constants of integration.
- E = Young's Modulus.
- F = Design stress.
- F_c = Critical buckling stress.
- F_b = Maximum permissible bending stress.
- F_y = Yield stress.
- G = Modulus of rigidity (shear or torsion modulus)
- I_u = Second moment of areas about UU axis.
- I_v = Second moment of area about VV axis.
- I_w = Warping moment of area.
- K_T = St. Venant torsional constant.
- $\overline{K}$ = Torsional component of the normal stress (see eq.5.4).
- L = Length of span.
- M = Component moment of the applied moment.
- M_{cr} = Critical buckling moment.
- M_y = Applied moment about Y axis + moment due to the dead weight of the beam.
- S = Shear centre.
- U Denotes the major principal axis.
- V Denotes the minor principal axis.
- W Denotes the polar axis.
- Y, X Denotes axes through the centroid, parallel to an angle leg.
- Z = Section modulus.
- Z_y = Section modulus about same axis as M_y.
- Z_v = Section modulus through the V axis.
- c = Centroid location.
- t = Thickness of angle leg.
- u = U—U axis co-ordinate.
- v = V—V axis co-ordinate.
- u_o = Shear centre co-ordinate.
- v_o = Shear centre co-ordinate.

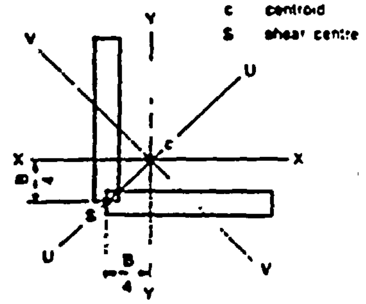


Fig.1 (a). Orientation of axes and locations of centroid and shear centre.

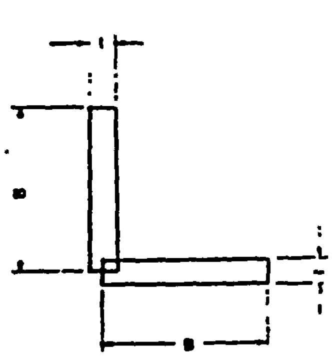


Fig.1 (b). Simplified angle section dimensions

w = Distance measured along the length of the beam.
σ = Actual section stress.
σ_b = Stress calculated using conventional beam formula:

$$\sigma_b = \frac{M_v}{Z_v}$$

σ_{cr} = Critical buckling stress.
ϕ = Angle of twist.
ϕ_o = Initial angle of twist due to imperfections.
α = Coefficient in solution of differential equations.
$'$ = Differentiation with respect to w.

This notation is coupled with the sign convention shown in Fig.2.

4. Loading cases

The behaviour of the beam is dependent on the axis about which the moment is applied, Fig.2. Four loading conditions are illustrated in Fig.3. These conditions can be used vectorially to represent all possible cross-section loadings. Taken individually they are:

Case I: Moment applied about an axis through the shear centre parallel to one leg.

Case II: Moment applied about the UU axis (strong axis).

Case III: Moment applied about the VV axis (weak axis).

Case IV: Moment applied about an axis midway between the UU axis and the YY axis.

Each of these cases will now be individually studied.

5. Case I

The problem to be solved is illustrated in Fig.3(a) and Fig.4. Galambos[13] has shown that for this case the following equations apply*:

Bending in the V Direction:

$$(3.81)\quad EI_u v'' + M\phi = -M \qquad \text{—(5.1)}$$

Bending in the U Direction:

$$(3.82)\quad EI_v u'' + M\phi = M \qquad \text{—(5.2)}$$

Torsional Equilibrium:

$$(3.83)\quad EI_\omega \phi''' - (GK_T + \bar{K})\phi' + Mu' + Mv' = o \qquad \text{—(5.3)}$$

where the symbols are as defined in Section 3 and the primes indicate differentiation with respect to w, the distance along the beam.

The equations are derived from the following set of assumptions:

(a) The material is elastic.
(b) The member is straight and prismatic.
(c) The cross-section is thin walled and open.
(d) Deflections are small.

All the constants are readily calculable (see Sect.4) with the exception of I_ω and $\bar{K}$. It has been shown in [18] that warping is insignificant for angle sections, therefore, $I_\omega = o$. $\bar{K}$ can be determined from the following constitutive equations given by Galambos:

$$(3.85)\quad \bar{K} = M(\beta_v - \beta_u) \qquad \text{—(5.4)}$$

$$(3.13)\quad \beta_v = \frac{1}{I_v}\int_s v(v^2 + u^2)t\,ds - 2v_o \qquad \text{—(5.5)}$$

$$(3.85)\quad \beta_u = \frac{1}{I_u}\int_s u(u^2 + v^2)t\,ds - 2u_o \qquad \text{—(5.6)}$$

*The left hand equation numbers correspond to those in Galambos[13].

For the idealised section (Fig.1)

$$v = \pm\left(u + \frac{B}{2\sqrt{2}}\right)$$

and $ds = \pm\sqrt{2}du = \sqrt{2}dv$

Integration of equations (5.5) and (5.6) yields

$\beta_v = o$ and $\beta_u = \sqrt{2}B$

whereupon:

$$\bar{K} = -\sqrt{2}BM \qquad \text{—(5.7)}$$

for equal angle sections.

The angle of twist ϕ may now be determined by substituting this solution into eq. (5.3) to give:

$$-(GK_T - \sqrt{2}BM)\phi' + Mu' + Mv' = o \qquad (5.8)$$

Differentiating this and substituting values of u'', v'' from (5.1), (5.2) gives:

$$\lambda_1\phi'' + \lambda_2\phi = -\lambda_3 \qquad \text{—(5.9)}$$

$$\lambda_1 = GK_T - \sqrt{2}MB \qquad \text{—(5.10)}$$

$$\lambda_2 = \frac{M^2}{E}\left(\frac{1}{I_v} + \frac{1}{I_u}\right) \qquad \text{—(5.11)}$$

$$\lambda_3 = -\frac{M^2}{E}\left(\frac{1}{I_v} - \frac{1}{I_u}\right) \qquad \text{—(5.12)}$$

The general solution is:—

$$\phi = A\cos\alpha w + D\sin\alpha w - \frac{\lambda_3}{\lambda_2} \qquad \text{—(5.13)}$$

where $\alpha = \left(\frac{\lambda_2}{\lambda_1}\right)^{1/2}$

with boundary conditions:

$\phi_{(w=0)} = \phi_{(w=L)} = 0$

one obtains

$$\phi_{max} = \frac{3}{5}\left(1 - \frac{1}{\cos\alpha L/2}\right) \qquad \text{—(5.14)}$$

6. Case I. Critical buckling

For Case I the critical buckling condition occurs when:

$$\alpha L = \pi$$

as at this value

$$\phi = -\infty \text{ (see eq. 5.14).}$$

Since

$$\alpha L = \left(\frac{\lambda_2}{\lambda_1}\right)^{\frac{1}{2}}.L$$

the critical moment is given by:

$$\pi = \frac{7{\cdot}65ML}{B^2Et^2\left(1 - \frac{5{\cdot}5M}{Et^2}\right)^{\frac{1}{2}}} \qquad \text{—(6.1)}$$

or

$$\left[\frac{M_v}{Et^2}\right]_{cr} = \frac{\pi^2}{15}\cdot\frac{B^2}{Lt}\left[\left(\left(\frac{B^2}{Lt}\right)^2 + \frac{10}{1{\cdot}3\pi^2}\right)^{\frac{1}{2}} - \frac{B^2}{Lt}\right] \qquad \text{—(6.2)}$$

where

$(M_v)_{cr} = \sqrt{2}M_{cr}$ = the critical applied moment and the dimensionless parameters $\frac{M_v}{Et_2}$ and $\frac{Lt}{B^2}$ are used to draw the critical buckling curve (Fig.5a). This curve allows an estimation of the critical applied moment for a given length and section. The horizontal lines represent the values of $\frac{M_v}{Et^2}$ to produce a stress of $3F_y$ (where F_y is the material yield stress), for yield stresses of 52 and 36 ksi and $\frac{B}{t}$ ratios of 6 and 18.

It has been shown[18, 19] that failure stresses will be unaffected by elastic buckling if the buckling stress is at least three times the material yield stress.

Thus, it can be established that F_b may be taken as 0·66 F_y for the following cases:

Case	B/t	Range for $F_b = 0{\cdot}66\ F_y$
$F_y = 52$ ksi	6	$0 < L/t < 680$
	11	$0 < L/t < 570$
	18	$0 < L/t < 330$
$F_y = 36$ ksi	6	$0 < L/t < 990$
	11	$0 < L/t < 850$
	18	$0 < L/t < 690$

The critical stress corresponding to the critical moment in eq.6.2 can be obtained by:

$$\sigma_{cr} = \frac{(M_v)_{cr}}{Z} = \frac{9}{\sqrt{2}}\cdot\frac{(M_v)_{cr}}{B{\cdot}t}$$

$$\sigma_{cr} = 0{\cdot}424\pi^2\,\frac{Et}{L}\cdot\left[\left(\left(\frac{B^2}{Lt}\right)^2 + \frac{10}{1{\cdot}3\pi^2}\right)^{\frac{1}{2}} - \frac{B^2}{Lt}\right] \qquad \text{—(6.3)}$$

This stress corresponds to F_o in Rule 5.4.3 of AS CA1 and the safe bending stress F_{bc} for the beam can be calculated using eqs.(4) and (5) of those rules as the purpose of these equations is to permit such conversions to be made [12, 13]. The result of converting σ_{cr} in eq.(6.3) into F_{bc} is shown in Fig.5b, which may thus be used directly for design.

7. Case I. Stress solution

The actual maximum section stress is obtained from the stress equation, which gives the stress at any point in the section as:

$$(2.74)\quad \sigma = \frac{M_u v}{I_v} - \frac{M_v u}{I_u} + E\omega_n\phi'' \qquad \text{—(7.1)}$$

where $M_u = M(1 + \phi)$, $M_v = M(1 - \phi)$

It has been shown that the effect of warping is insignificant and since:

$$(2.62)\quad I_\omega = \int \omega_n^2 t\,ds = o \qquad \text{—(7.2)}$$

then $\omega_n = o$

Equation (7.1) becomes:

$$\sigma = \frac{M(1 + \phi)v}{I_v} - \frac{M(1 - \phi)u}{I_u} \qquad \text{—(7.3)}$$

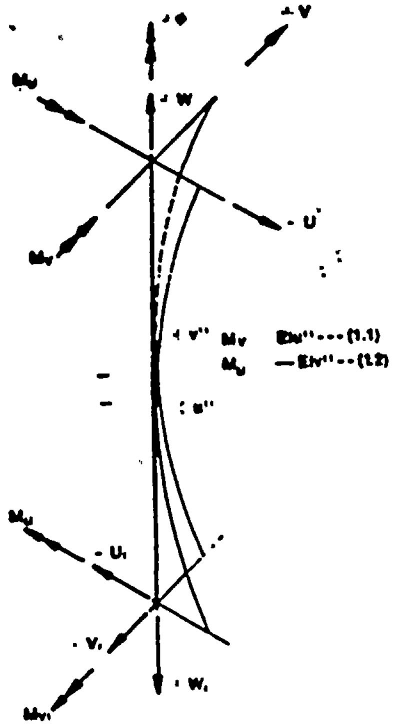
Substituting values for I_u, I_v gives:

$$\sigma = \frac{3M}{B^2t}\Big((V + 4U)\phi + V - 4U\Big) \qquad \text{—(7.4)}$$

This equation shows that the stress in the section is a linear function of the amount of twisting to which it has been subjected. The twist resulting from applied loads is given in eq.(5.14). Further twisting will result from initial eccentricities present in the unloaded angle. There are no specification limits for torsional eccentricity; however Massey[17] has measured the torsional eccentricity in steel I beams and suggests an average value of initial twist as:

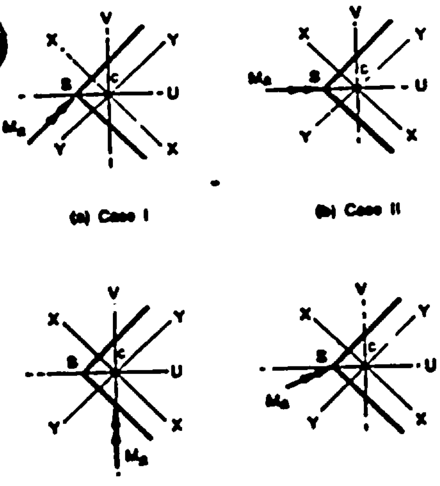
$$\phi_o = 0{\cdot}436 \times 10^{-4}L \text{ radians} \qquad \text{—(7.5)}$$

Values measured for two angle lengths are given in Fig.7 together with Massey's general estimate. The method of measurement is shown in Fig.8. The twist due to the weight stress is avoided by measuring the total twist ($\phi_1 \cdot \phi_2$) of the angle in two positions ninety degrees apart. The measured values are in agreement with Massey's equation.



