PACIFIC GAS & ELECTRIC COMPANY
MECHANICAL & NUCLEAR ENGINEERING
DIABLO CANYON UNIT 1
WALKDOWN OF PIPING DURING POWER ASCENSION

Procedure P-38
Revision 1

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Concurred By:	J Mannina Project Construction Supervisor	4/27/84 Date

1.0 Purpose

The purpose of this procedure is to verify that the following points are complied with during the initial power ascension:

a) Those safety-related piping systems subject to significant thermal expansion and/or thermal anchor motion during power ascension respond in accordance with calculated deflections.

b) Piping systems meet the acceptance criteria described in section 4.0.

c) Adequate piping clearances are maintained.

2.0 Scope

2.1 Hot deflection measurement will be conducted only for that piping identified in Section 2.2 and on the pipe stress analysis isometric drawings listed in Appendix A. All other piping outside containment, subject to ALARA considerations, will be subject to a visual walkdown.

2.2 The piping systems within the scope of the walkdown that will have their thermal defections recorded are:

2.2.1 Main Feedwater from the G-Line anchor to the Steam Generator

2.2.2 Main Steam

2.2.2.1 Main Steam from the G-Line to the Steam Generator

2.2.2.2 Safety-related portions of Steam Generator Blowdown

2.3 For convenience in performing the walkdown of piping described in Section 2.2 the analyses are grouped into the walkdown packages listed in Appendix B.

3.0 Responsibility

3.1 Engineering is responsible for the following activities:

3.1.1 Providing the pipe stress isometric and sets of thermal displacements corresponding to the latest revision of the analysis.

3.1.2 Assisting General Construction as required during the walkdown.

3.1.3 Resolving any problems between calculated movements and actual movements.

3.1.4 Final review and acceptance of each Power Ascension Walkdown Package and all associated Power Ascension Walkdown Problem Reports.

3.2 The Stress Walkdown Team Leader is responsible for interfacing with Startup Team, meeting the requirements of this procedure, assuring that all measurements are taken, and that problems, if any, are resolved prior to leaving a given power level.

4.0 Acceptance Criteria

4.1 Piping systems as described in Section 2.1 will be deemed acceptable for thermal expansion if the following criteria are satisfied.

4.1.1 The piping system and related or attached components should not be restrained against thermal expansion during the test except by design intent.

4.1.2 If the piping system is supported by spring hangers, these should not become extended or compressed beyond their working range during the thermal expansion of the piping.

4.1.3 If the piping system is restrained by snubbers, these will not become extended or compressed to the limits of their total travel, or bound-up due to the swing angle during the thermal expansion of the piping.

4.2 For piping systems where actual deflections are to be recorded, the measured deflections will be reviewed against the calculated deflections and should fall within the shaded acceptable range of the graph in Appendix C.

4.2.1 It is not the intent of this procedure, nor is it required, to verify during this test the movements predicted by the analysis for every point in the system. Instead the above objectives will be accomplished by monitoring pre-selected, strategically located snubbers, spring supports and rupture restraints and/or by visual observation of piping clearances. These pre-selected points are as noted on the pipe stress analysis isometrics (see Appendix A).

4.3 All exceptions to the acceptance criteria shall be documented and reconciled by Engineering (Refer to section 5.3.1.4).

5.0 Procedure

5.1 Prior to the beginning of Power Ascension for piping whose deflections will be measured, the walkdown packages will be assembled and the following efforts completed:

5.1.1 Data points will be chosen that can be used to monitor the overall behavior of the piping system.

5.1.2 Directions of displacement to be measured at each data point will be determined and marked on the stress analysis isometric.

5.1.3 Expected displacement at the data points will be calculated in the localized directions to be measured. These displacements will be recorded (to the nearest 0.1") on Attachment 3 under "Calculated Deflection".

5.1.4 The "Cold Position" will be measured (to the nearest 0.1"), recorded on Attachment 3, and signed off on Attachment 1.

5.2 Prior to walkdown activities at a given power level, the system's temperature will be held constant for a minimum of 60 minutes. This will allow thermal transients to decay, ensuring that the piping system is at steady state temperature.

5.3 The following work will be completed at each of the power levels (30%, 50%, 75%, and 100%). In addition, a walkdown of the main steam piping outside containment will be performed before ascension above 5% power with NRC participation. A modified version of the cover sheet will be used to document this walkdown.

5.3.1 For piping whose deflections will be compared to the calculated deflections the following activities will be performed.

5.3.1.1 The "Hot Positions" will be measured (to the nearest 0.1"), recorded on Attachment 3, and signed off on Attachment 1.

5.3.1.2 In the course of recording data for Section 5.3.1.1, the entire piping system shall be observed for compliance with the criteria listed in Section 4.1.

5.3.1.3 The "Measured Deflection" will be determined and compared with the "Calculated Deflections" using the acceptance chart in Appendix C.

5.3.1.4 A Power Ascension Walkdown Problem Report (Attachment 2) will be used to document any cases where measured deflections do not satisfy the acceptance criteria and/or where interferences occur. The problem resolution section will be completed by a Walkdown Team Member.

5.3.2 For piping that will be subject to only a visual inspection, the plant will be walked down area by area (i.e., 85' GE-GW). Certain areas, based on ALARA considerations, need not be walked down.

5.3.3 The Walkdown Team Leader will notify the G.C. Startup Team of the completion of walkdown activities at each power level.

5.4 At the completion of Walkdown activities the Walkdown Team Leader will indicate Final Engineering Acceptance by signing the Power Ascension Walkdown Cover Sheet, as well as each associated Power Ascension Walkdown Problem Report in the designated space. Upon acceptance the Walkdown Team Leader shall transfer completed Walkdown Packages to Project Engineering File 146.155.

6.0 Walkdown Package

The walkdown package shall be assembled and completed as follows:

6.1 The Power Ascension Walkdown Cover Sheet, Attachment 1, will be completed to indicate the piping system description and analysis numbers (see Appendix B), the completion of the walkdown for each power level, and final acceptance by Engineering.

6.2 The stress analysis isometrics will be included.

6.3 The Power Ascension Piping Deflection Sheet(s) (Attachment 3) will be completed. The Calculated Deflections and Cold Positions will be entered prior to Power Ascension (Section 5.1). Hot Positions will be entered during heatup (Section 5.3), and Measured Deflections will be calculated from the cold and hot position data.

6.4 One copy of each Power Ascension Walkdown Problem Report will remain as part of the package.

6.5 Applicable computer runs and hand calculations made prior to and during Power Ascension will be included in the package. This information will be listed in calculation MP-1065.

7.0 Documentation

7.1 All Walkdown Packages will be filed under file number 146.155.

7.2 Because the calculations performed to generate the anticipated thermal movements are not "design" calculations, an approving signature is not required.

8.0 Equipment

The engineering personnel performing this walkdown shall be provided with or have access (as required) to the following:

a) scales
b) binoculars
c) safety belts
d) flashlight
e) required forms
f) gloves
g) pyrometers

9.0 References

9.1 Startup Test Procedure No. 40.0

9.2 Operating Procedure No. L-1

10.0 Appendices

10.1 Appendix A - List of Pipe Stress Analysis Isometrics to be included in the Power Ascension Walkdown

10.2 Appendix B - List of Walkdown Packages

10.3 Appendix C - Piping Displacement Acceptance Chart

11.0 Attachments

11.1 Attachment 1 - Power Ascension Walkdown Cover Sheet

11.2 Attachment 2 - Power Ascension Walkdown Problem Report

11.3 Attachment 3 - Power Ascension Piping Deflections

List of Pipe Stress Analysis Isometrics to be Included in the Heatup Walkdown

	Iso. No.	Title
1.	1-100	Steam Generator 4 Blowdown Inside Containment
2.	1-101	Steam Generator 1 Blowdown Inside Containment
3.	1-102	Steam Generator 1 Blowdown Inside Containment
4.	1-103	Steam Generator 2 Blowdown Inside Containment
5.	1-104	Steam Generator 4 Blowdown Inside Containment
6.	1-105	Steam Generator 1 Blowdown Inside Containment
7.	1-106	Steam Generator 1 Outlet Inside Containment
8.	1-107	Steam Generator 2 Blowdown Inside Containment
9.	1-110	Steam Generator 3 Blowdown Inside Containment
10.	1-111	Steam Generator 1 Blowdown Outside Containment
11.	1-112	Steam Generator 2 Blowdown Outside Containment
12.	1-113	Steam Generator 3 Blowdown Outside Containment
13.	1-114	Steam Generator 4 Blowdown Outside Containment
14.	1-119	Steam Generator 2 Outlet Inside Containment
15.	1-120	Steam Generator 3 Outlet Inside Containment
16.	1-121	Steam Generator 4 Outlet Inside Containment
17.	2-100	Steam Generator 2 Feedwater Supply Inside Containment
18.	2-101	Steam Generator 3 Feedwater Supply Inside Containment
19.	2-102	Steam Generator 1 Feedwater Supply Inside Containment
20.	2-103	Steam Generator 4 Feedwater Supply Inside Containment
21.	MS-1	Steam Generator 1 Outlet Outside Containment
22.	MS-2	Steam Generator 2 Outlet Outside Containment
23.	MS-3	Steam Generator 3 Outlet Outside Containment
24.	MS-4	Steam Generator 4 Outlet Outside Containment
25.	FW1-4	Steam Generator 1-4 Feedwater Outside Containment

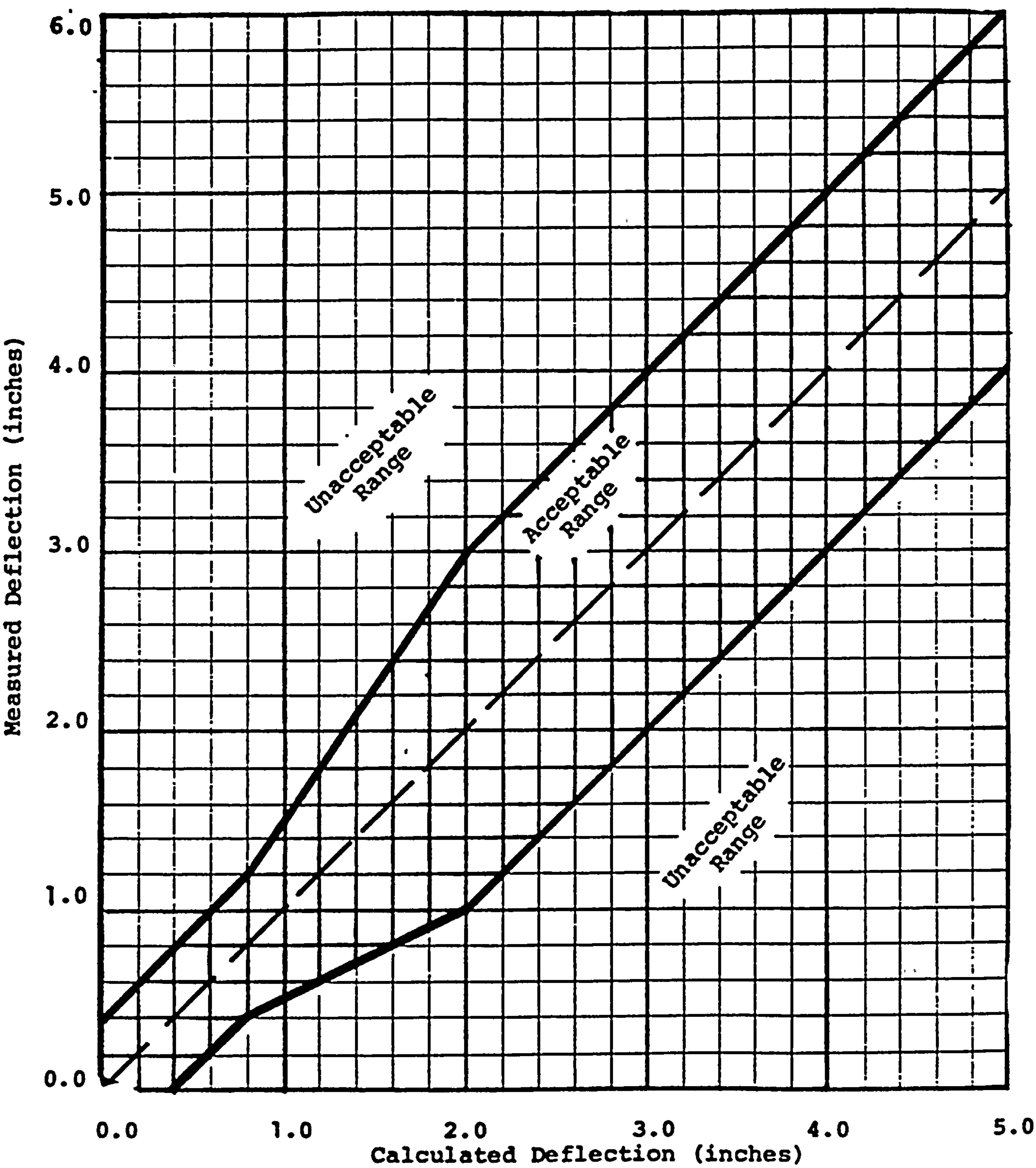
List of Walkdown Packages

Walkdown Package	Analyses	Description
1	2-102	Steam Generator 1 Feedwater Supply Inside Containment
2	2-100	Steam Generator 2 Feedwater Supply Inside Containment
3	2-101	Steam Generator 3 Feedwater Supply Inside Containment
4	2-103	Steam Generator 4 Feedwater Supply Inside Containment
5	1-106	Steam Generator 1 Outlet
6	1-119	Steam Generator 2 Outlet
7	1-120	Steam Generator 3 Outlet
8	1-121	Steam Generator 4 Outlet
9	1-101, 1-102, 1-105	Steam Generator 1 Blowdown Inside Containment
10	1-103, 1-107	Steam Generator 2 Blowdown Inside Containment
11	1-110	Steam Generator 3 Blowdown Inside Containment
12	1-100, 1-104	Steam Generator 4 Blowdown Inside Containment
13	1-111	Steam Generator 1 Blowdown Outside Containment
14	1-112	Steam Generator 2 Blowdown Outside Containment
15	1-113	Steam Generator 3 Blowdown Outside Containment

List of Walkdown Packages

Walkdown Package	Analyses	Description
16	1-114	Steam Generator 4 Blowdown Outside Containment
17	MS-1	Steam Generator 1 Outlet Outside Containment
18	MS-2	Steam Generator 2 Outlet Outside Containment
19	MS-3	Steam Generator 3 Outlet Outside Containment
20	MS-4	Steam Generator 4 Outlet Outside Containment
21	FW1-4	Steam Generator 4 Feedwater Outside Containment

ACCEPTANCE CRITERIA CHART



Power Ascension Walkdown Cover Sheet

Power Ascension Walkdown Package Number: ____________

Piping System Description: ______________________________

____

Analysis Numbers: ______________________________

- -

Cold Position Measurement - By:

______________________	________ Date
______________________	________ Date

- -

Power Ascension Walkdown - By:

			Problem Report Numbers
30%	______________	______ Date	______________
	______________	______ Date	
50%	______________	______ Date	______________
	______________	______ Date	
75%	______________	______ Date	______________
	______________	______ Date	
100%	______________	______ Date	______________
	______________	______ Date	

- -

Engineering Acceptance

______________________	________ Date

- -

Power Ascension Walkdown Problem Report

Power Ascension Walkdown Package Number: ______________ Problem No.: _________

Piping System Description: ____

Analysis Number of Piping Where Problem is Located: ____________________
(list only one)

Describe Problem: ____

____

____

____

____

____

Temperature: _____________________ Power Level: _________________

By: ____ ___________
Date

____ ___________
Date

Resolution: ____

____

____

____

____

By: ______________________________ ___________
Date

Engineering Acceptance

By: ______________________________ ___________
Date

Power Ascension Walkdown Piping Deflections

Power Ascension Walkdown Package Number: ______________________

Description of Test Condition: ______________________

Power Level (%)	Data Point	Direction of Deflection	Cold Position (in)	Hot Position (in)	Measured Deflection (in)	Calculated Deflection (in)	Remarks

ENCLOSURE 6

LICENSE CONDITION 2.C.(11), Item 6

"PGandE shall conduct a review of the "Pipe Support Design Tolerance Clarification" program (PSDTC) and "Diablo Problem" system (DP) activities. The review shall include specific identification of the following:

(a) Support changes which deviated from the defined PSDSTC program scope;

(b) Any significant deviations between as-built and design configurations stemming from the PSDTC or DP activities; and

(c) Any unresolved matters identified by the DP system.