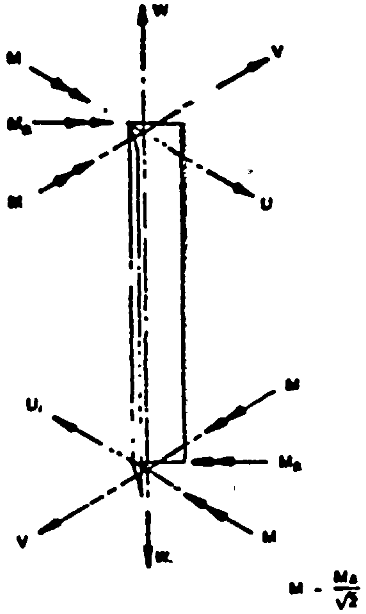
Note
Axes are drawn with W or W_1 as the outward drawn normal from the surface under consideration.

Fig.2. Sign convention



(a) Case I (b) Case II (c) Case III (d) Case IV

Fig.3. Loading for cases I to IV
(a) Case I
(b) Case II
(c) Case III
(d) Case IV



$M = \frac{M_a}{\sqrt{2}}$

Fig.4 Loading for case I

$$\left[\frac{M_a}{Et^3}\right]_{cr} = \frac{\pi^2}{15}\frac{B^2}{Lt}\left[\sqrt{\left(\frac{B^2}{Lt}\right)^2 + \frac{10}{1.3\pi^2}} - \frac{B^2}{Lt}\right] \cdots \text{eq 6.2}$$

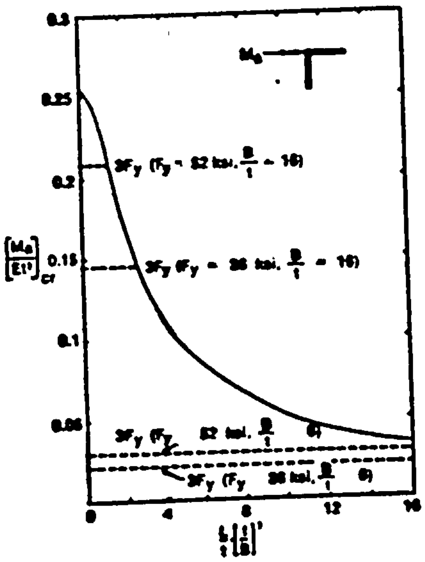


Fig.5 (a). Critical buckling curve for case I

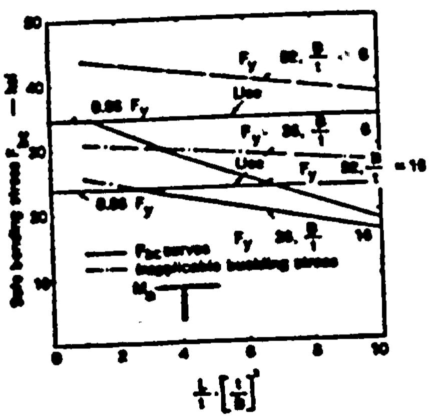


Fig.5 (b). Graph for determination of F_{bc} for case I

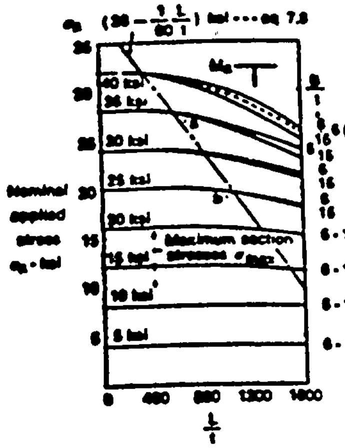


Fig.6. Graph for determination of actual max. section stress for case I

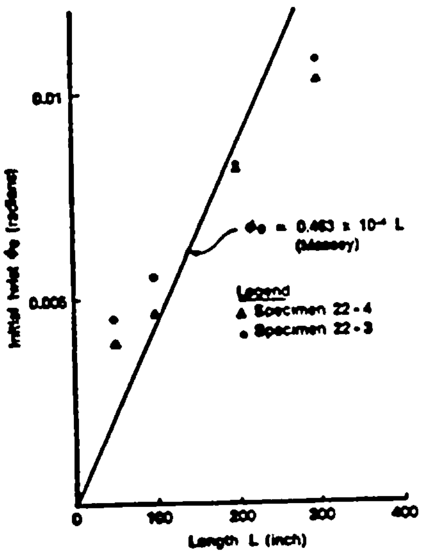
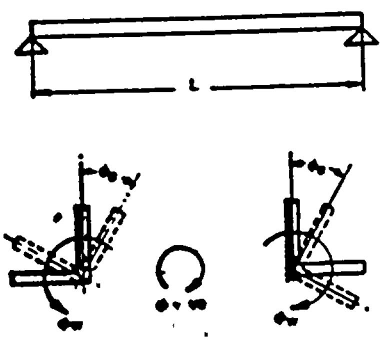


Fig.7. Initial twist ϕ_o test results for angle sections compared with Massey's relationship



$\phi_a = -\phi_w + \phi_o$ $\quad \phi_b = \phi_w + \phi_o$

where ϕ_o = initial angle of twist
ϕ_w = Angle of twist due to dead weight

hence $\phi_o = \frac{\phi_a + \phi_b}{2}$

Fig.8 Method for determination of initial angle of twist

If ϕ_o is considered, the stress equation (7.4) becomes:

$$\sigma_t = \frac{3M}{B^2t}\left((V+4U)\phi_t + V - 4U\right) \quad \text{—(7.6)}$$

If amplification effects near the buckling load are neglected

$$\phi_t = \phi + \phi_o = \frac{3}{5}\left(1 - \frac{1}{\cos \alpha L/2}\right) + 0{\cdot}436 \times 10^{-4}\left(\frac{L}{t}\right) \quad \text{—(7.7)}$$

where t in the ϕ_o part of the expression has been put equal to 1 to produce the maximum value of ϕ_o for values of $\left(\frac{L}{t}\right)$

It can be seen from Section 5 that

$$\phi = f\left(\frac{L}{t}, \frac{\sigma_a}{E}, \frac{B}{t}\right),$$

where $\sigma_a = \frac{M_x}{Z_x}$ does not include the stress due to twist. Thus, it is possible to produce curves of σ_a against L/t with contours of σ_{max}, the maximum section stress, as shown in Fig.6. Contours of σ_t (stress including initial twist) are also shown. Although the initial twist does cause a stress increase over σ_{max} for the range examined, the magnitude of this increase is small and only apparent in the graph for large values of M and $\left(\frac{L}{t}\right)$.

If the maximum section stress is calculated for Case I using conventional beam formulas and, if the applied moment is not resolved into components in the U and V axes, the calculated stress may be up to 50% less than the actual stress produced in the member. In terms of the symbols used above, σ_a may be up to 50% less than σ_{max}.

It is clear from the graph that twisting may be ignored if:

$$\sigma_a < \left(38 - \frac{1}{60} \cdot \frac{L}{t}\right) \text{ ksi} \quad \text{—(7.8)}$$

The expression is empirically determined from the form of the contours in Fig.6.

The two points 'a' and 'b' on Fig. 6 are obtained from the buckling solution given in Fig.5a, as the points where buckling does not influence the results. It is seen that the two approaches lead to similar results as 'a' and 'b' lie close to eq.(6.11). The buckling approach relies on the F_o — F_b conversion of eq.(4) and (5) of CA1, whereas the maximum stress approach is based on limiting the true peak stress to permissible values. The two solutions will therefore lead to similar but not identical results and the selection of a method will depend on the formulation of the problem.

8. Case II

Galambos[15] has shown that, for singly symmetric sections subject to the loading shown in Fig.3(b), the equations for lateral torsional buckling are:

$$(3.49) \quad EI_Y u^{iv} + M\phi'' = 0 \quad \text{—(8.1)}$$

$$(3.50) \quad EI_\omega \phi^{iv} - (GK_T + M\beta_x)\phi'' + Mu'' = 0 \quad \text{—(8.2)}$$

Since $\beta_x = 0$ (Sect.5) and warping is insignificant then

$$\lambda_a \phi^{iv} + \lambda_b \phi'' = 0 \quad \text{—(8.3)}$$

where

$$\lambda_a = GK_T \quad \text{—(8.4)}$$

$$\lambda_b = \frac{M^2}{EI_Y} \quad \text{—(8.5)}$$

The general solution is:

$$\phi'' = A \sin \alpha w + D \cos \alpha w \quad \text{—(8.6)}$$

where

$$\alpha = \left(\frac{\lambda_b}{\lambda_a}\right)^{\frac{1}{2}} \quad \text{—(8.7)}$$

Applying the end conditions of zero torsional restraining moment:

$$\phi''_{(w=0)} = \phi''_{(w=L)} = 0$$

gives

$$D = 0$$

and

$$\sin \alpha L = 0$$

The lowest critical moment occurs when:

$$\alpha L = \pi$$

i.e.

$$\left(\frac{\lambda_b}{\lambda_a}\right)^{\frac{1}{2}} \cdot L = \pi$$

or

$$M_{cr} = \frac{\pi E}{6\sqrt{1 \cdot 3}} \cdot \frac{B^2t^2}{L} \quad \text{—(8.8)}$$

This result can also be obtained using the St. Venant buckling solution,

$$M_{cr} > \frac{\pi}{L}\left(EI_y GK_T\right)^{\frac{1}{2}} \quad \text{—(8.9)}$$

Substituting $M_{cr} = \sigma_{cr} \cdot Z_x$ in eq.(8.1) gives:

$$\sigma_{cr} = \frac{\pi Et}{2\sqrt{2 \cdot 6}\, L} \quad \text{—(8.10)}$$

which is the critical elastic buckling stress for the member. Using the 'elastic critical stress to design stress' conversion of the SAA Steel Structures Code CA1, Rule 5.4.3, eqs.(4) and (5), together with eq.(8.10), allows Fig.9 to be drawn. This figure shows both the critical buckling stress curve of eq.(8.10) and the curves of the design bending stress for yield stresses of 52 and 36 ksi derived as indicated above.

It is apparent that when L/t < 200 for F_y = 52 and L/t < 300 for F_y = 36 ksi, F_b may be taken as ·66 F_y. This follows from the $F_o > 3\, F_y$ criterion used earlier.

Fig. 10 has been included to permit rapid estimation of F_b when the $\frac{L}{t}$ ratio and the yield stress are known. The maximum stress in a section may be determined directly from the applied moment and the section modulus.

9. Case III

The loading for Case III is shown in Fig. 3c. Since the moment is applied about the weak axis there is no possibility of buckling to a more stable configuration and the beam will continue to bend about this axis only. Therefore conventional beam formulas may be used. The maximum stress is given by:

$$\sigma_{max} = \frac{M_Y}{Z_Y} \quad \text{—(9.1)}$$

10. Loads not through the shear centre

Loads not through the shear centre will cause twisting of the angle section. Such loads will include the weight of the section acting through the centroid. These loads will cause an angle of twist given by:

$$\phi = \frac{TL^2}{8GK_T} \quad \text{—(10.1)}$$

For weight twisting, the value of T is:

$$T = \frac{wB}{4} \text{ in lb/in.}$$

where w = lb/in. length. The increased stresses due to additional twisting can be calculated from a generalised form of eq. (7.1).

$$\sigma = \frac{(M_u + \phi M_v)v}{I_u} - \frac{(M_v - \phi M_u)u}{I_v} \quad \text{—(10.2)}$$

A more exact and comprehensive solution to this problem can be found in Ref.20.

11. Case IV

The loading for Case IV is shown in Fig. 3d. In this case the moment can be resolved into moments about the U and V axes (principal axes) and the theory or Cases I and II applies.

More generally, if the applied moment acts in any position between the X or Y and U axes, the component moments M_x, M_y, resolved in the U, V directions, will produce stresses σ_x and σ_y. The design is satisfactory if:

$$\frac{\sigma_x}{F_x} + \frac{\sigma_y}{F_y} < 1 \quad \text{—(11.1)}$$

where F_x and F_y are the maximum permissible stresses associated with the axis under consideration.

12. Conclusions

It has been shown that for laterally unrestrained angle beams the following relationships apply:

Case I:

The stress at any point in the section is:

$$\sigma = \frac{3M}{B^2t}\left((V+4U)\phi_t + V - 4U\right)$$

The maximum section stress is:

$$\sigma_{max} = \frac{3M(3-\phi_t)}{\sqrt{2}B^2t}$$

where the angle of twist $\phi_t = \phi + \phi_o$.

If the maximum stress is calculated without resolving the applied load into U and V components, the result may be up to 50% less than the actual maximum stress. Twisting may be ignored if:

$$\sigma_a < \left(38 - \frac{1}{60} \cdot \frac{L}{t}\right) \text{ ksi.}$$

An alternative method of beam design for Case I is to consider the critical buckling moment given by:

$$\left[\frac{M_o}{Et^3}\right]_{cr} = \frac{\pi^2}{15} \cdot \frac{B^2}{Lt}\left[\left(\left(\frac{B^2}{Lt}\right)^2 + 0{\cdot}785\right)^{\frac{1}{2}} - \frac{B}{Lt}\right]$$

and then use Ref.7, Rule 5.4.3, to convert this into a design stress.

The values of L/t for which the safe bending stress, F_b, may be taken as $0{\cdot}66F_y$, are shown in Section 6.

Case II:

The angle of twist ϕ has no direct effect in this case and the safe bending stress can be calculated using [7], Rule 5.4.3, where the critical buckling stress F_o, is obtained from:

$$F_o = \frac{\pi E}{2\sqrt{2 \cdot 6}} \cdot \frac{t}{L}$$

This may be ignored if $\frac{L}{t} < 200$ for F_y = 52 and $\frac{L}{t} < 300$ for F_y = 36 ksi, and a design stress of 0·66 F_y may be used.

Case III:

No secondary effects will occur and conventional beam formulas may be used.

Case IV:

The design is satisfactory if:

$$\frac{\sigma_x}{F_x} + \frac{\sigma_u}{F_u} < 1$$

where F_x and F_u are the appropriate maximum permissible stresses.

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13. Summary

The design criteria for angle beams can be summarised as follows:

Case	Use Simple Principal Axis Loading if:	Additional Effects if Column 2 Not Satisfied
I	(i) Stress Solution: $\sigma_e < 38 - \frac{1}{\epsilon 0} \cdot \frac{L}{t}$	$\sigma_{max} = \frac{2 \cdot 12M}{Bt}(\beta - \phi_1)$ (Fig.6)
	(ii) Critical Buckling Solution: See Table below.	Use $F_e \rightarrow F_{bc}$ conversion of Ref.7.
II	$\frac{L}{t} < 200$ ($F_y = 52$ ksi) $\frac{L}{t} < 300$ ($F_y = 36$ ksi)	Use Fig.10.
III	All Sections	—
Intermediate Loadings		$\frac{\sigma_x}{F_x} + \frac{\sigma_u}{F_u} < 1$

Critical Buckling Solution Case I:

Yield Stress F_y	B/t	Range for $F_{bc} = 0{\cdot}66 F_y$
52	6	$0 < L/t < 680$
	11	$0 < L/t < 570$
	16	$0 < L/t < 330$
36	6	$0 < L/t < 990$
	11	$0 < L/t < 850$
	16	$0 < L/t < 690$

For other B/t values, interpolate.

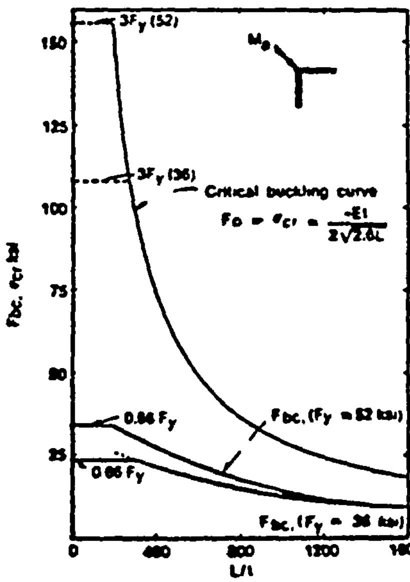


Fig.9. Critical buckling curve and curves for safe bending stress case II

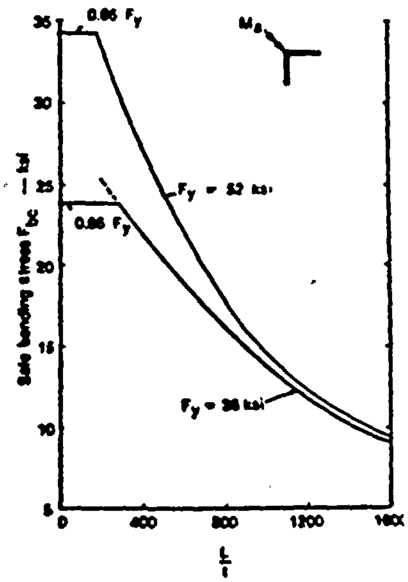


Fig.10 Graph for determination of F_{bc} for case II

The Behaviour of Laterally Unsupported Angles

By B. F. Thomas, J. M. Leigh, M.S., M.I.E.Aust. and M. G. Lay, B.C.E., M.Eng.Sc., Ph.D., F.I.E.Aust.*

Summary.—This paper reports the results of tests on laterally unsupported angles with equal and unequal legs subjected to a uniform moment over the entire laterally unsupported span. The moment is variously applied about the most common loading axes.

The results are compared with a theory proposed in earlier work. They show that the angle of twist, ϕ, normally reduces the maximum section stress produced. Since this twist has a significant influence on loading plane deflections, designers may safely use first order theory provided deflections do not exceed a typical limit of span/180.

The testing programme has also shown that practical angle sections are governed by stress and deflection criteria rather than by buckling.

LIST OF SYMBOLS†

- B Length of angle leg as defined in Fig. 1 $\left(\text{actual length} - \frac{t}{2}\right)$.
- C Centroid location.
- E Young's modulus.
- F_y Nominal yield stress.
- F Material yield stress.
- F_c Critical buckling stress.
- I Second moment of area about the axis perpendicular to the axis of load application.
- L Length of span.
- M Applied moment.
- M_U Component of the applied moment about the U-U axis.
- M_V Component of the applied moment about the V-V axis.
- M^{cr} Critical buckling moment.
- Q Length of angle leg as defined in Fig. 1 $\left(\text{actual length} - \frac{t}{2}\right)$.
- S Shear centre location.
- UU Major principal axis (U cross-section co-ordinate).
- VV Minor principal axis (V cross-section co-ordinate).
- W Polar axis.
- X Axis through the centroid parallel to the short angle leg.
- Y Axis through the centroid parallel to the long angle leg.
- Z_n Section modulus about the axis of load application.
- t Thickness of angle leg.
- x Deflection of the shear centre in the X-direction.
- y Deflection of the shear centre in the Y-direction.
- σ_n Nominal stress found from $\sigma_n = \frac{M}{Z_n}$.
- σ_{max} Actual maximum section stress.
- ϕ Angle of twist.
- $\hat{\phi}$ Maximum angle of twist.
- $'$ Differentiation with respect to w.
- θ Angle between the X and U axes (Fig. 1).

1.—INTRODUCTION

Most structural engineers are aware of the complex analysis involved in designing angle beams, if all the ramifications of their behaviour are to be taken into account. Common loading situations do not usually coincide with principal axes directions and such loading cases therefore produce biaxial bending deflections combined with axial twisting of the section. Structural design codes commonly (Refs. 1 and 2) give little or no design guidance for laterally unsupported angles and the 1968 SAA Steel Structures Code AS CA1 (Ref. 3) in Rule 5.4.3 stated:

> "The Standards Association of Australia is not prepared at this stage to make recommendations for angles which are not supported laterally."

This investigation, therefore, was aimed at developing a set of usable design formulae through a comprehensive testing programme.

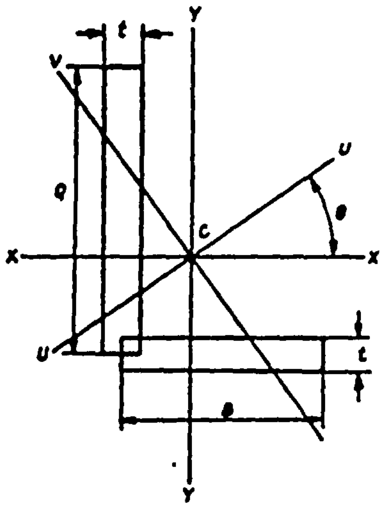
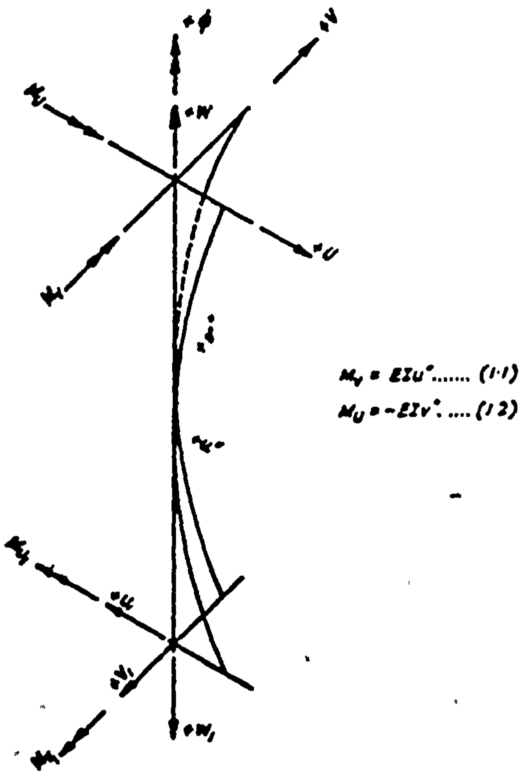


Fig. 1.—Simplified Angle Section Dimensions and Locations of Axes.



NOTE -
AXES ARE DRAWN WITH W OR W₁ AS THE OUTWARD DRAWN NORMAL FROM THE SURFACE UNDER CONSIDERATION.

Fig. 2.—Sign Convention.

*Paper No. 3197, submitted by the authors on June 7, 1972.

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†This notation is coupled with the sign convention given in Fig. 2.

During the initial stages of the programme a general theory describing the elastic behaviour of thin walled sections (Ref. 4) was used to determine the important parameters which governed the behaviour of equal angles (Ref. 5). The design rules proposed as a result of this study permitted a more enlightened approach to the experimental work.

The theory was subsequently developed into general design rules covering all angle sections (Ref. 6) and this permitted the compilation of the load tables for angle beams (Ref. 7). Since the testing programme proceeded in parallel with the theoretical analysis, it was possible to make progressive comparisons between the results of both sections of the work.