The purpose of this review is to ensure that all design changes and modifications have been resolved and documented in an appropriate manner. Upon completion, PGandE shall submit a report to the NRC Staff documenting the results of this review."

PGandE ACTION/STATUS

A program has been established to demonstrate that the DP and PSDTC programs, and their implementation, have not detracted from appropriate resolution and documentation of all design changes and modifications. The program consists of two components; one addresses DPs and the other PSDTCs.

DP Program

The DP program was originally established to facilitate and expedite the handling of questions or requests made by the construction department to the engineering department. DPs served as a means to status and track such questions and requests. This program was separate from and not to be used in lieu of the engineering design processes controlled by engineering manual procedures in accordance with Quality Assurance Program requirements.

To verify that the DP program for pipe supports has been conducted properly, all DPs issued prior to August 10, 1982, were reviewed to identify those associated with piping and pipe supports. The August 10, 1982, cutoff date has been established based on the results of the NRC inspection and review of the DP issue which concluded that adequate procedures were in place by that date for control of DPs. 3097 DPs were issued during the time period being reviewed. To date, 1041 of these DPs have been reviewed of which 298 are related to piping. The majority of these DPs clearly do not contain inappropriate design information and typically either make reference to the appropriate design change document or merely provide schedule or other non-design information. To date only 52 DPs have been identified which contain information that should have been transmitted by a formal design, design change, or other means allowed by the Quality Assurance Program. Included in this count are a number of DPs for non-seismic Category I installations which will be eliminated by further review.

Each DP containing design-related information will be investigated to assure that the associated installation is acceptable. One of the following disposition categories will apply:

a. The information contained in the DP is also provided by a drawing or other document which is part of the formal design/construction process.

b. The design associated with the DP has been superseded.

c. The information contained in the DP is supported by calculations or data provided in accordance with the Quality Assurance Program.

To date, 25 of the 52 DPs have been reviewed and, in all cases, documentation has been identified which demonstrates compliance with Quality Assurance Program requirements for design control. The results are considered preliminary until checking, now in progress, is complete. The review of the remaining DPs is scheduled for completion by May 14, 1984.

PSDTC Program

The PSDTC program was established to allow minor pipe support design changes to be made by a qualified pipe support engineer located at the construction site, providing design criteria were not violated. In all cases these changes were included in the as-built drawing of the support and the as-built drawing was subsequently reviewed, checked, and approved under the formal engineering process. This program was addressed in detail in PGandE letter DCL-84-131 dated April 4, 1984, pages 37 to 39.

To verify that this program provided designs which are properly documented to comply with the design criteria, regardless of the magnitude of the design changes, approximately 1100 small and large bore pipe support design changes of the 15,000 design changes allowed by the PSDTC program were reviewed to identify the more significant design changes. Based on the judgment of the reviewer, 40 of the most significant design changes, involving 20 small bore and 20 large bore supports, were selected for further, more detailed review. The more detailed review was made to verify that:

a. The change made under the PSDTC program was included in the as-built drawing.

b. The appropriate calculations associated with the drawing were updated to show qualification of the change as part of the as-built acceptance procedure.

The review is complete. All 40 changes were found to be incorporated in the drawing and qualified by the associated calculations. Attachment 6-1 lists the 40 supports and provides a brief description of the changes.

ATTACHMENT 6-1

Large Bore Piping

S.R. NO.	T.C. NO.	HANGER NO.	CALC. NO.	REV. NO.	REASON FOR CHANGE	INCORPORATED IN CALCULATION
1	1-14013	384-304R	S4220	R-5	Structural Interference	Yes
2	1-14329	92-9R	S1165	R-4	Structural Interference	Yes
3	1-14432	99-224R	S5391	R-4	Provide Welding Access	Yes
4	1-14006	15-60SL	S4960	R-5	Redesign to Suite Valve Body	Yes
5	1-14704	92-155R	S6285	R-1	Provide Access to Equipment	Yes
6	1-14496	100-103R	S5988	R-1	Rebar Interference	Yes
7	1-14881	63-35A	A-147	R-0	Structural Interference	Yes
8	1-14562	286-70R	S5724	R-2	Redesign to Suite Valve Body	Yes
9	A-14627	56S-155R	S-6164	R-1	Structural Interference	Yes
10	1-14577	86-84R	S-6257	R-1	Space Limitations	Yes
11	1-14558	286-74R	S-5728	R-2	Structural Interference	Yes
12	1-14463	1033-14SL	1-163	R-5	Structural Interference	Yes
13	1-14847	1033-14SL	I-163	R-5	Structural Interference	Yes
14	1-14842	1033-14SL	I-163	R-5	Structural Interference	Yes
15	1-14502	86-104R	S-6255	R-0	Interference with Expansion Joint	Yes
16	1-13810	15-255L	S1587	R-4	Space Limitation	Yes
17	1-14581	56S-157R	561661	R-3	Rebar Interference	Yes
18	1-14516	84-103R	S6254	R-1	Interference with Grating	Yes
19	1-14617	1032-196	I-126	R-6	Space Limitation	Yes
20	1-14591	1032-135L	I-175	R-3	Interference with FW Lines	Yes

ATTACHMENT 6-1 (cont'd)

Small Bore Piping

S.R. NO.	T.C. NO.	HANGER NO.	CALC. NO.	REV. NO.	REASON FOR CHANGE	INCORPORATED IN CALCULATION
1	1-13870	84-74	MP-422	R-5	Structural Interference	Yes
2	1-14818	2161-19	MP-1549	R-2	Material Availability	Yes
3	1-4820	2161-19	MP-1549	R-2	Base Plate Problems	Yes
4	1-14850	2161-19	MP-1549	R-2	Space Limitation	Yes
5	1-14980	FPS-337	MP-388	R-3	Material Availability	Yes
6	1-14940	FPS-337	MP-388	R-3	Provide Welding Access	Yes
7	1-14936	FPS-337	MP-388	R-3	Rebar Interference	Yes
8	1-114935	99-437	MP-1137	R-4	Welding Problem	Yes
9	1-14943	99-437	MP-1137	R-4	Minimum Edge Distance	Yes
10	1-14948	99-437	MP-1137	R-4	Base Plate Size	Yes
11	1-14947	99-437	MP-1137	R-4	Welding Problem	Yes
12	1-14021	55-6	MP-216	R-3	Rebar Interference	Yes
13	1-14849	2155-82	MP1732	R-2	Structural Interference	Yes
14	1-14173	2155-255	MP290	R-5	Structural Interference	Yes
15	1-14580	2165-55	MP1700	R-1	Equipment Interference	Yes
16	1-14294	2161-50	MP1322	R-3	Welding Problem	Yes
17	1-17749	94-23	MP732	R-4	Structural Interference	Yes
18	1-14061	98-22	MP1720	R-2	Interference with L.B. Support	Yes
19	1-14026	2179-235	MP1684	R-2	Welding Problem	Yes
20	1-14036	72-89	MP1397	R-3	Structural Interference	Yes

ENCLOSURE 7

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."

PGandE ACTION/STATUS

(a) 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 flanges 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 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.

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

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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.

Small Bore

To quantify these effects for the Diablo Canyon pipe supports, warping bending and warping shear stresses are being evaluated for small bore pipe supports analyzed by STRUDL as described in Enclosure 1. The Project will assess the significance of these effects for the previously reviewed supports that have not been re-reviewed for these effects. If these effects are found to cause modifications, then they will be considered for the remainder of the supports; otherwise, the evaluation of these effects will be terminated.

The results of this evaluation will be reviewed with the NRC Staff and any additional required action will be established at that time.

Large Bore

A random sample of 200 large bore supports will be selected. The sample will be reviewed to identify those supports having significant torsion in open sections. Those supports will be reviewed utilizing the data contained in Project Instruction I-55 (Attachment 1-1 to Enclosure 1) to determine the normal stresses due to warping. The warping shear stresses will also be calculated and included in the evaluation.

The results of this review will be discussed with the NRC Staff and any additional action will be determined at that time.

(b) 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

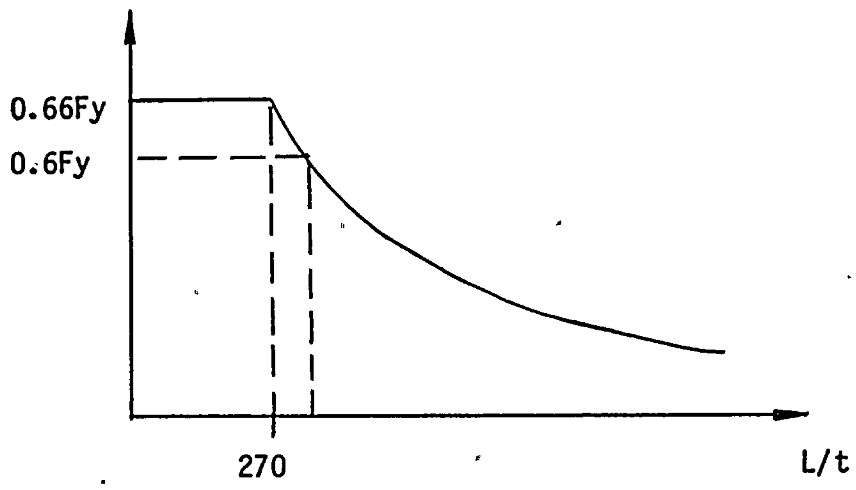
limitations of the Code but also suggests special investigation by the engineer. This position has been addressed in two previous responses to the NRC on the same subject (for convenience excerpts from these submittals are included as Attachment No. 7(b)-1 and 7(b)-2).

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:

- o 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.

- o 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.

(c) 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.

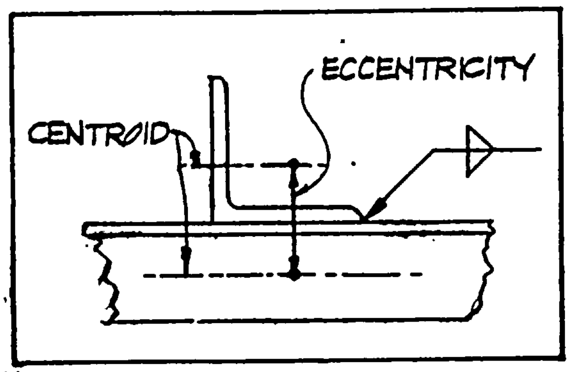
For reasons discussed above, lateral/torsional buckling under axial conditions is not a concern in the DCP design of pipe supports using angle members.

Reference

Alexander Chajes and George Winter, "Torsional-Flexural Buckling of Thin-Walled Members", Journal, Structural Division, ASCE, ST4, August, 1965.

(d) 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 as follows:

Small Bore

As discussed in Enclosure 1, approximently 50% of the remainder of the small bore calculations are currently under review for this issue. This is an explicit issue that will be addressed for each design calculation during this review. The results will be documented and discussed with the NRC Staff and any additional work activities will be determined.

Large Bore

As discussed in paragraph (a) above, 200 large bore pipe supports selected at random will be reviewed for any eccentric conditions not specifically included in the computer model. Where this condition is found, the support will be reevaluated to account for the eccentricity. The results of this review will be documented and discussed with the NRC Staff. A determination will then be made as to the necessity for further action.

(e) Fundamental Frequency By The Rayleigh Method

All supports will be reviewed to identify cases where the Rayleigh method of determining the frequency may not have approximated the dominant frequency with sufficient accuracy. The Rayleigh approximation, in general, used a deflected shape proportional to that caused by gravity loads. This approach 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 Rayleigh 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.

(f) 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 required.

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

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 them 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

J. M. LEIGH, B. F. THOMAS AND M. G. LAY*

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

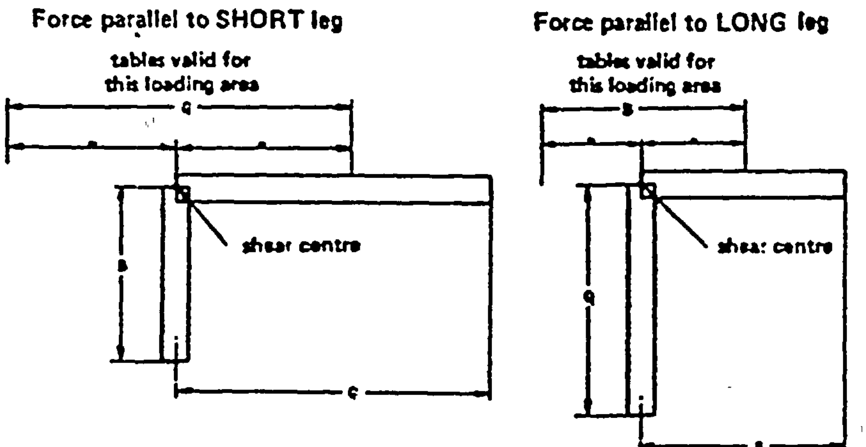


Fig. 1. Acceptable load locations

* The authors, who also computed the tables, are officers of the Melbourne Research Laboratories of the BHP Co. Ltd.

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
$\lambda_1\ \lambda_2$	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, $\lambda_1 = 1.0 \qquad \lambda_2 = ⅓$

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	0.753	0.405
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	0.766	0.404
3 x 2½	1.19	−0.303	0.858	0.368
3 x 2	1.60	−0.273	0.726	0.422
2½ x 2	1.23	−0.298	0.831	0.379

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

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_y 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_c 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

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

Force parallel to long leg

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

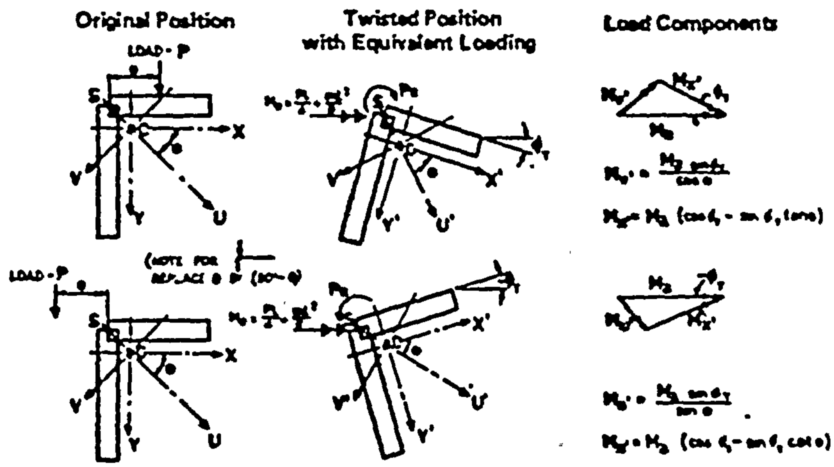


Fig. 2. Method of determination of load components for deflection equations

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

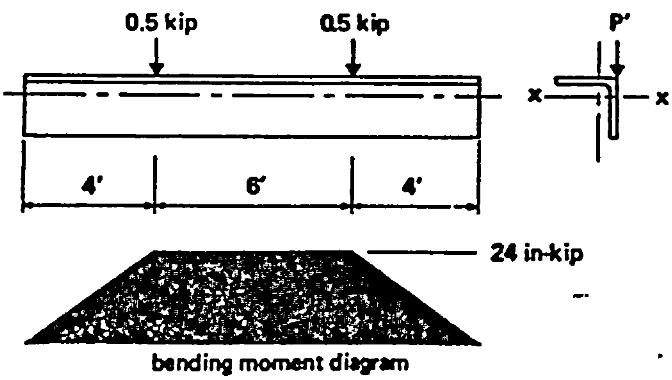
$$\Delta'' = \frac{P'' + mL}{P' + mL} \times \Delta \tag{15}$$

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

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

which satisfies the limit of 0.93''.

REFERENCES

1. *Standards Association of Australia* Use of Steel in Structures *AS CA1, 1968.*
2. *Australian Institute of Steel Construction* Safe Load Tables for Structural Steel *Australia, 1969.*
3. *Lay, M. G.* AS CA1—A Review and Explanation *Steel Construction, Special Issue, AISC (Australia), 1968.*
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5. *Leigh, J. M. and M. G. Lay* Laterally Unsupported Angles with Equal and Unequal Legs *BHP Melbourne Research Laboratory Report MRL 22/2, July 1970.*
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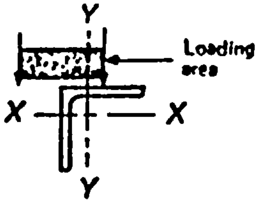
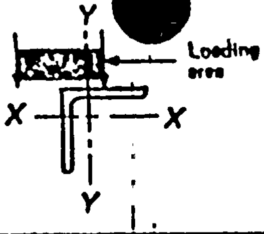


EQUAL ANGLES

HIGH STRENGTH STEEL (52 ksi)

BASED ON AS CA1 1968



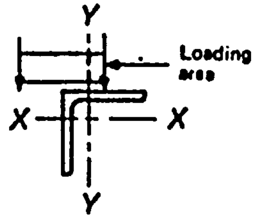
NOMINAL SIZE	THICKNESS t NOMINAL	WEIGHT PER FOOT	SAFE DISTRIBUTED LOADS (KIPS) / MAX DEFLECTION (in) SPANS IN FEET								
in	in	lb	2	4	6	8	10	12	14	20	25
8 x 8	1	51.0	49.3 / 0.01	49.3 / 0.05	47.0 / 0.17	36.0 / 0.30	27.9 / 0.48	23.0 / 0.67	19.6 / 0.91	13.2 / 1.86	10.1 / 2.9
	7/8	45.0	38.8 / 0.01	38.8 / 0.04	38.8 / 0.16	31.1 / 0.29	24.7 / 0.46	20.4 / 0.66	17.4 / 0.91	11.7 / 1.86	8.96 / 2.9
	3/4	38.9	29.3 / 0.01	29.3 / 0.04	29.3 / 0.13	27.1 / 0.29	21.8 / 0.46	17.8 / 0.66	15.1 / 0.90	10.2 / 1.86	7.78 / 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	8.55 / 1.87	6.51 / 2.9
	1/2	26.6	13.8 / 0.00	13.8 / 0.03	13.8 / 0.09	13.8 / 0.27	13.8 / 0.44	12.0 / 0.66	10.2 / 0.91	6.82 / 1.87	5.14 / 3.1
6 x 6	1	37.5	46.8 / 0.02	38.6 / 0.10	25.5 / 0.23	19.0 / 0.41	15.1 / 0.63	12.4 / 0.91	10.6 / 1.25	6.98 / 2.6	5.10 / 4.0
	7/8	33.1	38.3 / 0.02	34.5 / 0.10	22.9 / 0.23	17.0 / 0.40	13.5 / 0.63	11.1 / 0.91	9.42 / 1.24	6.07 / 2.6	4.56 / 4.0
	3/4	28.7	27.7 / 0.01	27.7 / 0.09	19.8 / 0.22	14.6 / 0.39	11.6 / 0.61	9.52 / 0.89	7.99 / 1.20	5.30 / 2.5	3.98 / 3.9
	5/8	24.2	20.0 / 0.01	20.0 / 0.08	16.6 / 0.22	12.4 / 0.39	9.81 / 0.61	8.08 / 0.88	6.84 / 1.20	4.49 / 2.5	3.38 / 3.9
	9/16	21.9	16.2 / 0.01	16.2 / 0.08	15.0 / 0.22	11.1 / 0.39	8.86 / 0.61	7.38 / 0.88	6.00 / 1.20	4.08 / 2.5	3.10 / 3.9
	1/2	19.5	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	8.78 / 0.39	6.96 / 0.61	5.86 / 0.88	4.79 / 1.19	3.18 / 2.6	2.39 / 4.0
	3/8	14.9	7.77 / 0.01	7.77 / 0.05	7.77 / 0.17	7.50 / 0.39	5.95 / 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	26.5 / 0.02	20.4 / 0.12	13.5 / 0.27	10.1 / 0.48	7.98 / 0.76	6.56 / 1.09	5.53 / 1.49	3.63 / 3.0	2.61 / 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	5.45 / 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 / 0.29	6.84 / 0.47	5.41 / 0.73	4.46 / 1.06	3.76 / 1.45	2.47 / 3.0	1.79 / 4.7
	3/8	12.3	7.69 / 0.01	7.69 / 0.09	7.01 / 0.26	5.21 / 0.46	4.13 / 0.73	3.39 / 1.06	2.86 / 1.45	1.83 / 3.0	1.36 / 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	2.85 / 1.07	2.40 / 1.48	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								
in	in	lb	2	4	6	8	10	12	14	20	25
4 x 4	5/8	15.7	18.2 / 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 / 3.8	1.29 / 6.0
	1/2	12.7	12.3 / 0.03	8.09 / 0.15	5.75 / 0.33	4.27 / 0.59	3.37 / 0.93	2.78 / 1.34	2.32 / 1.84	1.49 / 3.8	1.08 / 6.0
	3/8	9.7	7.33 / 0.02	6.69 / 0.14	4.43 / 0.33	3.29 / 0.59	2.59 / 0.92	2.13 / 1.34	1.79 / 1.82	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 / 0.59	2.15 / 0.92	1.76 / 1.34	1.48 / 1.85	0.94 / 3.9	0.66 / 6.3
	1/4	6.6	3.45 / 0.02	3.45 / 0.11	3.00 / 0.33	2.23 / 0.59	1.76 / 0.94	1.44 / 1.37	1.19 / 1.87	0.73 / 4.0	0.51 / 6.7
3½ x 3½	5/8	13.5	16.2 / 0.04	8.06 / 0.17	5.33 / 0.39	3.95 / 0.70	3.11 / 1.10	2.54 / 1.58	2.13 / 2.2	1.30 / 4.4	0.92 / 7.0
	1/2	11.0	11.9 / 0.04	6.70 / 0.17	4.43 / 0.39	3.28 / 0.69	2.66 / 1.07	2.08 / 1.55	1.75 / 2.1	1.08 / 4.4	0.76 / 7.0
	3/8	8.4	7.16 / 0.03	5.13 / 0.17	3.39 / 0.38	2.51 / 0.68	1.98 / 1.06	1.67 / 1.54	1.34 / 2.1	0.84 / 4.3	0.59 / 7.1
	5/16	7.1	5.00 / 0.02	4.26 / 0.17	2.83 / 0.39	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	2.23 / 0.38	1.66 / 0.67	1.32 / 1.05	1.08 / 1.54	0.89 / 2.1	0.55 / 4.6	0.38 / 7.9
3 x 3	5/8	11.4	11.5 / 0.05	5.72 / 0.21	3.77 / 0.47	2.79 / 0.83	2.19 / 1.30	1.78 / 1.87	1.49 / 2.6	0.91 / 5.3	0.61 / 8.2
	1/2	9.3	9.64 / 0.05	4.79 / 0.20	3.18 / 0.46	2.34 / 0.81	1.84 / 1.28	1.47 / 1.83	1.22 / 2.5	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 / 1.25	1.15 / 1.81	0.96 / 2.5	0.58 / 5.2	0.40 / 8.2
	5/16	6.0	4.92 / 0.04	3.07 / 0.19	2.03 / 0.44	1.50 / 0.79	1.19 / 1.24	0.96 / 1.80	0.80 / 2.5	0.48 / 5.1	0.32 / 8.3
	1/4	4.9	3.32 / 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	0.26 / 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.

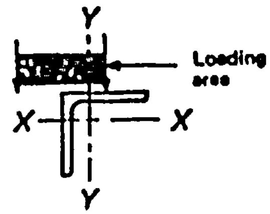
Safe Load Tables for Laterally Unsupported Angles
Normal-Strength Steel—36 ksi yield stress



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
2½ x 2½	½	7.7	6.45 0.06	3.20 0.25	2.11 0.56	1.58 0.99	1.22 1.58	0.99 2.3	0.79 3.1	0.47 6.3	
	⅜	5.9	5.11 0.06	2.54 0.24	1.67 0.56	1.20 0.96	0.94 1.52	0.78 2.2	0.63 3.0	0.37 6.1	
	5/16	5.0	4.20 0.06	2.09 0.24	1.37 0.53	1.01 0.96	0.79 1.51	0.64 2.2	0.53 3.0	0.31 6.2	
	¼	4.0	3.22 0.05	1.73 0.23	1.14 0.53	0.83 0.96	0.66 1.50	0.53 2.2	0.43 3.0	0.24 6.4	
	3/16	3.1	1.81 0.04	1.32 0.23	0.86 0.53	0.64 0.94	0.49 1.49	0.40 2.2	0.32 3.0	0.19 6.5	
2¼ x 2¼	⅜	5.2	4.06 0.07	2.02 0.27	1.33 0.61	0.98 1.09	0.75 1.70	0.61 2.5	0.49 3.4	0.29 7.0	
	5/16	4.4	3.45 0.07	1.71 0.27	1.11 0.60	0.82 1.07	0.64 1.69	0.52 2.5	0.42 3.3	0.24 6.9	
	¼	3.6	2.81 0.07	1.38 0.26	0.91 0.59	0.67 1.07	0.52 1.66	0.42 2.4	0.34 3.3	0.20 6.9	
	3/16	2.7	1.88 0.06	1.06 0.26	0.70 0.59	0.51 1.06	0.40 1.67	0.32 2.4	0.26 3.3	0.14 7.1	
2 x 2	⅜	4.6	3.14 0.08	1.56 0.31	1.02 0.69	0.75 1.24	0.57 1.93	0.46 2.8	0.37 3.8	0.21 7.9	
	5/16	3.9	2.67 0.08	1.33 0.30	0.87 0.69	0.63 1.22	0.49 1.92	0.39 2.8	0.32 3.8	0.18 8.0	
	¼	3.2	2.19 0.07	1.09 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	
	3/16	2.4	1.69 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	
	⅛	1.7	0.86 0.05	0.56 0.29	0.37 0.67	0.26 1.22	0.20 2.0	0.16 2.9	0.13 4.0		

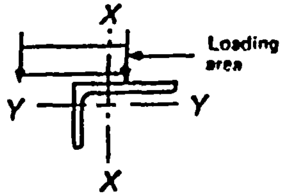
Deflection values to the right of the heavy line are greater than 1/180 of the span.

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	⅜	4.0	2.33 0.09	1.16 0.36	0.78 0.80	0.56 1.43	0.43 2.3	0.33 3.3	0.27 4.4	0.14 9.1	
	5/16	3.4	2.00 0.09	0.99 0.36	0.65 0.79	0.46 1.41	0.36 2.2	0.29 3.2	0.23 4.3	0.12 9.0	
	¼	2.7	1.68 0.09	0.83 0.33	0.53 0.77	0.39 1.39	0.30 2.2	0.24 3.1	0.19 4.3	0.10 9.2	
	3/16	2.1	1.29 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		
	⅛	1.5	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	5/16	2.86	1.41 0.10	0.70 0.41	0.46 0.94	0.33 1.67	0.26 2.6	0.20 3.8	0.16 5.1		
	¼	2.34	1.20 0.10	0.58 0.41	0.38 0.92	0.28 1.64	0.21 2.6	0.17 3.7	0.13 5.1		
	3/16	1.8	0.92 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		
	⅛	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	5/16	2.3	0.96 0.13	0.47 0.51	0.30 1.15	0.22 2.1	0.18 3.2	0.13 4.7	0.10 6.3		
	¼	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			
	3/16	1.48	0.62 0.12	0.30 0.48	0.20 1.10	0.14 2.0	0.11 3.1				
	⅛	1.0	0.44 0.12	0.22 0.48	0.14 1.10	0.10 2.0					
1 x 1	¼	1.5	0.49 0.18	0.24 0.64	0.15 1.44	0.11 2.6					
	3/16	1.16	0.39 0.15	0.19 0.62	0.12 1.40						
	⅛	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.

Safe Load Tables for Laterally Unsupported Angles

Normal-Strength Steel—36 ksi yield stress

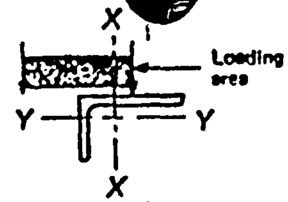


UNEQUAL ANGLES

Force parallel to SHORT leg

HIGH STRENGTH STEEL (52 ksi)

BASED ON AS CA1 1968



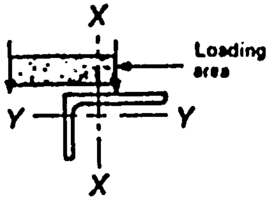
NOMINAL SIZE	THICKNESS t NOMINAL	WEIGHT PER FOOT	SAFE DISTRIBUTED LOADS (KIPS) / MAX DEFLECTION (in) SPANS IN FEET								
in	in	lb	2	4	6	8	10	12	14	20	25
6 x 4	¾	23.6	21.4 / 0.03	13.2 / 0.14	8.69 / 0.32	6.44 / 0.58	5.06 / 0.91	4.13 / 1.31	3.48 / 1.78	2.18 / 3.7	1.83 / 5.3
	⅝	19.9	15.8 / 0.03	11.3 / 0.14	7.45 / 0.32	5.52 / 0.58	4.35 / 0.90	3.55 / 1.31	2.97 / 1.78	1.97 / 3.7	1.32 / 5.9
	½	16.1	10.4 / 0.02	9.28 / 0.15	6.13 / 0.31	4.54 / 0.58	3.57 / 0.92	2.92 / 1.32	2.44 / 1.81	1.55 / 3.8	1.09 / 6.0
	7/16	14.2	8.17 / 0.02	8.17 / 0.15	5.45 / 0.33	4.04 / 0.59	3.18 / 0.93	2.60 / 1.35	2.17 / 1.85	1.38 / 3.9	0.98 / 6.2
	⅜	12.3	6.19 / 0.02	6.19 / 0.13	4.76 / 0.34	3.53 / 0.61	2.78 / 0.95	2.27 / 1.39	1.90 / 1.92	1.21 / 4.1	0.85 / 6.6
6 x 3½	⅝	18.8	14.8 / 0.03	8.64 / 0.18	5.69 / 0.37	4.20 / 0.66	3.29 / 1.03	2.68 / 1.48	2.22 / 2.0	1.36 / 4.2	0.92 / 6.6
	½	15.3	9.73 / 0.03	7.15 / 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
	7/16	13.8	7.70 / 0.02	6.39 / 0.17	4.21 / 0.38	3.11 / 0.67	2.44 / 1.06	1.99 / 1.53	1.65 / 2.1	1.02 / 4.4	0.69 / 7.1
	⅜	11.6	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
	5/16	9.7	4.07 / 0.02	4.07 / 0.18	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	⅝	16.7	15.2 / 0.04	8.35 / 0.17	5.51 / 0.37	4.08 / 0.66	3.20 / 1.04	2.61 / 1.50	2.17 / 2.0	1.35 / 4.2	0.93 / 6.7
	½	13.6	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
	⅜	10.3	6.10 / 0.02	5.38 / 0.17	3.55 / 0.38	2.61 / 0.68	2.06 / 1.07	1.68 / 1.56	1.40 / 2.1	0.88 / 4.5	0.61 / 7.4
	5/16	8.7	4.38 / 0.02	4.38 / 0.18	3.02 / 0.39	2.24 / 0.71	1.76 / 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	½	12.7	9.44 / 0.05	5.10 / 0.19	3.36 / 0.43	2.47 / 0.77	1.93 / 1.21	1.58 / 1.74	1.29 / 2.4	0.77 / 5.0	0.50 / 7.9
	7/16	11.2	7.46 / 0.04	4.55 / 0.19	2.99 / 0.43	2.21 / 0.77	1.72 / 1.21	1.39 / 1.76	1.16 / 2.4	0.69 / 5.0	0.45 / 8.1
	⅜	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
	5/16	8.1	4.01 / 0.03	3.36 / 0.20	2.21 / 0.45	1.63 / 0.81	1.28 / 1.29	1.03 / 1.86	0.86 / 2.6	0.51 / 5.5	0.34 / 9.0
	¼	6.6	2.72 / 0.02	2.72 / 0.21	1.82 / 0.48	1.34 / 0.86	1.05 / 1.36	0.85 / 2.0	0.71 / 2.8	0.43 / 6.1	0.28 / 10.2
4 x 3	⅝	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	1.56 / 2.4	0.95 / 5.0	0.63 / 8.0
	½	11.0	9.69 / 0.05	4.91 / 0.19	3.24 / 0.44	2.39 / 0.78	1.87 / 1.22	1.52 / 1.77	1.26 / 2.4	0.77 / 5.1	0.52 / 8.1
	⅜	8.4	6.07 / 0.04	3.84 / 0.20	2.53 / 0.44	1.87 / 0.79	1.47 / 1.24	1.19 / 1.80	0.99 / 2.6	0.61 / 5.3	0.41 / 8.6
	5/16	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	0.34 / 8.8
	¼	5.8	2.95 / 0.03	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 / 5.8	0.27 / 9.3
3½ x 3	½	10.2	9.78 / 0.05	4.86 / 0.20	3.21 / 0.45	2.37 / 0.78	1.86 / 1.25	1.51 / 1.80	1.26 / 2.5	0.78 / 5.1	0.53 / 8.2
	⅜	7.9	6.45 / 0.04	3.80 / 0.20	2.51 / 0.44	1.85 / 0.79	1.45 / 1.25	1.18 / 1.81	0.99 / 2.5	0.59 / 5.1	0.40 / 8.2
	5/16	6.6	4.56 / 0.04	3.21 / 0.20	2.12 / 0.45	1.57 / 0.80	1.23 / 1.28	1.00 / 1.84	0.81 / 2.5	0.50 / 5.2	0.33 / 8.5
	¼	5.3	3.06 / 0.01	2.64 / 0.20	1.69 / 0.44	1.25 / 0.80	0.98 / 1.26	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
Normal-Strength Steel—36 ksi yield stress