The loading rig, which is described in Section 4.1, was designed to apply a uniform moment to a beam laterally unrestrained between two torsional restraint points. This loading constituted the most critical buckling situation (Ref. 8) in accordance with the other buckling rules of CA1 (Refs. 9 and 10). Test specimens included both equal and unequal angles loaded about axes parallel to one leg.

Although the project was primarily concerned with the elastic behaviour of angle beams, a number of failure tests were also carried out to determine the ultimate load carrying capacity and failure modes of these sections.

2.—LOADING CASES

The loading cases considered (Fig. 3) represent the most common loading conditions for angle beams. These are:

2.1 Equal Angles:

Case I—Moment applied about an axis parallel to either the X-X or the Y-Y axis.

2.2 Unequal Angles:

Case I—Moment applied about an axis parallel to the Y-Y axis, that is, parallel to the long leg.

Case II—Moment applied about an axis parallel to the X-X axis, that is, parallel to the short leg.

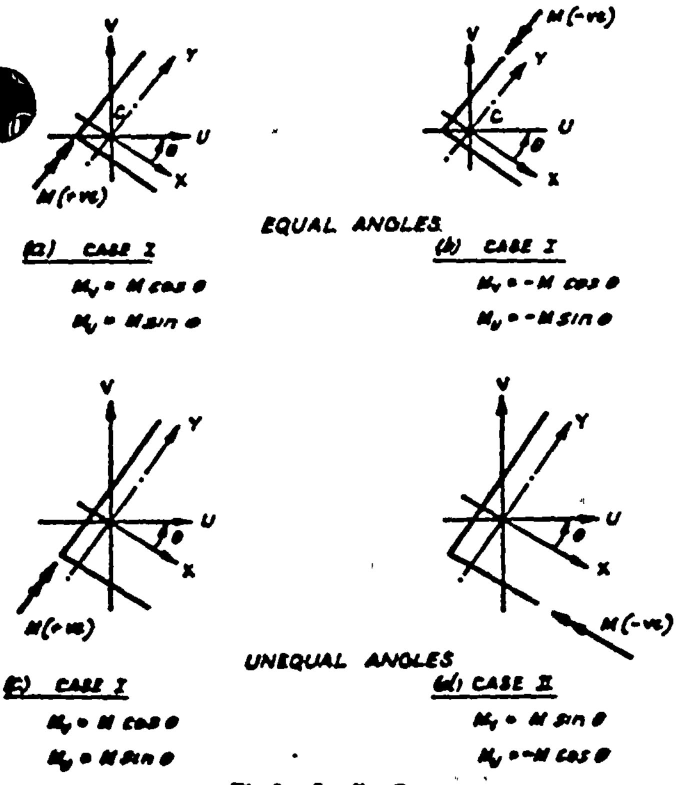

Fig. 3.—Loading Cases.

3.—TESTING PROGRAMME

The parameter which has the most pronounced influence on the lateral stability of angle sections is $\frac{B+Q}{2t}$. The effect of this parameter is demonstrated by the critical buckling analysis for equal angles (Ref. 5) which shows that for Case I loading the critical buckling moment is a function of B/t (analogous to $\frac{B+Q}{2t}$ for unequal angles) and L/t. If failure stresses are to remain unaffected by elastic buckling (Refs. 10 and 12) the value of L/t for a given $\frac{B}{t}$ ratio must be within the range given by Table I.

TABLE I

Elastic Buckling Effects of Various Angle Sections (36 ksi Steel)

Yield Stress	$\frac{B}{t}$	Range for No Elastic Buckling Effects
$F_Y = 36$ ksi	6	$0 < \frac{L}{t} < 990$
	11	$0 < \frac{L}{t} < 850$
	16	$0 < \frac{L}{t} < 690$

The table shows that sections with the practical upper limit B/t of 16 have less buckling resistance than sections with a lower value.

This criterion was used for selecting the sections tested. Both equal and unequal angles were tested and the values of B/t for each section are given in Table II, which also summarises the testing programme.

A single specimen was used for each series. This was made possible by testing the longer lengths first and keeping the stresses below yield, until a "destruction" test was required.

TABLE II

Testing Programme Summary

Test No.	Section Dimensions	$\frac{B+Q}{2t}$	$\frac{L}{t}$	Loading Case	Applied Moment Sense
EA2	3" × 3" × 0.187"	16	1600	Case I	+
EA3	3" × 3" × 0.187"		1600	Case I	−
EA4	3" × 3" × 0.187"		1200	Case I	+
EA5	3" × 3" × 0.187"		1200	Case I	−
EA6	3" × 3" × 0.187"		800	Case I	+
EA7	3" × 3" × 0.187"		800	Case I	−
EA8	3" × 3" × 0.187"		400	Case I	+
UE1	2.5" × 2" × 0.25"	16	1200	Case II	−
UE2	2.5" × 2" × 0.25"		1200	Case I	+
UE5	2.5" × 2" × 0.25"		1200	Case I	Follow-up test to failure after UE2. Always referred to as UE2 in text.
UE6	2.5" × 2" × 0.25"		400	Case I	+
UE7	2.5" × 2" × 0.25"		400	Case II	−
UE3	3.5" × 2.5" × 0.187"	9	1600	Case I	+
UE4	3.5" × 2.5" × 0.187"		1600	Case II	−
UE8	3.5" × 2.5" × 0.187"		400	Case II	−
UE9	3.5" × 2.5" × 0.187"		400	Case I	+

4.—EXPERIMENTAL APPARATUS

4.1 Loading Rig:

The purpose of the loading rig was to apply a uniform loading moment to an angle test specimen while applying only torsional restraint and vertical support at the ends, i.e. $\phi = \phi'' = 0$ at both ends. Spherical joints were used where necessary to ensure that the load application did not provide lateral restraint to the test piece. The loading beam thus rotated horizontally and vertically with the test specimen and only applied moments in a vertical plane parallel to the angle at the support. It was not in any way connected to the floor.

The rig comprised two independent, identical assemblies which could be positioned on the loading floor at any required distance to accommodate changes in the test length. Each assembly consisted of a stand surmounted by a roller bearing, for vertical support, and a frame which housed adjustable horizontal restraint supports which had additional function of providing torsional restraint to the ends of a, mens being tested.

The addition of weights to the loading beam produced a force in the vertical link which was connected to the end of the test piece. The loading beam was supported by a needle roller bearing in the vertical link which ensured that the vertical link would always locate normal to the loading beam, whatever its position. The assembled rig is shown pictorially in Fig. 5. Detailed drawings of the rig are given in Ref. 16.

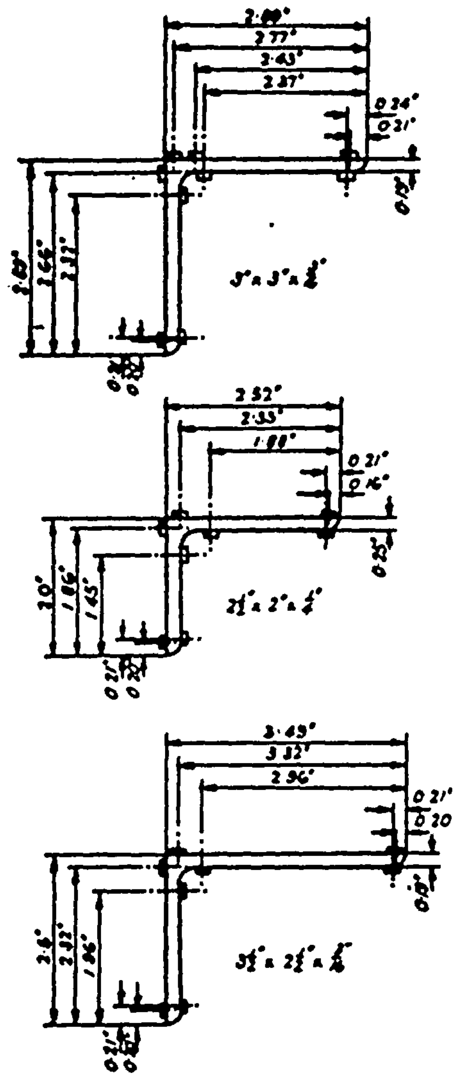

Fig. 4.—Strain Gauge Locations.

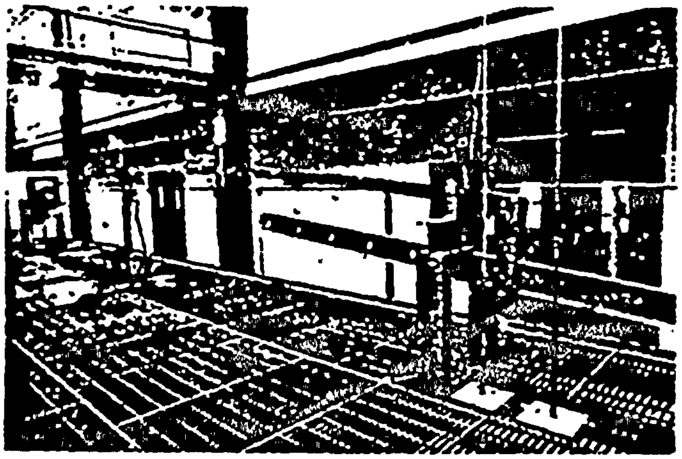

Fig. 5.—Assembled Rig.

4.2 Measuring Instruments:

[illegible]tions were measured by the optical lever principle using a [illegible] mirror system. This system also avoided the introduction of specimen restraints. The maximum value of the angle of twist (ϕ_{max}) was also measured at mid-span using a bevel protractor. Vertical loading plane deflections were measured at mid-span and quarter points by means of a theodolite and graduated scale. Horizontal deflections were measured with a steel tape.

A number of strain gauges were placed on the section at its mid-span position (Fig. 4). The gauges were read using an automatic data logging system (Ref. 11) which converted the readings directly to stress values. By following the testing programme outlined earlier, it was possible to use the one set of strain gauges for a complete series of tests.

5.—TENSILE TESTS

Tension tests were carried out on coupons obtained from each angle section tested. The tests were performed on an Instron Universal testing machine with a crosshead speed of 0.079 inch/min which corresponds to a strain rate of 0.005 min^{-1}. Average values of material yield stress and tensile stress were found and these indicate that the material conformed to the requirements of AS A149 (Ref. 14).

6.—RESULTS

6.1 Stress Levels:

Stress values were measured at mid-span using strain gauges located as shown in Fig. 4. The nominal section stress has been used as a yard-stick for comparison with the maximum section stress σ_{max} which occurred at mid-span at the tip of the vertical leg. The maximum (flange tip) stress was determined by linear extrapolation from the stress distribution across the section. The nominal stress can be found from

$$\sigma_s = \frac{M}{Z_s}$$

where M is the applied moment and Z_s is the section modulus about the axis of load application.

Figs. 6 and 7 show curves describing the relationship between σ_s and σ_{max} for

(*a*) Tests results.

(*b*) Theory with zero angle of twist ($\phi = 0$).

(*c*) Theory with $\phi \neq 0$.

(*d*) Theory with measured test values of ϕ.

Fig. 6 includes the results of tests on the 3" × 3" × 3/16" section with an L/t ratio of 1600. This section was subjected to applied moments in a positive (EA2) and negative (EA3) sense about an axis parallel to one leg.

For the same applied load the stress levels recorded were up to 15% greater for a loading sense which produced tensile stresses in the horizontal leg (EA2). This difference was only 4% at a stress level of $0.66F_y$ where F_y is the nominal yield stress (36 ksi). Hence within the design range the difference in behaviour for reversed loading sense is negligible.

The difference results from the fact that the constitutive equations (Refs. 5 and 6) were based on small deflections theory whereas in fact for large spans, mid-span loading plane deflections were in the order of 12 inches. Since deflections of this magnitude are not experienced in practice, and since small deflection theory conservatively predicts angle behaviour where deflections exceed a limit of span/180, a more refined analysis is not justified.