UNEQUAL ANGLES

Force parallel to SHORT leg

HIGH STRENGTH STEEL (52 ksi)

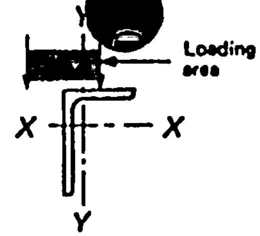
BASED ON AS CA1 1968

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
3½ x 2½	½	9.4	6.73 0.06	3.33 0.23	2.19 0.53	1.61 0.94	1.26 1.47	1.00 2.1	0.83 2.9	0.49 6.1	0.30 9.7
	⅜	7.1	5.26 0.06	2.80 0.23	1.71 0.52	1.28 0.94	0.98 1.47	0.79 2.1	0.65 2.9	0.38 6.2	0.24 10.1
	5/16	6.0	4.14 0.06	2.23 0.23	1.47 0.53	1.08 0.95	0.84 1.49	0.68 2.2	0.56 3.0	0.33 6.4	0.21 10.6
	¼	4.8	2.73 0.05	1.82 0.24	1.20 0.56	0.88 0.98	0.69 1.56	0.56 2.3	0.46 3.2	0.26 6.7	0.18 11.2
	3/16	3.7	1.66 0.03	1.43 0.25	0.94 0.58	0.69 1.05	0.54 1.68	0.44 2.5	0.35 3.4	0.20 7.5	0.12 12.9
3 x 2½	½	8.5	6.68 0.06	3.26 0.24	2.15 0.54	1.58 0.96	1.23 1.51	1.00 2.2	0.82 3.0	0.49 6.2	0.32 9.9
	⅜	6.5	5.17 0.06	2.57 0.24	1.69 0.53	1.26 0.95	0.97 1.50	0.78 2.2	0.65 3.0	0.39 6.3	0.24 9.9
	5/16	5.5	4.43 0.06	2.20 0.24	1.45 0.53	1.07 0.95	0.84 1.51	0.68 2.2	0.56 3.0	0.32 6.3	0.21 10.2
	¼	4.4	2.90 0.05	1.79 0.24	1.18 0.54	0.87 0.97	0.68 1.54	0.53 2.2	0.44 3.0	0.28 6.6	0.18 10.8
	3/16	3.4	1.78 0.04	1.36 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	½	7.7	4.13 0.07	2.04 0.29	1.33 0.66	0.97 1.19	0.76 1.86	0.60 2.7	0.48 3.7	0.26 7.7	0.14 12.2
	⅜	5.9	3.30 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
	5/16	5.0	2.84 0.07	1.40 0.29	0.92 0.65	0.67 1.17	0.52 1.85	0.42 2.7	0.34 3.7	0.18 7.9	0.10 13.0
	¼	4.0	2.32 0.07	1.15 0.29	0.75 0.66	0.55 1.19	0.42 1.89	0.34 2.8	0.27 3.8	0.15 8.4	
	3/16	3.1	1.66 0.07	0.90 0.31	0.59 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	⅜	5.2	3.18 0.07	1.67 0.30	1.03 0.67	0.76 1.19	0.58 1.87	0.47 2.7	0.38 3.7	0.21 7.9	0.12 12.9
	5/16	4.4	2.72 0.07	1.36 0.29	0.88 0.67	0.65 1.19	0.50 1.88	0.40 2.7	0.33 3.8	0.19 8.1	0.10 12.9
	¼	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	0.16 8.3	
	3/16	2.7	1.57 0.07	0.85 0.30	0.56 0.69	0.40 1.23	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)

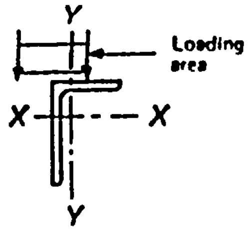
BASED ON AS CA1 1968



NOMINAL SIZE	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	33.3 0.02	27.3 0.10	18.1 0.23	13.6 0.42	10.7 0.86	8.84 0.96	7.49 1.29	5.00 2.7	3.71 4.3
	⅝	19.9	24.0 0.02	23.1 0.10	16.3 0.23	11.4 0.41	9.07 0.86	7.49 0.94	6.34 1.29	4.19 2.7	3.12 4.4
	½	16.1	16.0 0.01	16.0 0.08	12.4 0.23	9.26 0.41	7.36 0.85	6.08 0.95	5.14 1.32	3.36 2.7	2.40 4.4
	7/16	14.2	12.6 0.01	12.6 0.08	10.9 0.23	8.14 0.42	6.46 0.86	5.34 0.97	4.38 1.32	2.82 2.8	2.08 4.7
	⅜	12.3	9.43 0.01	9.43 0.07	9.42 0.23	7.02 0.42	5.41 0.85	4.44 0.97	3.32 1.31	2.37 2.9	1.71 4.9
6 x 3½	⅝	18.8	26.4 0.02	22.8 0.10	16.0 0.23	11.1 0.42	8.80 0.66	7.34 0.97	6.19 1.34	3.97 2.8	2.90 4.6
	½	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 1.36	3.14 2.8	2.28 4.8
	7/16	13.6	13.6 0.01	13.6 0.08	10.8 0.23	7.90 0.42	6.27 0.87	5.17 0.99	4.24 1.34	2.71 2.9	1.93 5.1
	⅜	11.6	10.1 0.01	10.1 0.07	9.03 0.24	6.73 0.43	5.34 0.89	4.28 0.99	3.58 1.38	2.21 3.1	1.64 5.2
	5/16	9.7	7.10 0.01	7.10 0.07	7.10 0.23	5.40 0.43	4.23 0.89	3.42 1.00	2.82 1.47	1.89 3.3	1.14 5.2
5 x 3½	⅝	16.7	22.4 0.02	16.8 0.12	10.5 0.29	7.80 0.60	6.16 0.79	5.08 1.12	4.27 1.84	2.79 3.2	2.07 5.2
	½	13.6	16.1 0.02	13.0 0.12	8.67 0.29	6.36 0.60	5.03 0.77	4.17 1.11	3.48 1.83	2.28 3.3	1.63 5.2
	⅜	10.3	8.86 0.01	8.86 0.11	6.48 0.29	4.79 0.40	3.78 0.77	3.09 1.13	2.60 1.67	1.64 3.3	1.16 5.5
	5/16	8.7	6.35 0.01	6.35 0.09	5.39 0.29	3.97 0.40	3.13 0.79	2.57 1.18	2.17 1.83	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
Normal-Strength Steel—36 ksi yield stress

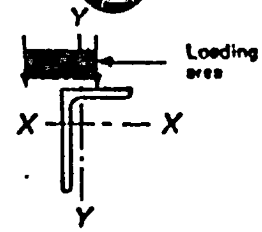


UNEQUAL ANGLES

Force parallel to LONG 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	26
5 x 3	1/2	12.7	16.3 0.02	12.6 0.12	8.31 0.28	6.18 0.51	4.87 0.78	3.98 1.15	3.34 1.59	2.17 3.6	1.53 5.8
	7/16	11.2	12.8 0.02	11.1 0.12	7.33 0.28	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	9.64 0.12	6.30 0.28	4.66 0.51	3.66 0.80	2.98 1.19	2.48 1.69	1.64 3.7	1.07 6.1
	5/16	8.1	6.83 0.01	6.83 0.11	5.20 0.28	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	4.62 0.01	4.62 0.10	4.14 0.29	3.01 0.52	2.34 0.86	1.87 1.32	1.62 1.66	0.86 4.0	0.55 6.4
4 x 3	5/8	13.6	16.1 0.04	10.6 0.16	7.04 0.36	5.09 0.63	4.01 0.99	3.29 1.43	2.78 1.95	1.76 4.0	1.27 6.4
	1/2	11.0	13.8 0.03	8.20 0.15	5.41 0.35	4.02 0.62	3.18 0.98	2.59 1.39	2.17 1.93	1.38 4.0	0.99 6.6
	3/8	8.4	8.23 0.03	6.29 0.15	4.14 0.34	3.08 0.62	2.42 0.96	1.97 1.40	1.65 1.93	1.04 4.2	0.73 6.7
	5/16	7.1	5.98 0.02	5.29 0.15	3.50 0.34	2.59 0.61	2.03 0.97	1.65 1.41	1.37 1.94	0.85 4.2	0.58 7.0
	1/4	5.8	3.98 0.02	3.98 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	6.50 0.18	4.30 0.40	3.11 0.70	2.45 1.09	1.99 1.59	1.67 2.2	1.05 4.4	0.74 7.1
	3/8	7.8	7.69 0.03	4.88 0.17	3.23 0.39	2.40 0.69	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	4.09 0.17	2.70 0.39	2.00 0.69	1.57 1.07	1.27 1.56	1.06 2.2	0.66 4.6	0.45 7.4
	1/4	5.3	3.60 0.02	3.32 0.17	2.20 0.38	1.61 0.69	1.26 1.08	1.07 1.59	0.85 2.2	0.51 4.8	0.34 7.8

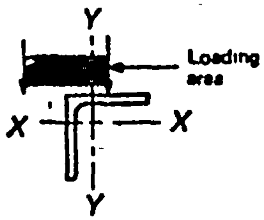
Deflection values to the right of the heavy line are greater than 1/180 of the span.
Load values to the left of the broken line are based on shear capacity, and are less than the permissible flexural load.

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
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	3.10 0.40	2.30 0.71	1.80 1.11	1.46 1.64	1.22 2.2	0.75 4.8	0.51 7.9
	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	3.89 0.02	3.19 0.17	2.08 0.40	1.53 0.71	1.19 1.14	0.96 1.69	0.79 2.4	0.46 5.3	0.29 9.2
	3/16	3.7	2.36 0.02	2.36 0.17	1.68 0.40	1.14 0.74	0.89 1.22	0.70 1.86	0.57 2.6	0.30 5.2	0.17 9.2
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	7.09 0.05	3.53 0.20	2.33 0.46	1.71 0.82	1.34 1.29	1.09 1.86	0.91 2.6	0.56 5.4	0.37 8.8
	5/16	5.6	5.38 0.04	3.00 0.20	1.98 0.45	1.46 0.82	1.14 1.27	0.92 1.86	0.76 2.6	0.47 5.4	0.31 8.8
	1/4	4.4	3.61 0.04	2.41 0.20	1.57 0.46	1.17 0.82	0.90 1.28	0.73 1.88	0.60 2.6	0.36 5.6	0.23 9.1
	3/16	3.4	2.13 0.03	1.83 0.20	1.21 0.46	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	8.98 0.06	4.47 0.22	2.87 0.48	2.12 0.84	1.67 1.35	1.38 1.94	1.13 2.7	0.70 5.7	0.47 9.2
	3/8	5.9	6.84 0.06	3.40 0.21	2.22 0.47	1.66 0.84	1.28 1.35	1.04 1.95	0.88 2.7	0.52 5.8	0.34 9.9
	5/16	5.0	5.83 0.05	2.90 0.21	1.89 0.47	1.40 0.84	1.08 1.33	0.87 1.95	0.72 2.7	0.43 6.1	0.27 10.1
	1/4	4.0	3.90 0.04	2.28 0.21	1.51 0.47	1.10 0.85	0.86 1.38	0.68 2.0	0.56 2.9	0.31 6.3	0.18 10.0
	3/16	3.1	2.37 0.03	1.75 0.21	1.13 0.48	0.82 0.86	0.63 1.46	0.50 2.2	0.40 3.2	0.20 6.5	0.11 10.2
2½ x 2	3/8	5.2	4.83 0.06	2.40 0.25	1.64 0.56	1.14 1.00	0.89 1.57	0.71 2.3	0.58 3.1	0.36 6.6	0.23 10.8
	5/16	4.4	4.01 0.09	1.99 0.24	1.31 0.56	0.97 0.99	0.76 1.57	0.61 2.3	0.50 3.2	0.29 6.6	0.18 10.9
	1/4	3.6	3.28 0.06	1.63 0.24	1.08 0.55	0.79 0.99	0.61 1.66	0.50 2.3	0.40 3.2	0.23 7.1	0.14 11.0
	3/16	2.7	1.98 0.05	1.22 0.24	0.80 0.56	0.58 1.00	0.45 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.

Safe Load Tables for Laterally Unsupported Angles

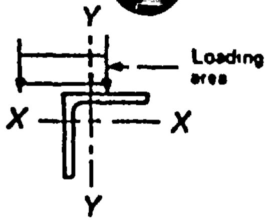
Normal Strength Steel - 36 ksi yield ...



EQUAL ANGLES

NORMAL STRENGTH STEEL (36 ksi)