The unequal angles tested (Fig. 7) behaved in a similar manner for both Cases I and II. For all sections, tests on shorter lengths (EA6, EA7, EA8, UE6, UE7, UE8, UE9) indicated close agreement with the theory.

Fig. 8 shows a typical stress distribution of the sections tested for points which lie closest to stress levels of $0.66F_y$ and F_y.

Fig. 7 (tests UE2, UE4) show the two buckling failures which occurred before the full material yield stress was reached. These failures are discussed further in Section 6.3. For all tests the theoretical and experimental stress values were in good agreement up to a maximum section stress of $0.66F_y$. Beyond this level, for beams with large values of L/t the predictions of first order theory (i.e. assuming $\phi = 0$) were conservative. Therefore, the simple no-twist relationship (Refs. 5 and 6)

$$\sigma_{max} = 1.25\ \sigma_s$$

should give conservative results at all stress levels.

6.2 Angles of Twist ϕ_{max}:

Angles which are loaded about axes other than principal axes will twist axially to align the weak principal axis with the axis of load application. This twist in the test series was measured at mid-span and recorded values are compared with the theoretical predictions in Fig. 9 for all sections tested. Points A and B coincide with nearest attainment of $0.66F_y$ and F_y level stresses. In each case the direction of load application opposed the dead weight load, hence initial twisting due to the dead weight of the beam was reduced as the beam was loaded. The zero twist position, therefore, corresponded approximately with the condition when the applied load effect equalled the dead weight load. This load has been taken as the starting point for plotting the theoretically predicted values of angle of twist (ϕ_{max}) (Ref. 6).

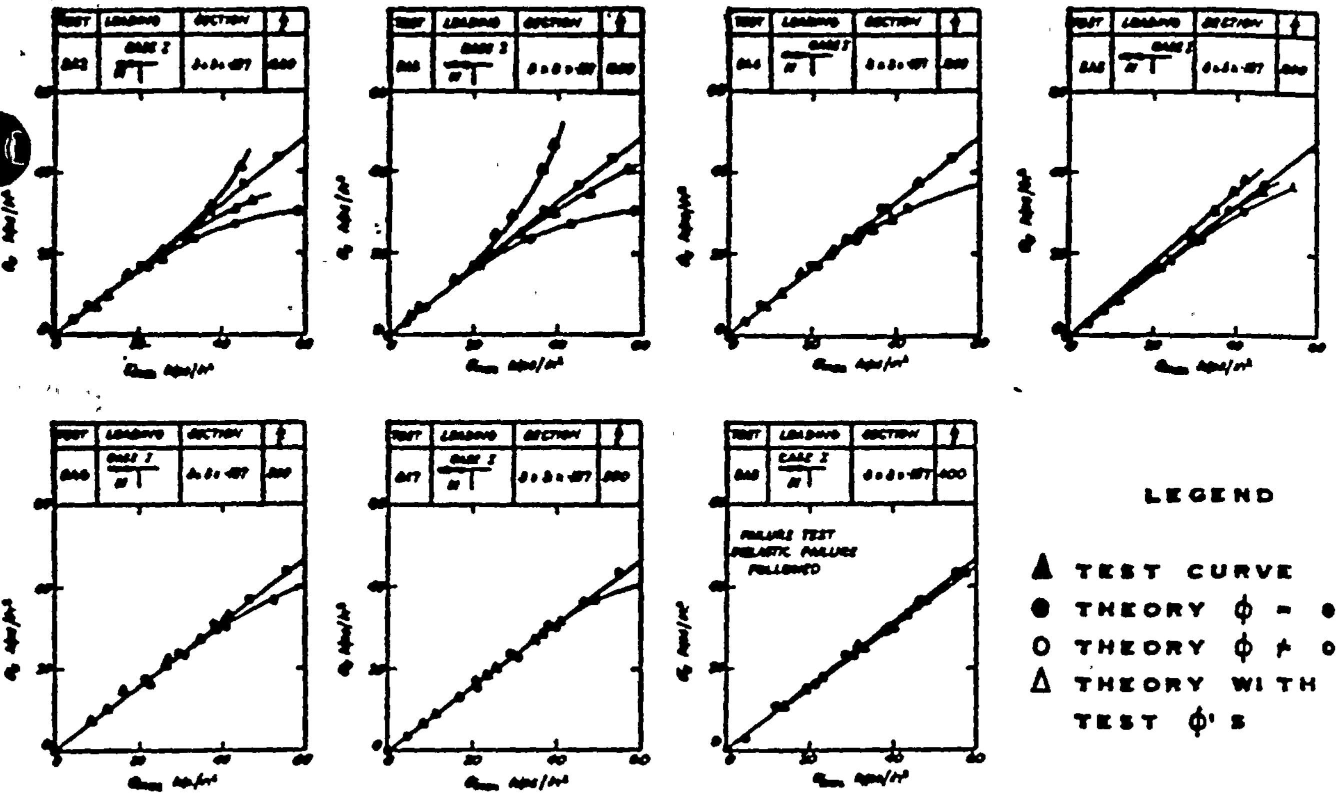

Fig. 6.—Relationship between Nominal Applied Stress σ_a and Maximum Stress σ_{max} for Tests on Equal Angle Sections.

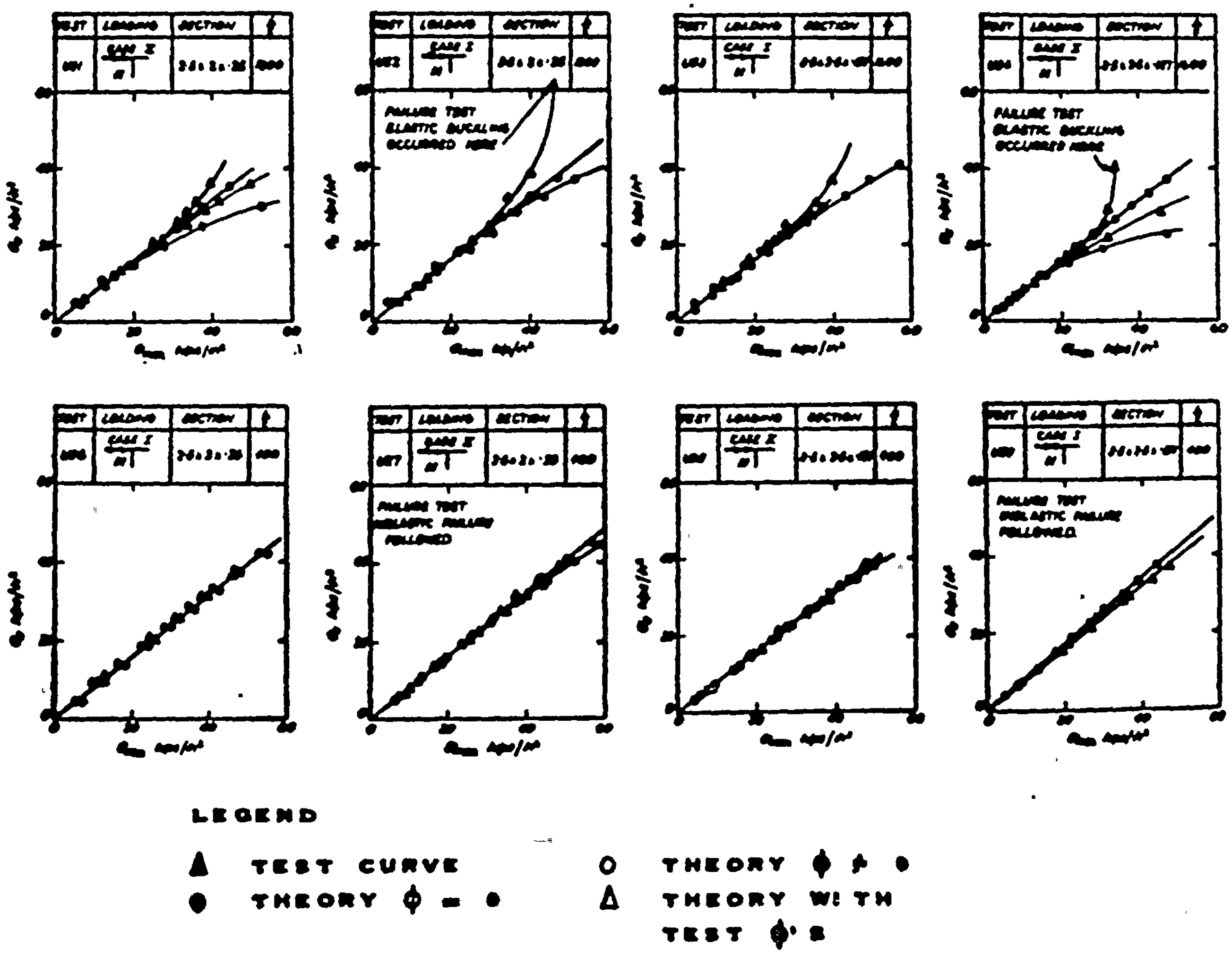

Fig. 7.—Relationship between Nominal Applied Stress σ_a and Maximum Stress σ_{max} for Tests on Unequal Angle Sections.

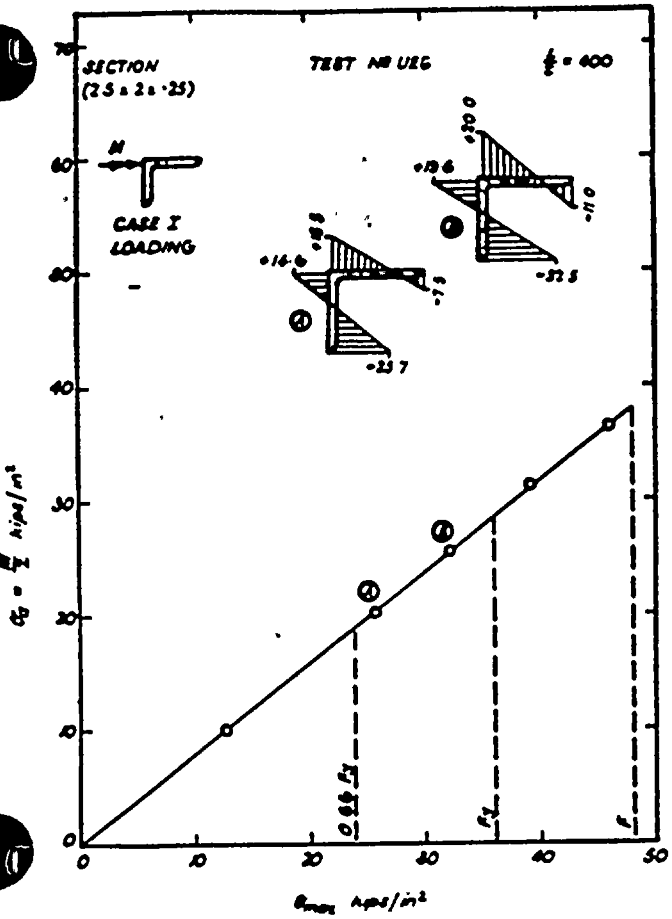

Fig. 8.—Relationship between Nominal Applied Stress σ_a and Maximum Section Stress σ_{max} for Test No. UE6.

The curves of Fig. 9 (tests UE1, UE2, UE3, UE4, EA2, EA3) are consistent with the stress curves of Fig. 7 (same tests) in that they show graphically the torsional stiffening which takes place at large values of applied load and correspondingly large angles of twist. This effect is less significant for shorter lengths, consequently theoretical and experimental results are in closer agreement (Fig. 9—tests UE6, UE7, UE8, UE9, EA6, EA7). Sections subjected to Case II loading must twist through an angle of $(90-\theta)°$ to align with the minimum principal axis where θ is the angle between the X and U axes, whilst Case I loading requires only a twist through $\theta°$. This is reflected in the large angles of twist recorded for Case II loadings (Fig. 9—tests UE1, UE4, UE7).