BASED ON AS CA1 1968



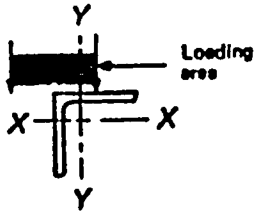
NOMINAL SIZE	THICKNESS t NOMINAL	WEIGHT PER FOOT	SAFE DISTRIBUTED LOADS (KIPS) / MAX DEFLECTION (in) SPANS IN FEET								
in	in	lb	2	4	6	8	10	12	14	20	26
8 x 8	1	51.0	34.1 / 0.00	34.1 / 0.03	32.9 / 0.12	24.5 / 0.30	19.4 / 0.32	16.0 / 0.48	13.8 / 0.63	8.93 / 1.30	6.09 / 2.0
	7/8	45.0	29.9 / 0.00	29.9 / 0.03	26.9 / 0.11	21.8 / 0.20	17.3 / 0.32	14.2 / 0.48	12.0 / 0.63	7.86 / 1.29	5.96 / 2.0
	3/4	38.9	20.3 / 0.00	20.3 / 0.03	20.3 / 0.10	19.0 / 0.20	15.0 / 0.32	12.4 / 0.46	10.6 / 0.63	6.94 / 1.29	5.06 / 1.99
	5/8	32.7	14.8 / 0.00	14.8 / 0.02	14.8 / 0.08	14.8 / 0.19	12.4 / 0.31	10.2 / 0.46	8.81 / 0.61	5.80 / 1.28	4.28 / 1.98
	1/2	26.4	9.57 / 0.00	9.57 / 0.02	9.57 / 0.06	9.57 / 0.15	9.57 / 0.30	8.22 / 0.45	6.96 / 0.61	4.80 / 1.27	3.44 / 2.0
6 x 6	1	37.5	31.7 / 0.01	28.8 / 0.07	17.6 / 0.16	13.1 / 0.28	10.3 / 0.44	8.47 / 0.63	7.12 / 0.86	4.80 / 1.76	3.35 / 2.8
	7/8	33.1	25.2 / 0.01	23.8 / 0.07	15.8 / 0.16	11.7 / 0.28	9.25 / 0.43	7.59 / 0.62	6.38 / 0.86	4.13 / 1.74	3.00 / 2.7
	3/4	28.7	19.2 / 0.01	19.2 / 0.08	13.8 / 0.15	10.3 / 0.28	8.11 / 0.43	6.66 / 0.62	5.60 / 0.86	3.62 / 1.73	2.64 / 2.7
	5/8	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	3.08 / 1.73	2.14 / 2.7
	9/16	21.9	11.4 / 0.01	11.4 / 0.06	10.8 / 0.15	7.83 / 0.27	6.36 / 0.42	5.00 / 0.60	4.23 / 0.83	2.88 / 1.72	1.98 / 2.7
	1/2	19.6	9.20 / 0.01	9.20 / 0.06	9.20 / 0.15	6.92 / 0.27	5.46 / 0.42	4.48 / 0.60	3.77 / 0.82	2.44 / 1.70	1.74 / 2.7
	7/16	17.2	7.08 / 0.01	7.08 / 0.04	7.08 / 0.13	6.05 / 0.27	4.78 / 0.42	3.92 / 0.60	3.29 / 0.82	2.13 / 1.70	1.51 / 2.7
	3/8	14.9	5.38 / 0.01	5.38 / 0.04	5.38 / 0.11	5.26 / 0.36	4.15 / 0.42	3.40 / 0.60	2.86 / 0.82	1.86 / 1.71	1.31 / 2.6
5 x 5	3/4	23.6	18.4 / 0.01	14.1 / 0.08	9.32 / 0.19	6.91 / 0.33	5.44 / 0.52	4.46 / 0.75	3.72 / 1.02	2.37 / 2.1	1.68 / 3.3
	5/8	19.9	13.3 / 0.01	12.1 / 0.08	7.98 / 0.19	5.91 / 0.33	4.66 / 0.52	3.81 / 0.75	3.19 / 1.02	2.03 / 2.1	1.46 / 3.3
	1/2	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	1.18 / 3.2
	3/8	12.3	5.28 / 0.01	5.28 / 0.06	4.88 / 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
	5/16	10.4	3.76 / 0.01	3.76 / 0.06	3.76 / 0.16	3.06 / 0.32	2.40 / 0.50	1.98 / 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	26
4 x 4	5/8	15.7	12.8 / 0.03	7.46 / 0.10	4.91 / 0.24	3.63 / 0.42	2.86 / 0.66	2.31 / 0.94	1.93 / 1.29	1.19 / 2.6	0.81 / 4.2
	1/2	12.7	8.53 / 0.02	6.17 / 0.10	4.07 / 0.23	3.01 / 0.41	2.36 / 0.66	1.92 / 0.93	1.60 / 1.27	0.98 / 2.6	0.64 / 4.1
	3/8	9.7	5.08 / 0.02	4.78 / 0.10	3.16 / 0.23	2.33 / 0.41	1.83 / 0.64	1.46 / 0.91	1.21 / 1.25	0.78 / 2.6	0.49 / 4.1
	5/16	8.1	3.53 / 0.01	3.53 / 0.09	2.68 / 0.22	1.98 / 0.40	1.49 / 0.63	1.21 / 0.91	1.00 / 1.26	0.62 / 2.6	0.41 / 4.2
	1/4	6.6	2.39 / 0.01	2.39 / 0.07	2.11 / 0.22	1.56 / 0.40	1.22 / 0.63	0.98 / 0.92	0.83 / 1.26	0.49 / 2.6	0.33 / 4.1
3½ x 3½	5/8	13.5	11.2 / 0.03	5.56 / 0.12	3.66 / 0.27	2.70 / 0.48	2.11 / 0.76	1.71 / 1.09	1.42 / 1.46	0.86 / 3.1	0.56 / 4.8
	1/2	11.0	8.26 / 0.03	4.63 / 0.12	3.04 / 0.27	2.25 / 0.48	1.76 / 0.74	1.42 / 1.07	1.18 / 1.47	0.71 / 3.0	0.47 / 4.8
	3/8	8.4	4.96 / 0.02	3.61 / 0.12	2.38 / 0.26	1.75 / 0.47	1.37 / 0.74	1.11 / 1.07	0.92 / 1.48	0.54 / 3.0	0.36 / 4.7
	5/16	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
	1/4	5.7	2.36 / 0.01	2.39 / 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	5/8	11.4	7.96 / 0.04	3.96 / 0.14	2.59 / 0.32	1.91 / 0.57	1.48 / 0.89	1.19 / 1.29	0.98 / 1.76	0.57 / 3.6	0.36 / 5.7
	1/2	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	0.48 / 3.6	0.30 / 5.6
	3/8	7.1	4.80 / 0.03	2.59 / 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
	5/16	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
	1/4	4.8	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	0.15 / 5.6
	3/16	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 / 1.68	0.18 / 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.

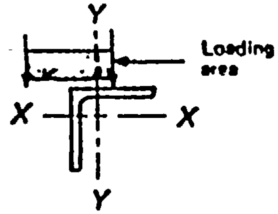
Safe Load Tables for Laterally Unsupported Angles



EQUAL ANGLES

NORMAL STRENGTH STEEL (36 ksi)

BASED ON AS CA1 1968



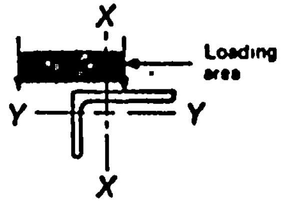
NOMINAL SIZE	THICKNESS t NOMINAL	WEIGHT PER FOOT	SAFE DISTRIBUTED LOADS (KIPS) / MAX DEFLECTION (in) — SPANS IN FEET								
in	in	lb	2	4	6	8	10	12	14	20	25
2½ x 2½	½	7.7	4.46 0.04	2.20 0.17	1.44 0.38	1.08 0.68	0.82 1.07	0.68 1.54	0.53 2.1	0.29 4.3	0.17 6.9
	⅜	5.9	3.64 0.04	1.78 0.17	1.18 0.38	0.84 0.67	0.66 1.06	0.52 1.52	0.42 2.1	0.23 4.3	0.13 6.8
	5/16	5.0	2.99 0.04	1.48 0.17	0.97 0.37	0.71 0.66	0.56 1.04	0.44 1.51	0.36 2.1	0.19 4.2	0.11 6.7
	¼	4.0	2.42 0.04	1.19 0.17	0.78 0.38	0.58 0.65	0.44 1.03	0.36 1.49	0.28 2.0	0.14 4.3	
	3/16	3.1	1.32 0.04	0.91 0.18	0.60 0.38	0.43 0.66	0.34 1.03	0.27 1.48	0.22 2.0	0.12 4.4	
2¼ x 2¼	⅜	5.2	2.81 0.05	1.39 0.19	0.91 0.42	0.68 0.75	0.51 1.17	0.41 1.70	0.33 2.3	0.18 4.8	
	5/16	4.4	2.38 0.05	1.18 0.19	0.77 0.42	0.58 0.74	0.43 1.16	0.35 1.69	0.28 2.3	0.14 4.8	
	¼	3.6	1.99 0.05	0.98 0.18	0.64 0.41	0.47 0.74	0.38 1.16	0.28 1.68	0.22 2.3	0.12 4.7	
	3/16	2.7	1.28 0.05	0.74 0.18	0.48 0.40	0.36 0.72	0.27 1.15	0.21 1.64	0.17 2.3		
2 x 2	⅜	4.7	2.17 0.05	1.07 0.21	0.70 0.46	0.51 0.86	0.39 1.33	0.31 1.93	0.25 2.6	0.13 5.5	
	5/16	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	
	¼	3.2	1.56 0.05	0.76 0.21	0.50 0.47	0.38 0.83	0.28 1.31	0.22 1.90	0.17 2.6		
	3/16	2.4	1.20 0.05	0.59 0.21	0.37 0.46	0.27 0.82	0.21 1.30	0.16 1.86	0.13 2.6		
	⅛	1.7	0.59 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	25
1¾ x 1¾	⅜	4.0	1.61 0.08	0.79 0.25	0.52 0.55	0.37 0.99	0.28 1.55	0.22 2.2	0.18 3.1		
	5/16	3.4	1.38 0.08	0.68 0.24	0.44 0.54	0.32 0.97	0.24 1.52	0.19 2.2	0.15 3.0		
	¼	2.7	1.16 0.08	0.57 0.24	0.37 0.54	0.27 0.96	0.21 1.51	0.16 2.2	0.12 3.0		
	3/16	2.1	0.91 0.08	0.44 0.24	0.29 0.53	0.21 0.94	0.16 1.49	0.12 2.2			
	⅛	1.5	0.59 0.08	0.30 0.21	0.19 0.52	0.14 0.94	0.11 1.50				
1½ x 1½	5/16	2.86	0.98 0.07	0.48 0.29	0.31 0.64	0.22 1.15	0.17 1.80	0.13 2.6	0.10 3.6		
	¼	2.35	0.83 0.07	0.41 0.28	0.26 0.63	0.19 1.13	0.14 1.80	0.11 2.6			
	3/16	1.8	0.65 0.07	0.32 0.28	0.21 0.63	0.15 1.12	0.11 1.75				
	⅛	1.2	0.44 0.07	0.22 0.27	0.14 0.62	0.10 1.10					
1¼ x 1¼	5/16	2.3	0.66 0.09	0.32 0.35	0.21 0.79	0.15 1.41	0.11 2.2				
	¼	1.9	0.56 0.09	0.27 0.34	0.18 0.77	0.12 1.38					
	3/16	1.48	0.43 0.08	0.21 0.33	0.14 0.76						
	⅛	1.0	0.31 0.08	0.15 0.33							
1 x 1	¼	1.5	0.34 0.11	0.16 0.44	0.10 0.99						
	3/16	1.15	0.27 0.11	0.13 0.43							
	⅛	0.8	0.19 0.10								

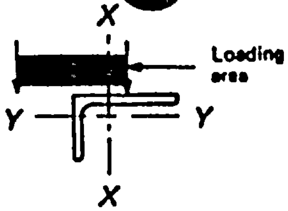
The tables are calculated for loads applied anywhere within half a leg length on either side of the shear centre—see fig. 1.

Safe Load Tables for Laterally Unsupported Angles
High-Strength Steel—50 ksi yield stress



UNEQUAL ANGLES

Force parallel to SHORT leg



NORMAL STRENGTH STEEL (36 ksi)

BASED ON AS CA1 1968

NOMINAL SIZE	THICKNESS / 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	3/4	23.6	14.6 0.02	9.05 0.10	6.06 0.22	4.30 0.39	3.42 0.61	2.77 0.88	2.28 1.20	1.36 2.4	0.87 3.8
	5/8	19.9	10.8 0.01	7.74 0.10	5.10 0.21	3.75 0.38	2.93 0.60	2.37 0.88	1.96 1.17	1.17 2.4	0.76 3.8
	1/2	16.1	7.21 0.01	6.41 0.10	4.22 0.21	3.11 0.38	2.43 0.59	1.98 0.88	1.62 1.17	0.97 2.4	0.63 3.8
	7/16	14.2	5.66 0.01	5.66 0.10	3.75 0.21	2.76 0.38	2.16 0.60	1.75 0.86	1.44 1.17	0.87 2.4	0.57 3.8
	3/8	12.3	4.28 0.01	4.28 0.10	3.27 0.21	2.41 0.38	1.89 0.60	1.53 0.86	1.26 1.17	0.76 2.4	0.50 3.8
6 x 3½	5/8	18.8	10.2 0.03	5.94 0.11	3.90 0.24	2.86 0.43	2.21 0.67	1.77 0.97	1.45 1.32	0.82 2.7	0.49 4.3
	1/2	15.3	6.74 0.02	4.93 0.11	3.23 0.26	2.37 0.44	1.84 0.69	1.48 0.99	1.21 1.36	0.69 2.8	0.42 4.3
	7/16	13.5	5.33 0.02	4.41 0.10	2.89 0.24	2.12 0.43	1.65 0.68	1.32 0.98	1.08 1.33	0.62 2.7	0.38 4.3
	3/8	11.8	3.97 0.02	3.82 0.10	2.51 0.24	1.84 0.47	1.43 0.68	1.15 0.96	0.94 1.30	0.54 2.7	0.33 4.3
	5/16	9.7	2.81 0.01	2.81 0.10	2.12 0.24	1.56 0.43	1.21 0.67	0.97 0.96	0.80 1.31	0.46 2.7	0.28 4.5
5 x 3½	5/8	16.7	10.6 0.03	5.76 0.11	3.78 0.25	2.78 0.45	2.16 0.70	1.74 1.01	1.43 1.36	0.83 2.8	0.52 4.4
	1/2	13.6	7.17 0.02	4.78 0.11	3.15 0.25	2.32 0.45	1.80 0.70	1.45 1.01	1.20 1.38	0.69 2.8	0.43 4.4
	3/8	10.3	4.27 0.02	3.71 0.11	2.44 0.25	1.79 0.44	1.40 0.67	1.13 1.00	0.92 1.34	0.54 2.8	0.34 4.5
	5/16	8.7	3.03 0.01	3.03 0.11	2.05 0.24	1.51 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.

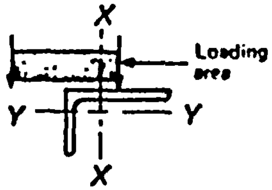
NOMINAL SIZE	THICKNESS / NOMINAL	WEIGHT PER FOOT	SAFE DISTRIBUTED LOADS (KIPS) / MAX DEFLECTION (in) SPANS IN FEET								
in	in	lb	2	4	6	8	10	12	14	20	26
5 x 3	1/2	12.7	6.53 0.03	3.80 0.12	2.29 0.28	1.67 0.50	1.29 0.78	1.03 1.13	0.84 1.56	0.46 3.2	0.26 5.1
	7/16	11.2	5.18 0.02	3.13 0.12	2.06 0.28	1.50 0.50	1.16 0.78	0.92 1.12	0.76 1.53	0.41 3.2	0.23 5.1
	3/8	9.7	3.92 0.02	2.75 0.12	1.80 0.29	1.32 0.50	1.02 0.77	0.81 1.12	0.66 1.53	0.36 3.2	0.20 5.1
	5/16	8.1	2.78 0.02	2.22 0.12	1.52 0.28	1.11 0.50	0.86 0.78	0.69 1.12	0.56 1.53	0.31 3.2	0.17 5.2
	1/4	6.6	1.86 0.01	1.80 0.12	1.28 0.27	0.92 0.49	0.71 0.77	0.57 1.12	0.46 1.54	0.25 3.2	0.14 5.4
4 x 3	5/8	13.6	8.50 0.03	4.21 0.13	2.76 0.31	2.02 0.56	1.57 0.88	1.26 1.23	1.02 1.66	0.57 3.4	0.33 5.4
	1/2	11.0	6.82 0.03	3.38 0.13	2.20 0.29	1.62 0.52	1.26 0.82	1.01 1.18	0.82 1.61	0.46 3.3	0.27 5.3
	3/8	8.4	4.20 0.02	2.62 0.13	1.73 0.29	1.27 0.51	0.98 0.81	0.79 1.16	0.64 1.59	0.37 3.3	0.22 5.3
	5/16	7.1	3.06 0.02	2.26 0.13	1.48 0.29	1.09 0.51	0.84 0.80	0.68 1.16	0.56 1.59	0.32 3.3	0.19 5.4
	1/4	5.8	3.76 0.03	1.86 0.13	1.22 0.29	0.90 0.52	0.70 0.81	0.56 1.18	0.46 1.60	0.26 3.4	0.16 5.6
3½ x 3	1/2	10.2	6.86 0.03	3.40 0.14	2.23 0.31	1.64 0.56	1.27 0.86	1.02 1.20	0.84 1.66	0.48 3.4	0.29 5.4
	3/8	7.8	5.33 0.03	2.64 0.13	1.74 0.30	1.27 0.54	0.99 0.84	0.80 1.21	0.66 1.64	0.38 3.4	0.23 5.4
	5/16	6.6	4.62 0.03	2.24 0.13	1.47 0.30	1.08 0.54	0.84 0.84	0.68 1.21	0.57 1.66	0.31 3.3	0.19 5.3
	1/4	5.3	3.70 0.03	1.83 0.13	1.20 0.30	0.89 0.53	0.67 0.84	0.55 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.

Safe Load Tables for Laterally Unsupported Angles

High-Strength Steel—52 ksi yield stress

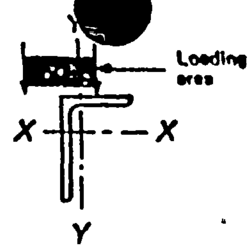
UNEQUAL ANGLES



Force parallel to LONG leg

NORMAL STRENGTH STEEL (36 ksi)

BASED ON AS CA1 1968



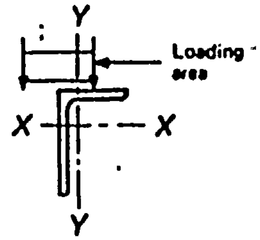
NOMINAL SIZE	THICKNESS NOMINAL	WEIGHT PER FOOT	SAFE DISTRIBUTED LOADS (KIPS) / MAX DEFLECTION (in) SPANS IN FEET								
in	in	lb	2	4	6	8	10	12	14	20	25
3½ x 2½	½	9.4	4.81 / 0.04	2.38 / 0.18	1.66 / 0.37	1.13 / 0.66	0.87 / 1.02	0.66 / 1.42	0.53 / 1.93	0.28 / 4.0	0.14 / 8.2
	⅜	7.1	3.71 / 0.04	1.84 / 0.18	1.20 / 0.36	0.88 / 0.63	0.67 / 0.99	0.54 / 1.42	0.43 / 1.94	0.23 / 4.0	0.12 / 8.3
	5/16	6.0	2.87 / 0.04	1.56 / 0.18	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 / 0.18	0.84 / 0.35	0.59 / 0.60	0.46 / 0.95	0.36 / 1.37	0.29 / 1.87	0.18 / 3.9	
	3/16	3.7	1.15 / 0.02	0.97 / 0.15	0.63 / 0.34	0.46 / 0.60	0.36 / 0.94	0.28 / 1.36	0.23 / 1.87	0.12 / 3.9	
3 x 2½	½	8.5	4.00 / 0.04	2.28 / 0.17	1.49 / 0.37	1.09 / 0.66	0.84 / 1.04	0.67 / 1.50	0.54 / 2.0	0.29 / 4.2	0.16 / 6.6
	⅜	6.5	3.61 / 0.04	1.79 / 0.18	1.17 / 0.38	0.85 / 0.65	0.66 / 1.01	0.53 / 1.44	0.43 / 2.0	0.23 / 4.1	0.13 / 6.5
	5/16	5.6	3.07 / 0.04	1.54 / 0.18	1.01 / 0.36	0.74 / 0.64	0.57 / 1.00	0.46 / 1.46	0.37 / 2.0	0.20 / 4.0	0.11 / 6.3
	¼	4.4	2.01 / 0.03	1.26 / 0.18	0.82 / 0.36	0.60 / 0.63	0.46 / 1.00	0.37 / 1.47	0.30 / 2.0	0.15 / 4.1	
	3/16	3.4	1.22 / 0.03	0.96 / 0.15	0.62 / 0.36	0.45 / 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 / 0.46	0.68 / 0.81	0.50 / 1.27	0.39 / 1.83	0.30 / 2.5	0.14 / 5.1	
	⅜	5.9	2.36 / 0.06	1.18 / 0.20	0.78 / 0.45	0.52 / 0.77	0.39 / 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	0.48 / 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 / 0.78	0.23 / 1.20	0.18 / 1.73	0.14 / 2.4		
2½ x 2	⅜	5.3	2.24 / 0.06	1.10 / 0.20	0.72 / 0.46	0.52 / 0.83	0.40 / 1.29	0.31 / 1.87	0.26 / 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 / 1.23	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 / 1.23	0.22 / 1.79	0.18 / 2.4		
	3/16	2.7	1.08 / 0.05	0.58 / 0.19	0.38 / 0.43	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.

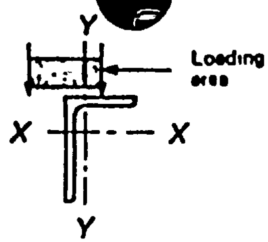
NOMINAL SIZE	THICKNESS NOMINAL	WEIGHT PER FOOT	SAFE DISTRIBUTED LOADS (KIPS) / MAX DEFLECTION (in) SPANS IN FEET								
in	in	lb	2	4	6	8	10	12	14	20	25
6 x 4	¾	23.6	23.0 / 0.01	19.2 / 0.07	12.7 / 0.16	9.23 / 0.29	7.30 / 0.44	6.00 / 0.64	5.06 / 0.88	3.39 / 1.81	2.43 / 2.8
	⅝	19.9	16.6 / 0.01	16.1 / 0.07	10.7 / 0.16	7.94 / 0.28	6.28 / 0.44	5.16 / 0.64	4.36 / 0.88	3.04 / 1.83	2.03 / 2.9
	½	16.1	11.0 / 0.01	11.0 / 0.06	8.88 / 0.16	6.46 / 0.28	5.11 / 0.44	4.20 / 0.64	3.54 / 0.88	2.23 / 1.81	1.64 / 2.9
	7/16	14.2	8.66 / 0.02	8.66 / 0.06	7.87 / 0.16	5.63 / 0.28	4.46 / 0.44	3.66 / 0.64	3.08 / 0.88	2.01 / 1.87	1.42 / 3.0
	⅜	12.3	6.53 / 0.01	6.53 / 0.04	6.46 / 0.16	4.80 / 0.28	3.80 / 0.44	3.12 / 0.64	2.63 / 0.88	1.72 / 1.82	1.20 / 3.0
6 x 3½	⅝	18.8	18.2 / 0.01	15.9 / 0.07	10.6 / 0.16	7.83 / 0.29	6.20 / 0.46	5.08 / 0.66	4.30 / 0.91	2.71 / 1.87	2.00 / 2.9
	½	15.3	11.9 / 0.01	11.9 / 0.07	8.40 / 0.18	6.26 / 0.29	4.96 / 0.45	4.07 / 0.66	3.43 / 0.91	2.24 / 1.90	1.68 / 3.1
	7/16	13.6	9.38 / 0.01	9.38 / 0.06	7.42 / 0.18	5.52 / 0.29	4.32 / 0.45	3.56 / 0.66	2.99 / 0.91	1.96 / 1.93	1.38 / 3.0
	⅜	11.6	6.96 / 0.01	6.96 / 0.06	6.31 / 0.18	4.70 / 0.29	3.71 / 0.45	3.06 / 0.67	2.57 / 0.92	1.63 / 1.92	1.18 / 3.2
	5/16	9.7	4.82 / 0.01	4.82 / 0.04	4.82 / 0.15	3.82 / 0.29	3.10 / 0.47	2.46 / 0.67	2.04 / 0.92	1.31 / 2.1	0.96 / 3.3
5 x 3½	⅝	16.7	15.5 / 0.01	11.2 / 0.09	7.38 / 0.19	5.46 / 0.34	4.32 / 0.54	3.54 / 0.78	2.97 / 1.07	1.91 / 2.2	1.32 / 3.3
	½	13.6	10.5 / 0.01	9.16 / 0.09	6.06 / 0.19	4.50 / 0.34	3.56 / 0.54	2.91 / 0.78	2.44 / 1.06	1.57 / 2.2	1.07 / 3.3
	⅜	10.3	6.13 / 0.01	6.13 / 0.07	4.61 / 0.19	3.34 / 0.34	2.64 / 0.53	2.16 / 0.77	1.81 / 1.05	1.17 / 2.3	0.79 / 3.3
	5/16	8.7	4.39 / 0.01	4.39 / 0.06	3.80 / 0.19	2.77 / 0.34	2.19 / 0.54	1.79 / 0.77	1.50 / 1.06	0.90 / 2.2	0.66 / 3.2

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
High-Strength Steel—52 ksi yield stress



UNEQUAL ANGLES



Force parallel to LONG leg

NORMAL STRENGTH STEEL (36 ksi)

BASED ON AS CA1 1968

Each cell: safe distributed load (kips) / max deflection (in)

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
5 x 3	1/2	12.7	11.8 / 0.01	8.78 / 0.08	5.81 / 0.19	4.32 / 0.34	3.41 / 0.54	2.79 / 0.79	2.36 / 1.08	1.61 / 2.3	1.08 / 3.7
	7/16	11.2	9.87 / 0.01	7.78 / 0.08	5.18 / 0.19	3.83 / 0.36	3.02 / 0.54	2.48 / 0.79	2.08 / 1.09	1.29 / 2.3	0.92 / 3.7
	3/8	9.7	8.70 / 0.01	6.78 / 0.08	4.47 / 0.19	3.32 / 0.36	2.62 / 0.56	2.18 / 0.81	1.77 / 1.09	1.12 / 2.4	0.77 / 3.8
	5/16	8.1	4.73 / 0.01	4.73 / 0.07	3.64 / 0.19	2.80 / 0.36	2.13 / 0.56	1.76 / 0.81	1.47 / 1.13	0.90 / 2.4	0.56 / 4.0
	1/4	6.6	3.20 / 0.01	3.20 / 0.08	2.93 / 0.19	2.18 / 0.36	1.88 / 0.56	1.37 / 0.82	1.15 / 1.16	0.67 / 2.6	0.44 / 4.2
4 x 3	5/8	13.6	14.7 / 0.03	7.31 / 0.11	4.83 / 0.24	3.57 / 0.44	2.81 / 0.68	2.29 / 0.99	1.91 / 1.36	1.19 / 2.8	0.83 / 4.4
	1/2	11.0	9.67 / 0.02	5.86 / 0.11	3.87 / 0.24	2.86 / 0.43	2.22 / 0.67	1.81 / 0.97	1.51 / 1.33	0.94 / 2.8	0.64 / 4.4
	3/8	8.4	6.70 / 0.02	4.44 / 0.11	2.93 / 0.23	2.17 / 0.42	1.71 / 0.67	1.39 / 0.97	1.18 / 1.33	0.70 / 2.8	0.47 / 4.3
	5/16	7.1	4.13 / 0.02	3.74 / 0.10	2.47 / 0.24	1.83 / 0.42	1.44 / 0.67	1.17 / 0.97	0.97 / 1.32	0.60 / 2.8	0.40 / 4.6
	1/4	5.8	2.78 / 0.01	2.78 / 0.09	2.00 / 0.24	1.49 / 0.43	1.14 / 0.66	0.93 / 0.97	0.77 / 1.36	0.48 / 3.0	0.29 / 4.6
3½ x 3	1/2	10.2	9.12 / 0.03	4.63 / 0.12	2.99 / 0.28	2.20 / 0.49	1.67 / 0.76	1.36 / 1.10	1.12 / 1.50	0.67 / 3.1	0.44 / 4.7
	3/8	7.8	5.26 / 0.03	3.38 / 0.12	2.23 / 0.26	1.64 / 0.47	1.29 / 0.74	1.04 / 1.06	0.87 / 1.45	0.63 / 3.0	0.33 / 4.9
	5/16	6.5	3.71 / 0.02	2.86 / 0.12	1.90 / 0.27	1.40 / 0.47	1.07 / 0.75	0.87 / 1.08	0.72 / 1.46	0.44 / 3.1	0.29 / 5.0
	1/4	5.3	2.49 / 0.02	2.39 / 0.12	1.57 / 0.27	1.17 / 0.48	0.88 / 0.74	0.72 / 1.08	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: 2	4	6	8	10	12	14	20	25
in	in	lb	2	4	6	8	10	12	14	20	25
3½ x 2½	1/2	9.4	8.68 / 0.03	4.30 / 0.12	2.84 / 0.28	2.10 / 0.49	1.64 / 0.77	1.34 / 1.12	1.11 / 1.53	0.68 / 3.2	0.43 / 6.0
	3/8	7.1	5.66 / 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 / 6.1
	5/16	6.0	4.00 / 0.02	2.85 / 0.12	1.88 / 0.28	1.36 / 0.48	1.06 / 0.77	0.86 / 1.12	0.71 / 1.53	0.42 / 3.3	0.28 / 6.3
	1/4	4.8	2.89 / 0.02	2.28 / 0.12	1.44 / 0.27	1.06 / 0.48	0.78 / 0.77	0.68 / 1.15	0.58 / 1.56	0.33 / 3.4	0.20 / 6.6
	3/16	3.7	1.63 / 0.01	1.63 / 0.12	1.11 / 0.27	0.81 / 0.48	0.63 / 0.77	0.50 / 1.17	0.41 / 1.61	0.23 / 3.5	0.13 / 6.8
3 x 2½	1/2	8.5	8.40 / 0.04	3.18 / 0.16	2.09 / 0.32	1.54 / 0.57	1.20 / 0.90	0.97 / 1.31	0.80 / 1.77	0.47 / 3.7	0.30 / 5.8
	3/8	6.5	5.03 / 0.04	2.50 / 0.14	1.64 / 0.32	1.21 / 0.57	0.94 / 0.90	0.78 / 1.29	0.61 / 1.75	0.36 / 3.8	0.23 / 6.0
	5/16	5.5	3.72 / 0.03	2.12 / 0.14	1.40 / 0.32	1.03 / 0.56	0.80 / 0.89	0.62 / 1.27	0.52 / 1.76	0.30 / 3.8	0.19 / 6.0
	1/4	4.4	2.43 / 0.03	1.66 / 0.14	1.10 / 0.31	0.81 / 0.56	0.63 / 0.86	0.51 / 1.28	0.42 / 1.80	0.24 / 3.7	0.15 / 6.0
	3/16	3.4	1.47 / 0.02	1.31 / 0.14	0.86 / 0.31	0.62 / 0.56	0.50 / 0.86	0.39 / 1.30	0.32 / 1.80	0.18 / 3.7	0.10 / 6.3
3 x 2	1/2	7.7	6.18 / 0.04	3.06 / 0.15	2.02 / 0.33	1.49 / 0.59	1.18 / 0.93	0.94 / 1.36	0.78 / 1.84	0.44 / 3.8	0.28 / 6.1
	3/8	5.9	4.80 / 0.04	2.36 / 0.14	1.57 / 0.33	1.16 / 0.58	0.90 / 0.92	0.73 / 1.34	0.60 / 1.83	0.34 / 3.9	0.22 / 6.3
	5/16	5.0	4.14 / 0.04	2.06 / 0.15	1.28 / 0.33	0.96 / 0.60	0.66 / 1.00	0.56 / 1.36	0.49 / 1.84	0.29 / 4.0	0.18 / 6.5
	1/4	4.0	2.70 / 0.03	1.63 / 0.14	1.04 / 0.32	0.79 / 0.59	0.60 / 0.92	0.48 / 1.38	0.38 / 1.87	0.22 / 4.1	0.13 / 6.6
	3/16	3.1	1.64 / 0.04	1.21 / 0.14	0.80 / 0.32	0.59 / 0.59	0.46 / 0.96	0.38 / 1.38	0.30 / 1.96	0.18 / 4.3	
2½ x 2	3/8	5.2	3.31 / 0.04	1.64 / 0.17	1.08 / 0.38	0.79 / 0.69	0.61 / 1.08	0.49 / 1.57	0.40 / 2.2	0.20 / 4.3	0.11 / 7.2
	5/16	4.4	2.82 / 0.04	1.40 / 0.17	0.88 / 0.38	0.67 / 0.68	0.52 / 1.08	0.42 / 1.56	0.33 / 2.2	0.17 / 4.4	0.10 / 7.1
	1/4	3.6	2.34 / 0.04	1.16 / 0.17	0.78 / 0.38	0.63 / 0.68	0.43 / 1.10	0.36 / 1.60	0.28 / 2.2	0.15 / 4.4	
	3/16	2.7	1.37 / 0.03	0.84 / 0.17	0.66 / 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.