6.3 Tests to Failure:

Five tests to failure were performed on both equal and unequal angles for a variety of L/t values. These tests are summarised in Table III which also shows the value of the applied moment at which failure occurred and failure modes. In all cases failure occurred at the end of supports.

TABLE III

Summary of Failure Test Results

Test No.	Section	$\frac{L}{t}$	Loading Case	Failure Moment (kips-in)	Failure Mode
EA8	3" × 3" × 0.187"	400	I	20.5	Torsional Buckling
UE4	3.5" × 2.5" × 0.187"	1600	II	22.0	Torsional Buckling
UE9	3.5" × 2.5" × 0.187"	400	I	19.2	Torsional Buckling
UE2	2.5" × 2" × 0.25"	1200	I	15.7	Flexural Buckling
UE7	2.5" × 2" × 0.25"	400	II	18.9	Torsional Buckling

Fig. 10 shows the relationship between the critical buckling curve and design curve for each of the unequal angle failure tests. The critical buckling curve is obtained from the analysis given in Ref. 16 and the design curve can be found using Rule 5.4.3 of CA1 (Ref. 3). The horizontal line, denoted $3F_y$, represents the value of $\frac{M^{cr}}{Et^3}$ to produce a stress of $3F_y$ where F_y is the nominal yield stress (36 ksi). It is normally assumed (Refs. 10 and 12) that failure stresses will be unaffected by elastic buckling if the buckling stress is at least three times the material yield stress. All sections tested behaved in accordance with this assumption. The elastic buckling failure (test UE4) plots to the right of the intersection of the critical buckling curve and the $3F_y$ line, i.e. $F_e < 3F_y$. The actual failure moment was somewhat greater than that predicted by the critical buckling analysis, due probably to higher order effects for this very "slender" case.

Elastic buckling also occurred in test UE2 at a critical moment corresponding to the theoretically predicted value for this less "slender" case. The sections of tests UE9 and UE7 obtained full yield stress before failure. Good agreement with the $3F_y$ assumption is apparent from Fig. 9 (tests UE7, UE9) which show that the failure points plot within the region expected for inelastic buckling. The results for the equal angle test (EA8) have not been graphed since failure occurred inelastically in agreement with similar failures on the unequal angles tested.

6.4 Deflections:

Biaxial bending deflections resulting from non-principal axis loading were measured as shear centre displacements in the loading plane and normal to the loading plane. Fig. 11 includes the recorded results, for the section 3" × 3" × 0.187" loaded as shown in Fig. 3(*a*), compared with the predicted curves for both first and second order theory ($\phi = 0, \phi \neq 0$) for a variety of L/t ratios.

This limit is recommended by CA1 for structural applications where angles could be used. Beyond this level, for $L/t > 1000$, second order theory conservatively predicted the loading plane deflections whereas first order theory under-estimated this deflection by up to 20%.

Similar results were obtained for the reverse loading case and for unequal angles with Case II loadings (Fig. 11—tests UE1, UE4, UE7, UE8). Unequal angles with Case I loading and $L/t > 1000$ deflected less than predicted by either first or second order theory for load values which caused deflections in excess of $L/180$ (Fig. 11—tests UE2, UE3).

The tests indicate that provided deflections are limited to $L/180$, first order theory will give an accurate estimate of the loading plane deflection. Consequently, if it is desired to use sections which develop the full bending stress of $\sigma_{max} = 0.66F_y$, then the use of the design formula (Refs. 5 and 6)

$$\frac{L}{B+Q} < \frac{600}{F_y}$$

will permit the attainment of full stresses and avoid lateral buckling problems. Above this limit the use of second order theory will ensure that results are conservative for all cases.

CONCLUSIONS

A total of 15 tests were performed on laterally unsupported angles with equal and unequal leg lengths for a variety of L/t values. The loading condition was a uniform moment over the entire laterally unsupported span and this load was applied about an axis parallel to an angle leg. Uniform moment is the most critical design situation. Adjustments for other loadings would probably follow standard procedures (Refs. 4 and 8), however this was not studied in the present work. The solutions given will always be conservative.

From the results of these tests it was concluded that for the loading conditions stated above:

1. The angle of twist (ϕ) causes a reduction in the maximum section stress produced. Therefore, first order theory gives a conservative estimate of this stress, i.e.
$$\sigma_{max} = 1.25\,\sigma_a$$
2. The angle of twist (ϕ) has a significant influence on the maximum loading plane deflection beyond a deflection of $L/180$ and second order theory gives a conservative estimate of deflection above this level.
3. The five tests to failure indicated that laterally unsupported angles will be unaffected by elastic buckling provided that the critical buckling stress is at least three times the material yield stress.
4. Practical angle sections are seen to be governed by stress or deflection limitations rather than by buckling.

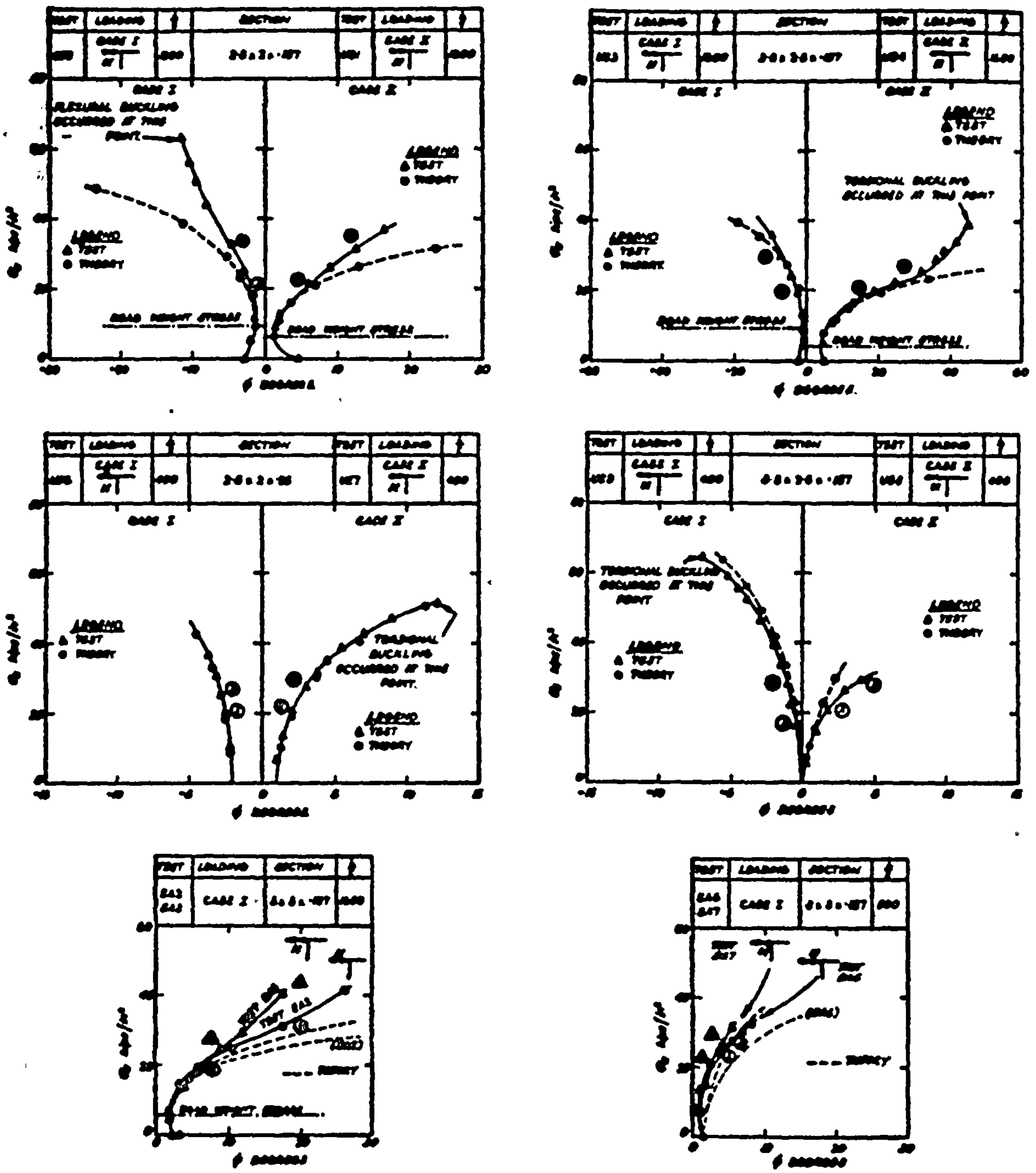

Fig. 9.—Relationship between Nominal Applied Stress σ_n and Maximum Angle of Twist ϕ.

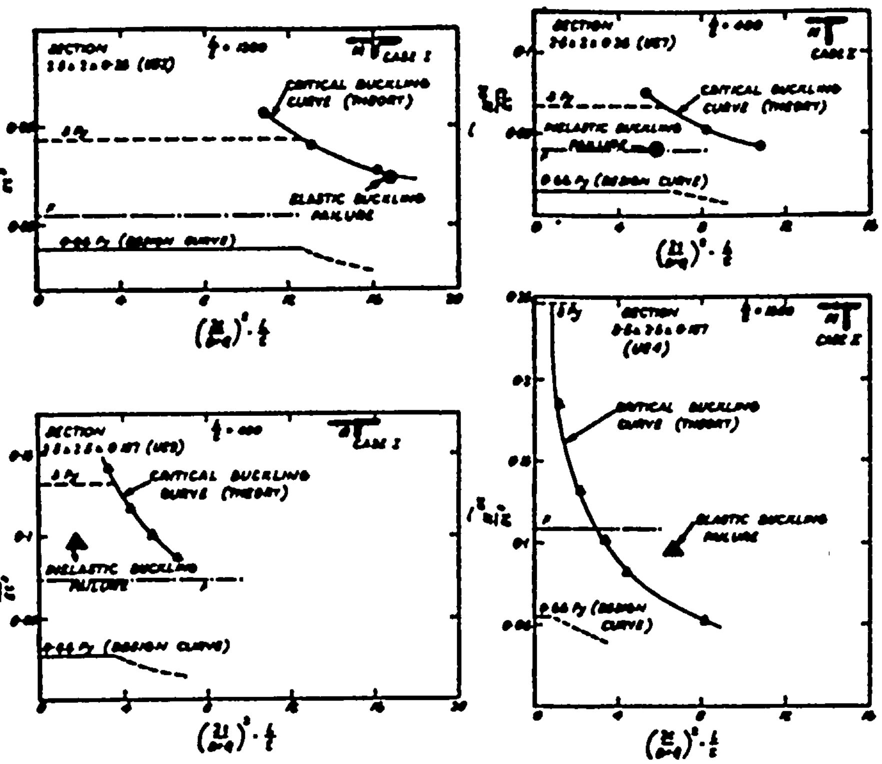

Fig. 10.—Test to Failure Results Compared with Critical Buckling Curves and Design Curves.

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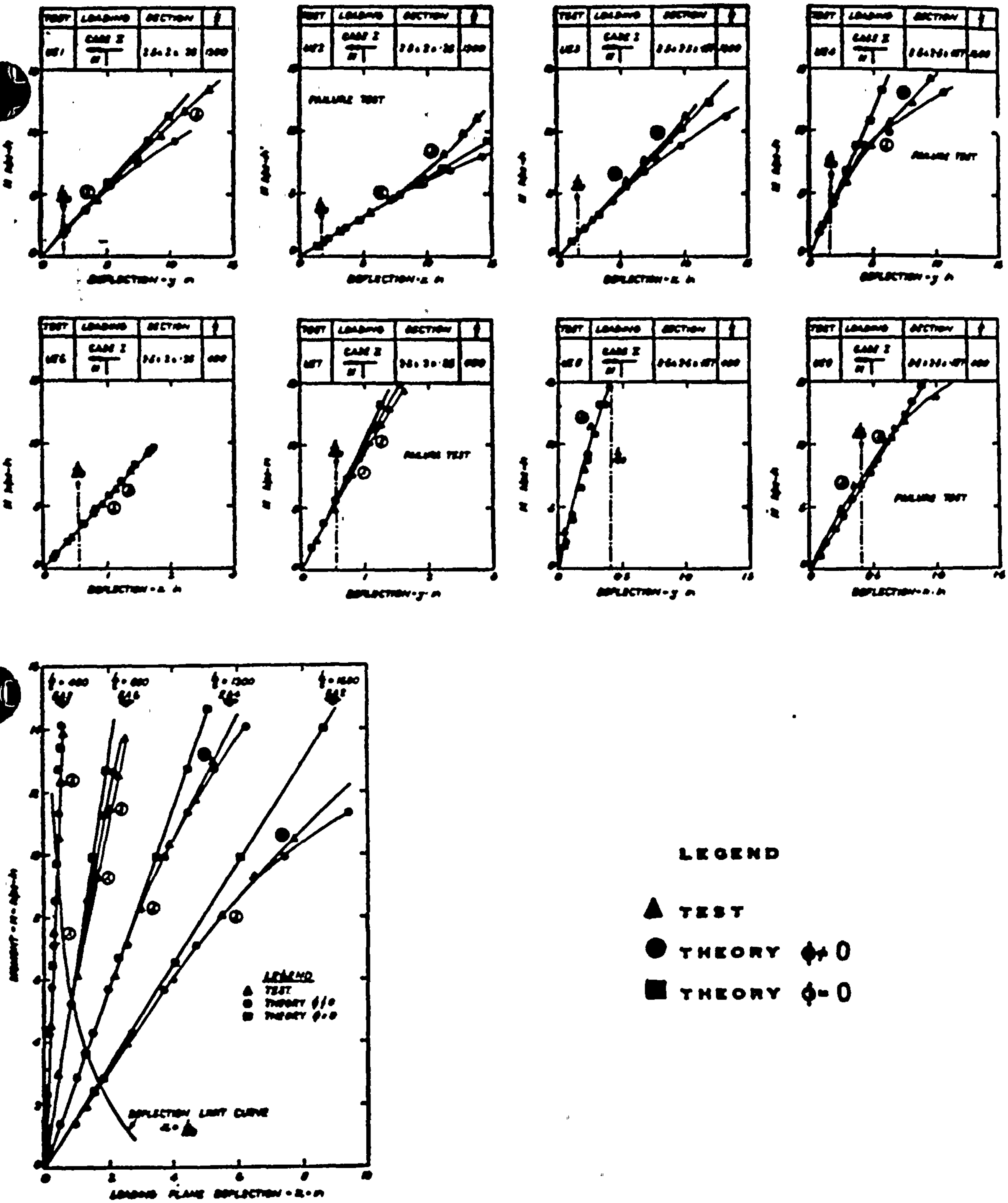
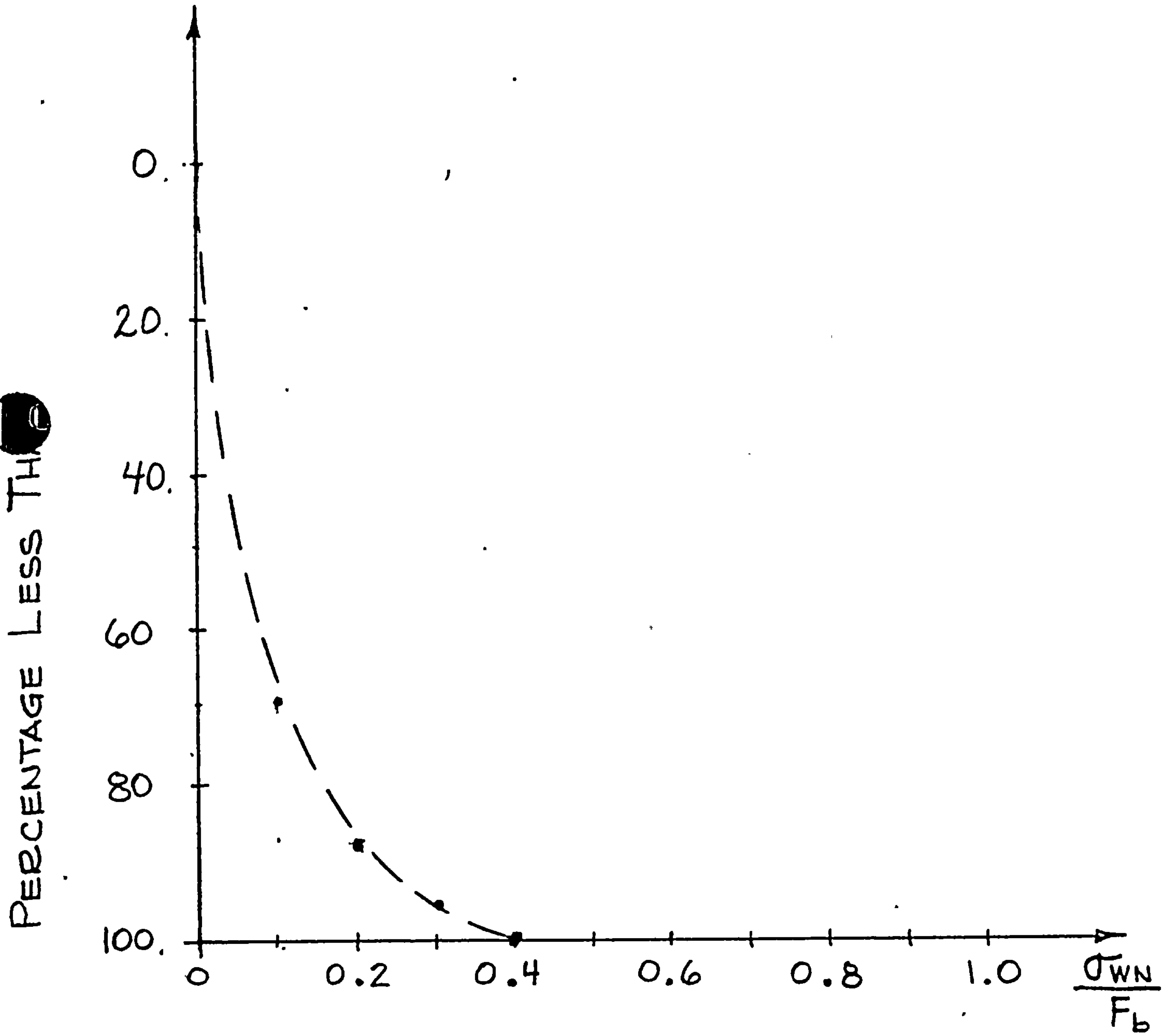


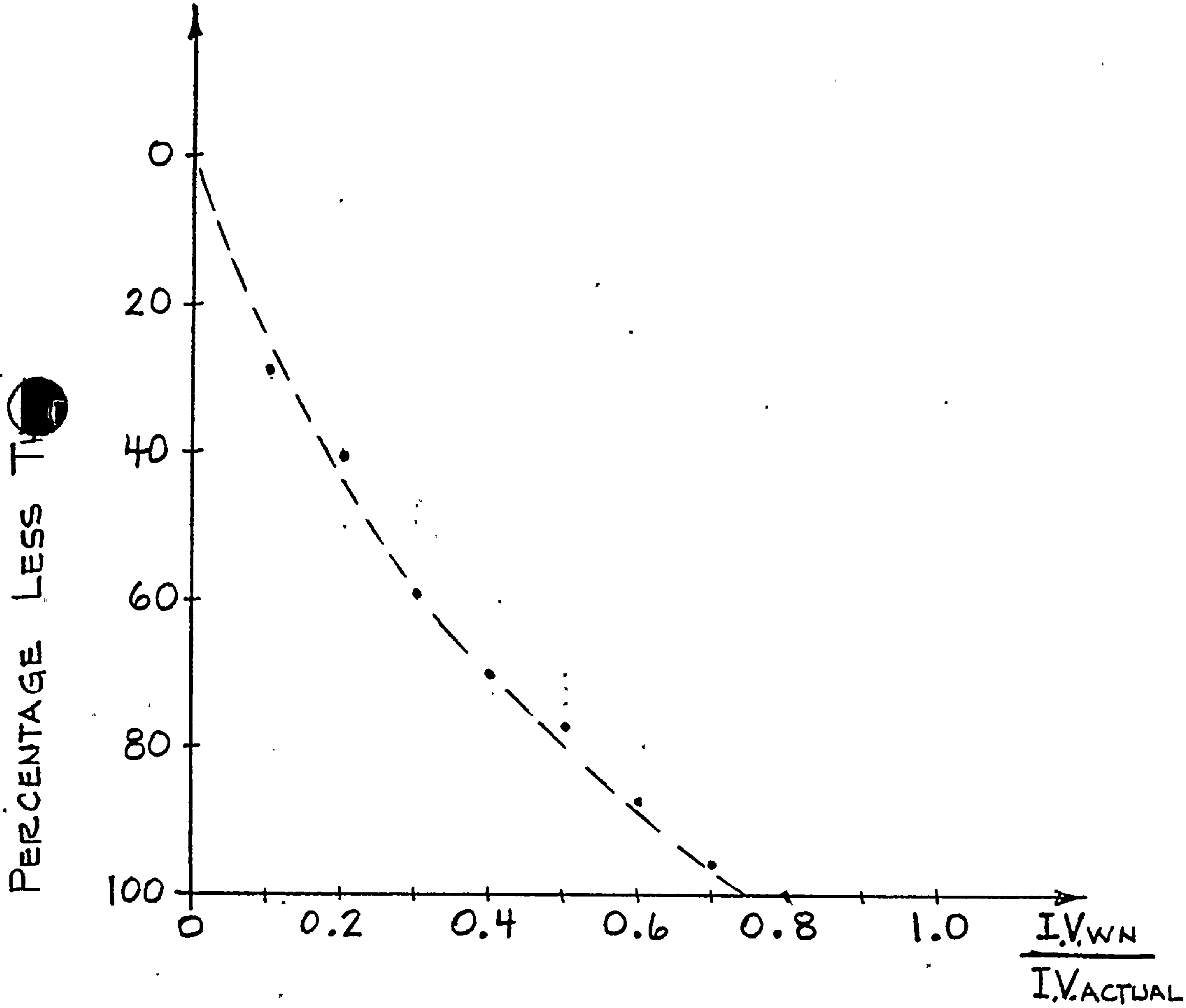
Fig. 11.—Relationship between Applied Moment M and Maximum Loading Plane Deflections.