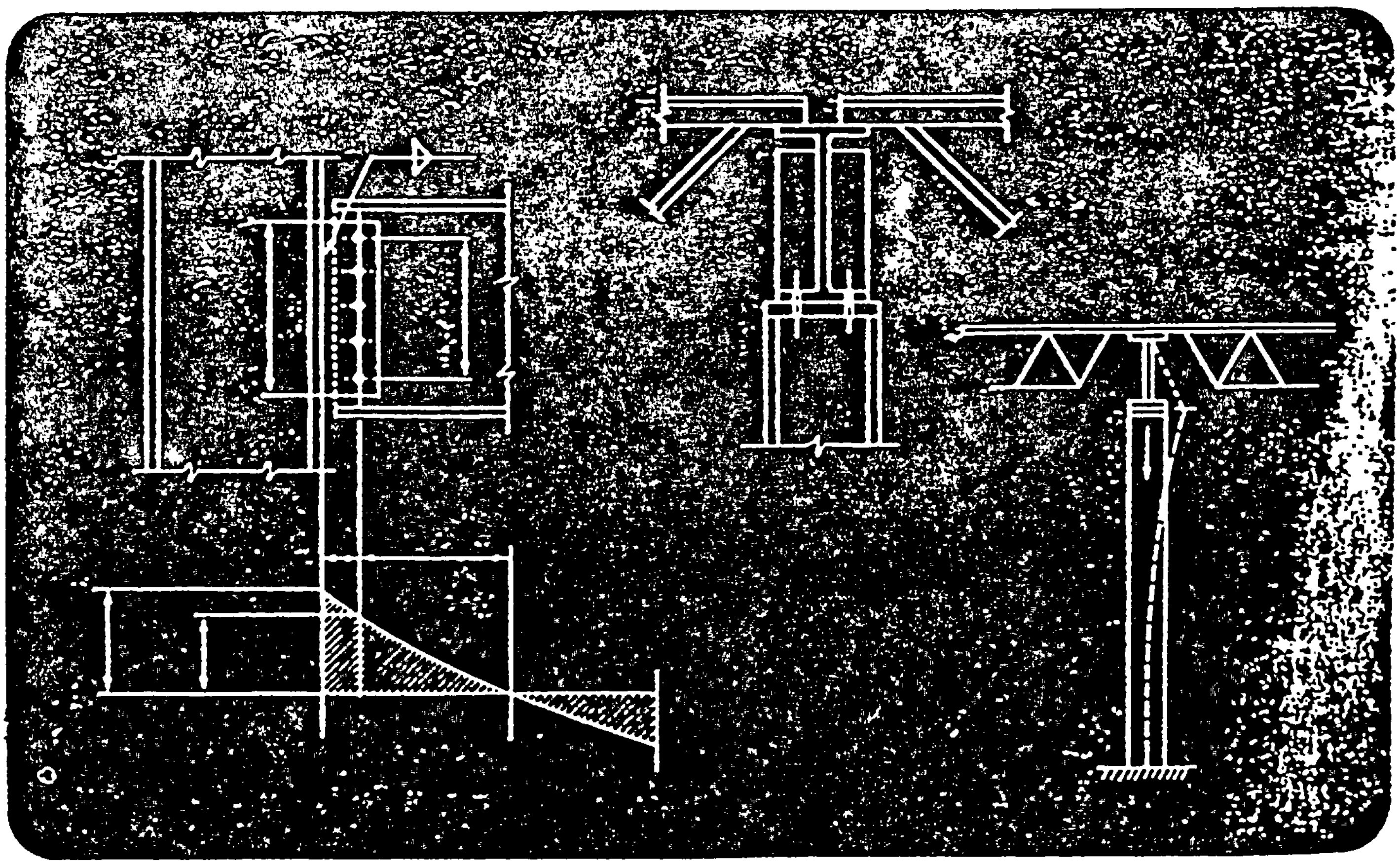
Safe Load Tables for Laterally Unsupported Angles

ATTACHMENT 7(b)-4

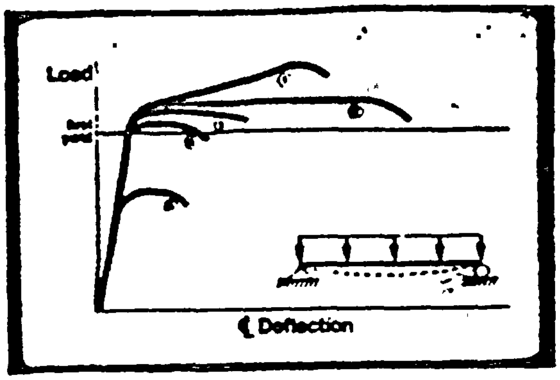
2

BENDING MEMBERS

STEEL DESIGN *CURRENT PRACTICE*



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.

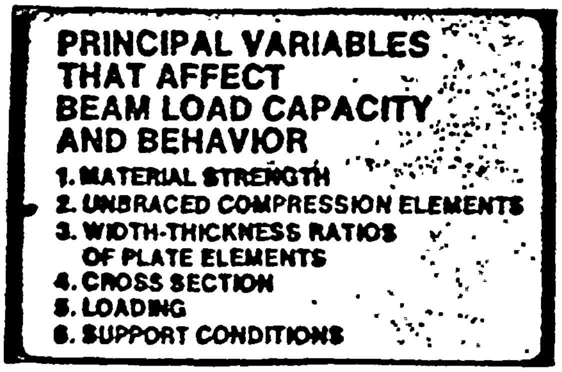
* The various allowable stresses permitted in the latest AISC Spec. are related to the behavior depicted in the 5 curves.

* 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.

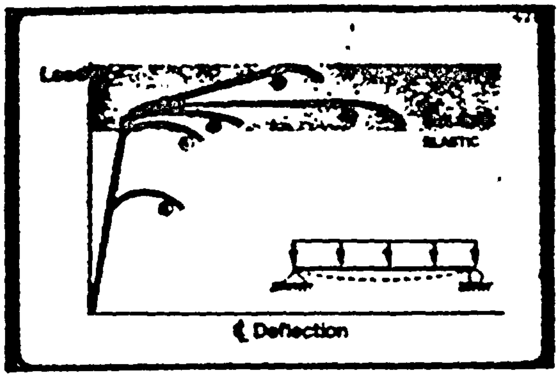
* The proper design is the economical one, not the lightest one.

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.

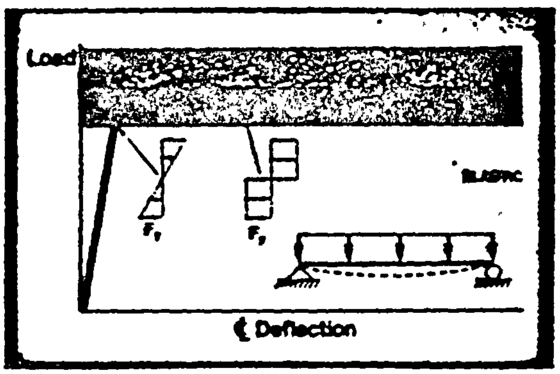
* Curves 1 and 2 will generally provide the lightest beams but sometimes the fabrication and detail cost is increased because of the added bracing, stiffeners, etc.

* 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



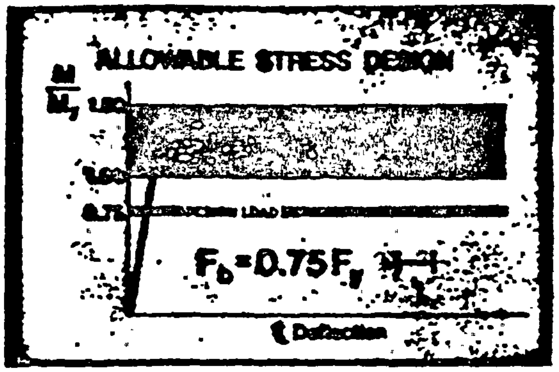
* Curves 1 and 2 treated together because the design provisions are basically the same.

* 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. 2-13



* 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.

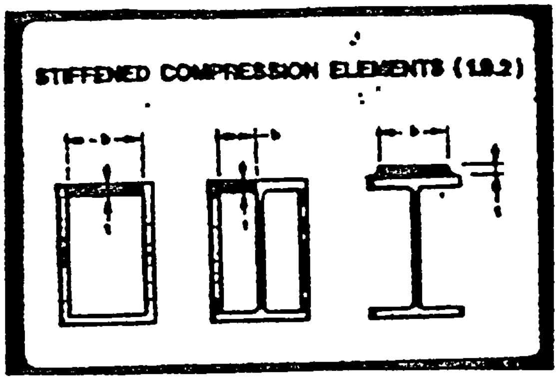
  2. Thinner round sections fail by excessive ovalization of the cross section. This is a type of inelastic instability problem in which the decrease in moment capacity caused by the reduction in the section modulus due to flattening occurs more rapidly than the increase in moment.

  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

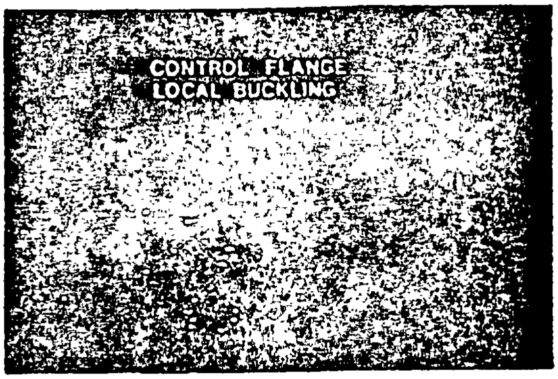
Slide No. 2-18



* 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.

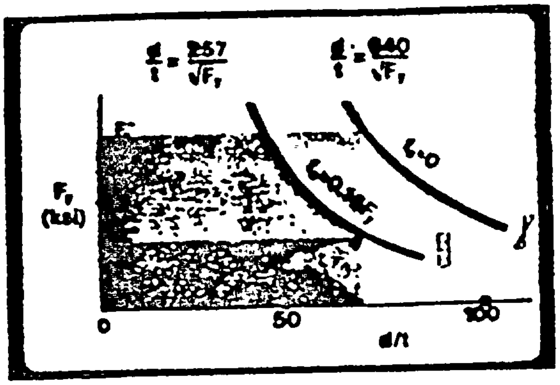
* The differences in ASD and PD requirements is that PD may require large rotation capacity - thus local buckling more critical.

* 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. 2-23



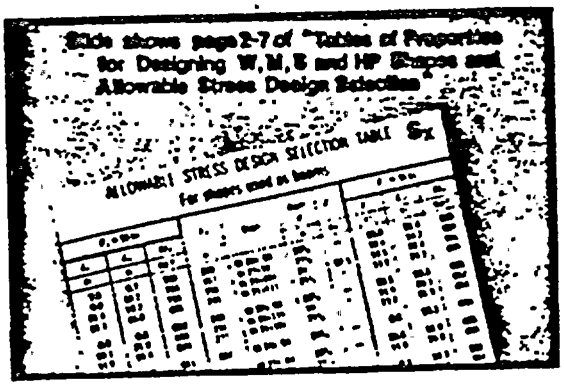
* If no axial load is present $F_y \le F''_y$, the web is compact.

* 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.

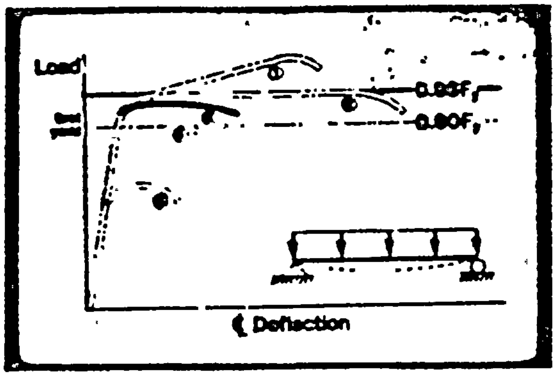
Slide No. 2-25

* 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.

lide No. 2-29



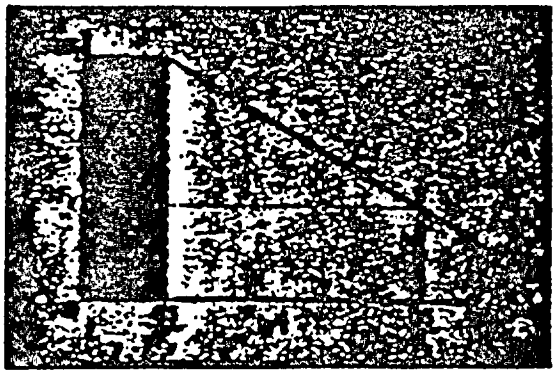
* Behavior illustrated by curve (3) should be expected if lateral buckling is controlled but flange or web slenderness ratios exceed compactness limits.

* 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.

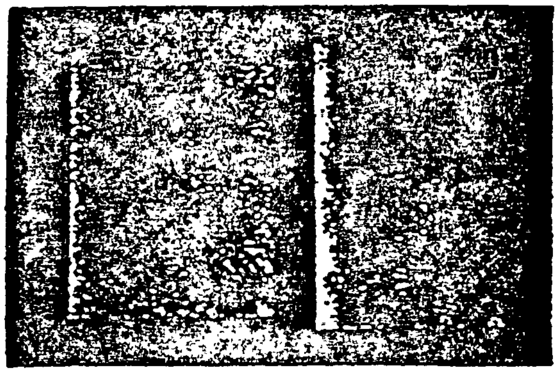
* $F_b = 0.66\ F_y$ for b/t up to $65/\sqrt{F_y}$.

* 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. 2-33



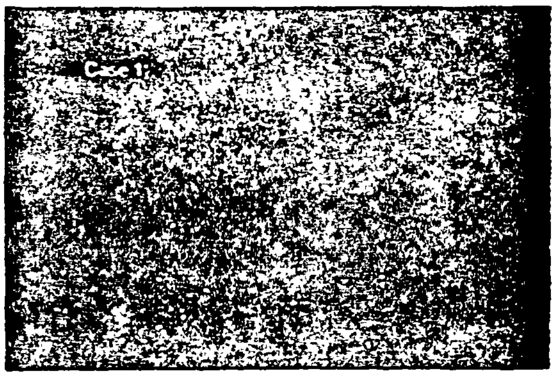
* The following slides will show how practical angle sections are usually governed by stress ($F_b = 0.6F_y$) or be deflection limitations rather than buckling.

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

* Loading is as shown with M_s 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:

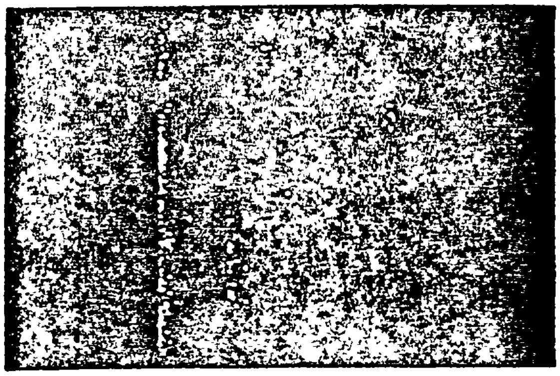
* The stress at any point in the section is
  $$\frac{3M}{b^3 t}\,[\,(V + 4U)\,\phi_1 + V - 4U]$$

  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. 2-38



* 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

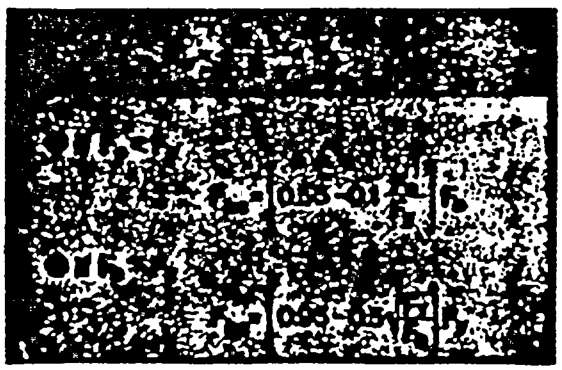
	Case	B/t Range for $F_b = 0.66\,F_y$
F=36 ksi	6	0<L/t< 990
	11	0<L/t< 850
	16	0<L/t< 690

* 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.

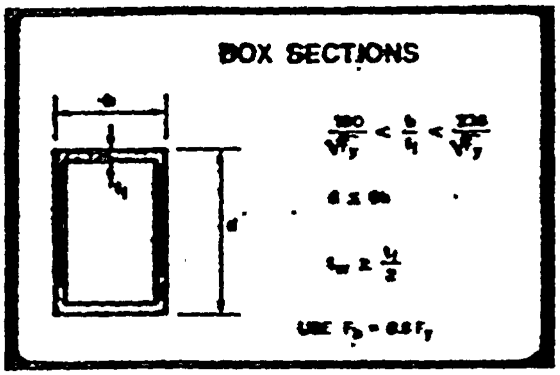
Slide No. 2-40



* 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. 2-44

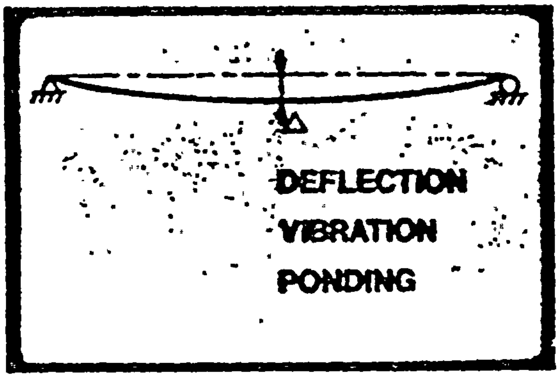


* If box sections do not meet compactness requirements use $F_b = 0.6\ F_y$.

* 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:

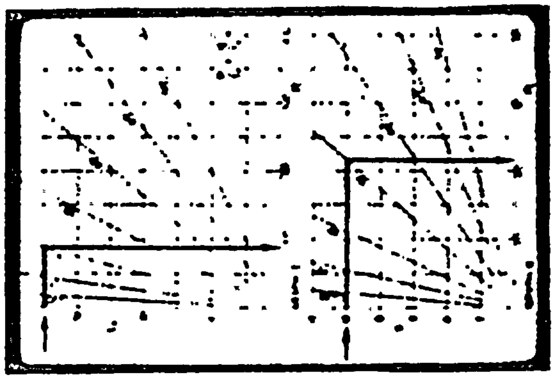
  Fully stressed floor beams and girders; F_y depth not less than $F_y/800$ times the span.

  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.