ATTACHMENT 7-2



PERCENT VS. WARPING NORMAL-TO-BENDING ALLOWAE

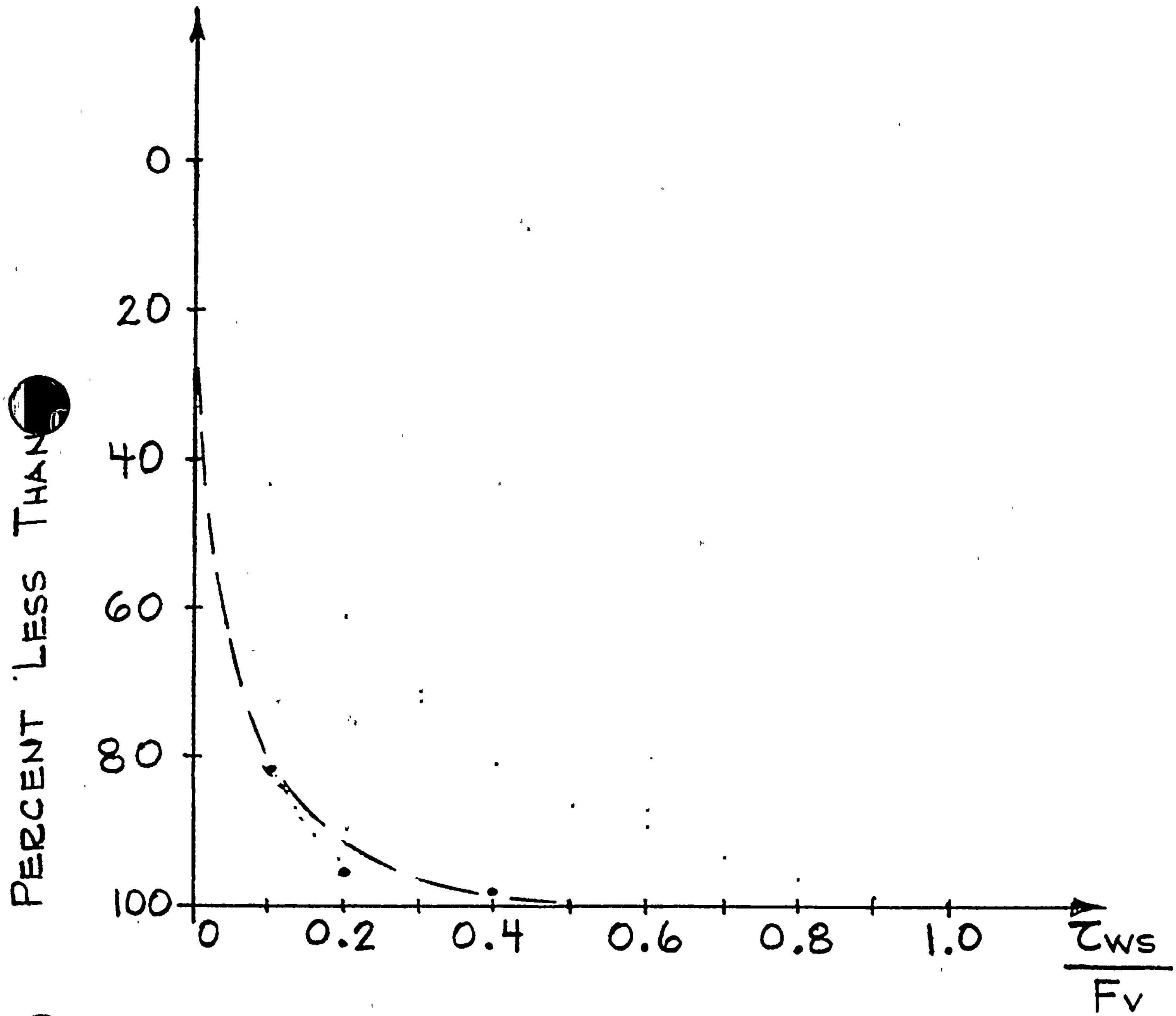
FIGURE 1



ERCENTAGE VS. WARPING NORMAL INTERACTION RATIO

FIGURE 2

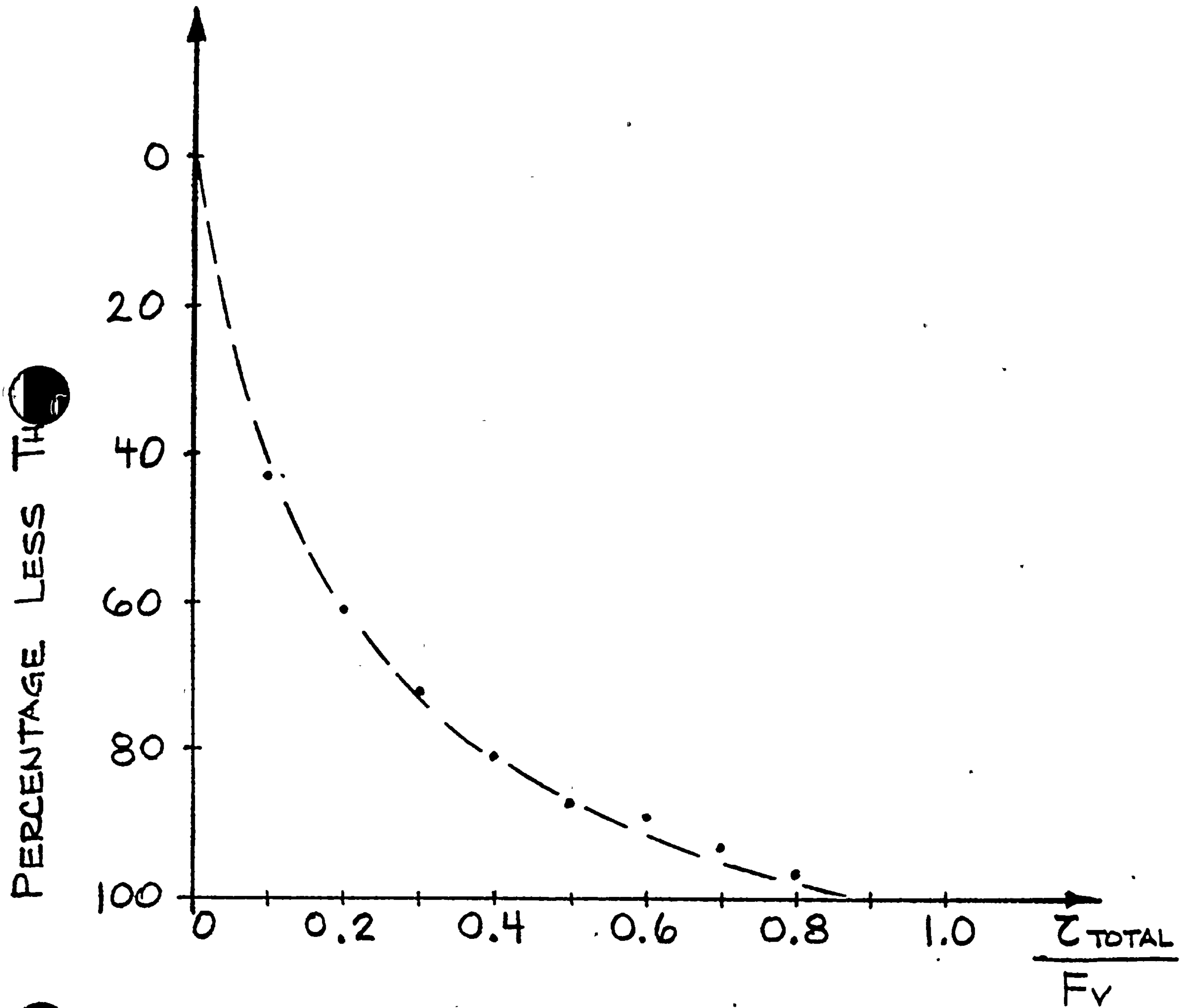
ATTACHMENT 7-4



PERCENTAGE VS WARPING SHEAR-TO-ALLOWABLE RATIO

FIGURE 3

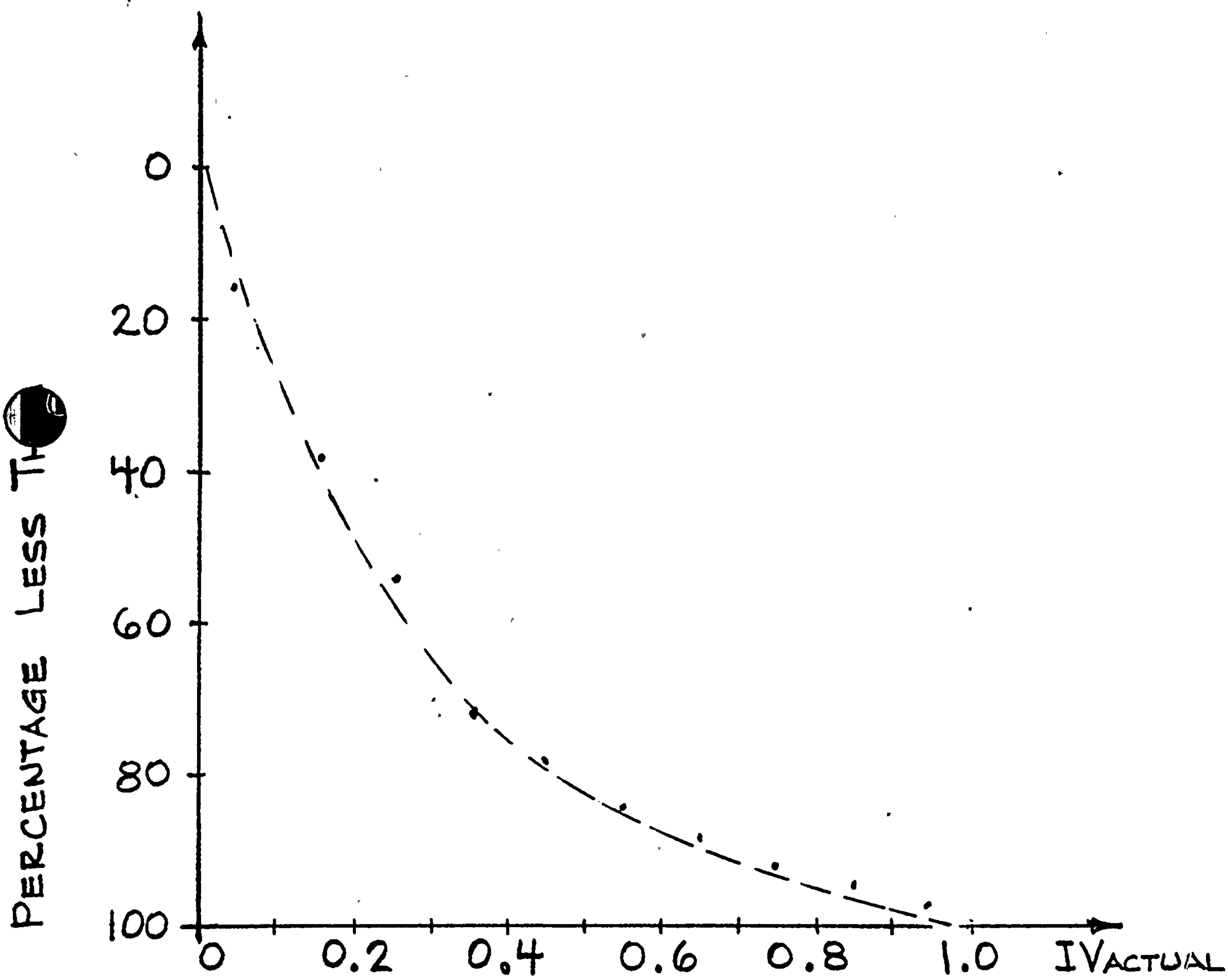
ATTACHMENT 7-5



PERCENTAGE VS. TOTAL SHEAR-TO-ALLOWABLE STRESS RATIO

FIGURE 4

ATTACHMENT 7-6



PERCENTAGE VS. ACTUAL INTERACTION VALUE

FIGURE 5

COPY

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June 8, 1984

PGandE Letter No.: DCL-84-219

Mr. Darrell G. Eisenhut, Director
Division of Licensing
Office of Nuclear Reactor Regulation
U. S. Nuclear Regulatory Commission
Washington, D.C. 20555

Re: Docket No. 50-275, OL-DPR-76
Diablo Canyon Unit 1
License Condition 2.C.(11) - Final Report

Dear Mr. Eisenhut:

In accordance with License Condition 2.C.(11) of Facility Operating License DPR-76, PGandE submitted a final report for reviews performed on piping and pipe supports on June 1, 1984, in PGandE Letter DCL-84-203. Information on Items 2, 3, and 6 was included in that submittal. On June 7, 1984, PGandE submitted final information for Items 4 and 5 in PGandE Letter DCL-84-214.

The enclosure to this letter provides final information on Item 7 of the License Condition. Final information for Item 1 is scheduled to be submitted on June 11, 1984.

As noted in this final report, lines at the side of each page indicate revisions to previously submitted information, and additional results of PGandE's review are included in the attachments to the enclosure. A revised table of contents is also included to reflect this additional information.

Kindly acknowledge receipt of this material on the enclosed copy of this letter and return it in the enclosed addressed envelope.

Sincerely,

W. A. Raymond

for J. O. Schuyler

Enclosure

cc: J. B. Martin
H. E. Schierling
Service List