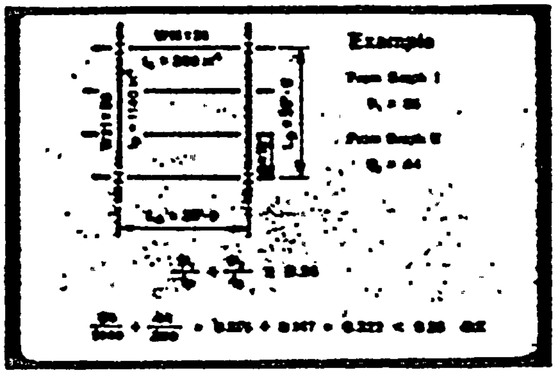
* Large open floor areas free of partitions or other sources of damping may be susceptible to transient vibration due to pedestrian traffic. While there are some design methods available to check a floor system for vibration susceptibility they necessarily involve trying to evaluate the difficult problem 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. 2-49



* 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.

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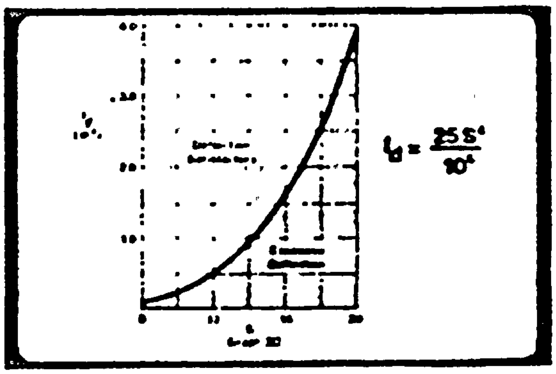


* Illustrates the use of graph III and the check for steel deck.

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_o = 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_a = 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_a = Section modulus about same axis as M_a.
- 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.

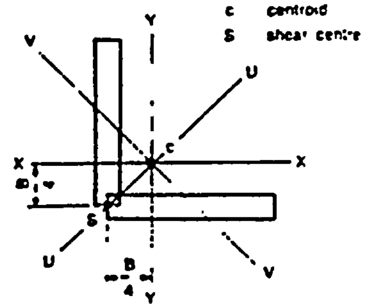


Fig 1 (a). Orientation of axes and locations of centroid and shear centre.

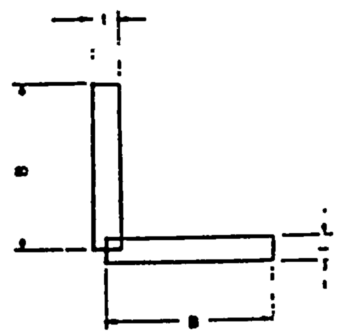
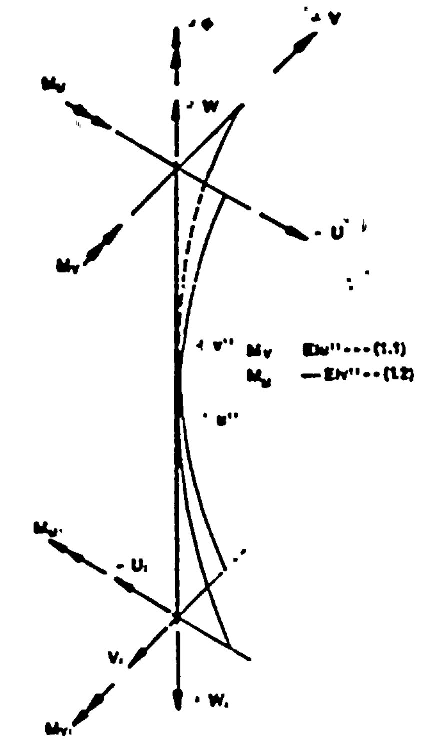
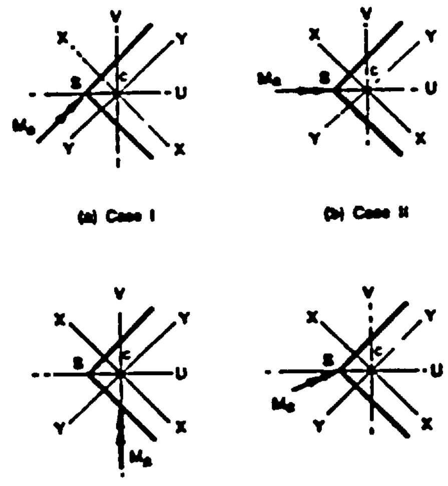


Fig.1 (b). Simplified angle section dimensions



Note
Axes are drawn with W or W₁ 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

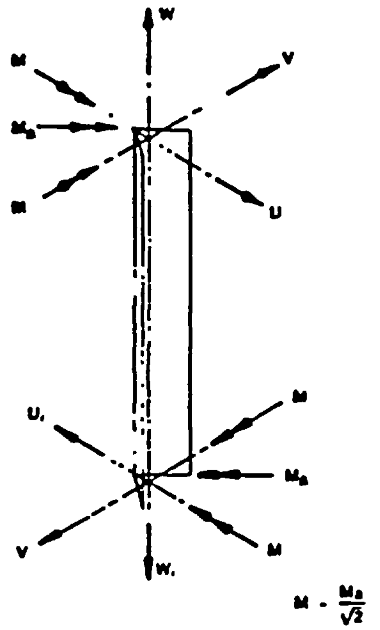


Fig.4 Loading for case I

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

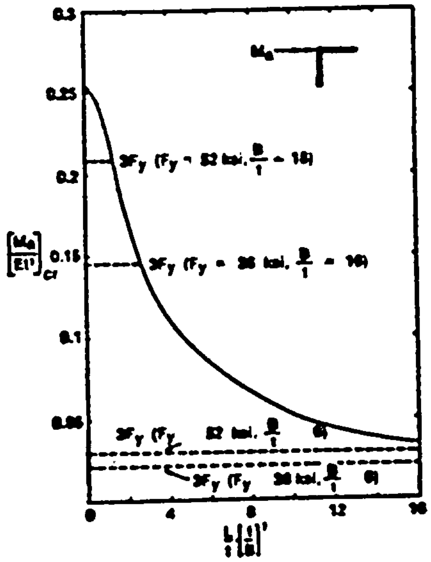


Fig.5 (a). Critical buckling curve for case I

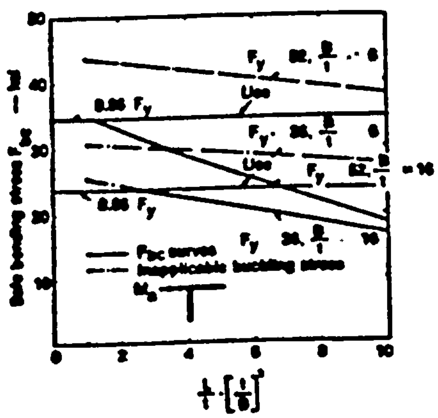


Fig.5 (b). Graph for determination of F_{bc} for case I

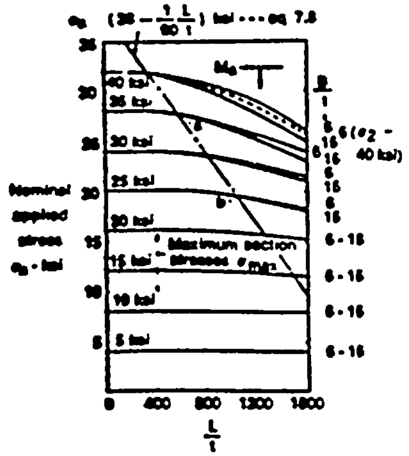


Fig 6. Graph for determination of actual max. section stress for case I

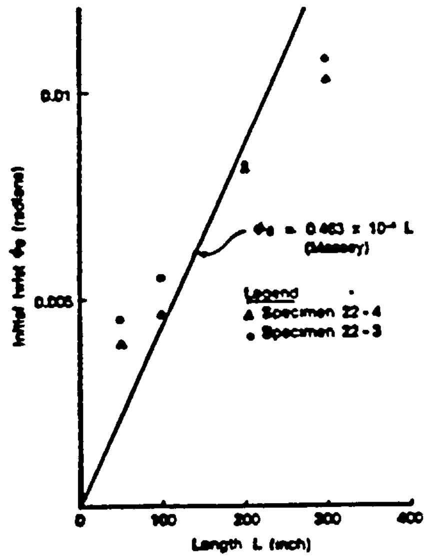


Fig.7. Initial twist ϕ_o test results for angle sections compared with Massey's relationship

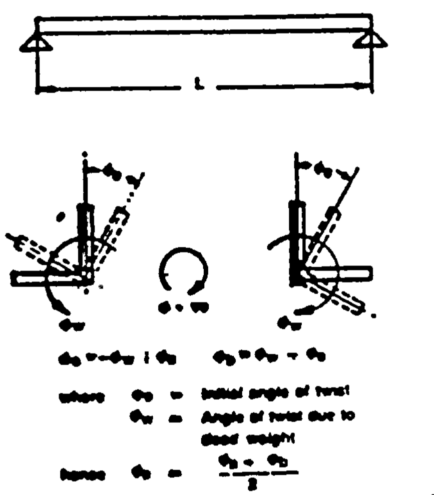


Fig.8 Method for determination of initial angle of twist

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

Case IV:
The design is satisfactory if:

$$\frac{\sigma_x}{F_x} + \frac{\sigma_y}{F_y} < 1$$

where F_x and F_y 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: $\sigma_c < 38 - \frac{1}{E0} \cdot \frac{L}{t}$ (ii) Critical Buckling Solution: See Table below.	$\sigma_{max} = \frac{2 \cdot 12M}{B \cdot t}(3 - \phi_1)$ (Fig.6) Use $F_o \rightarrow F_{bc}$ 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		$\frac{\sigma_x}{F_x} + \frac{\sigma_y}{F_y} < 1$

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.

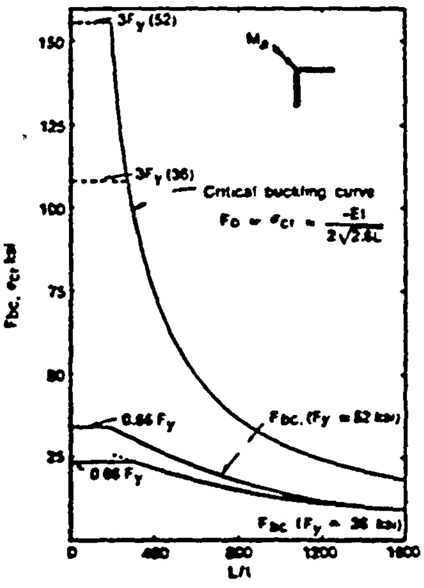

Fig.9. Critical buckling curve and curves for safe bending stress case II

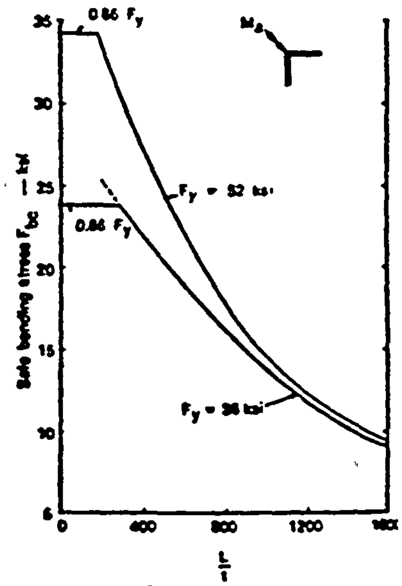

Fig.10 Graph for determination of F_{bc} for case II

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 safe 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.

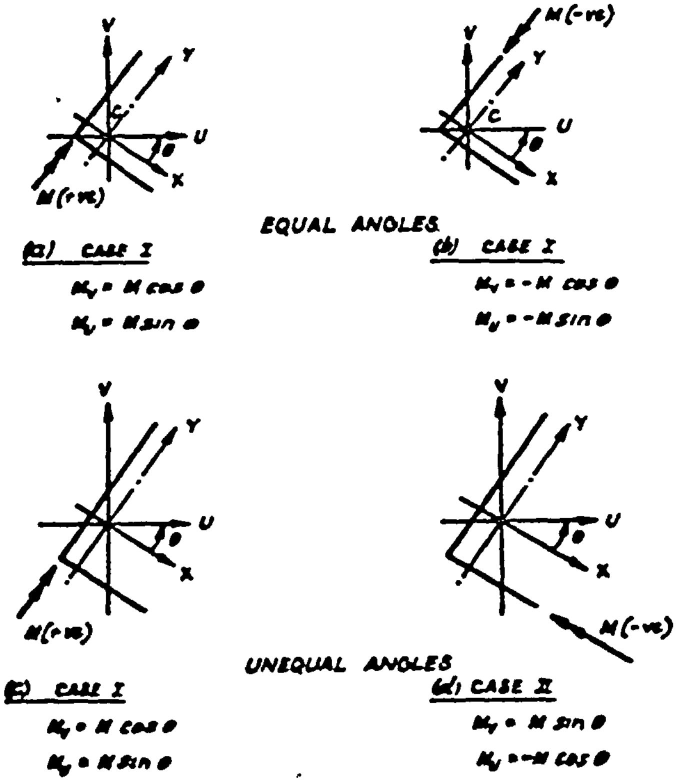

Fig. 3.—Loading Cases.

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 member 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.

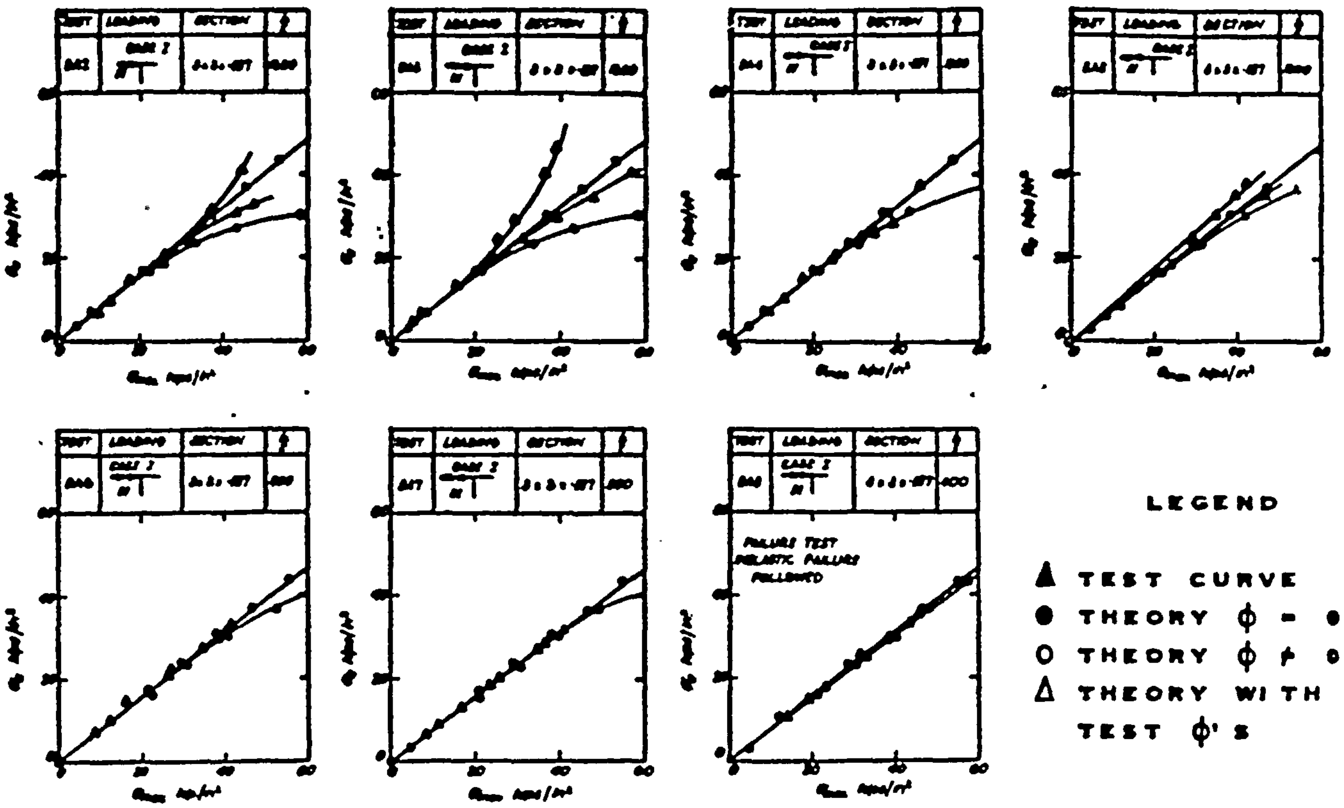

Fig. 6.—Relationship between Nominal Applied Stress σ_a and Maximum Stress σ_{max} for Tests on Equal Angle Sections.

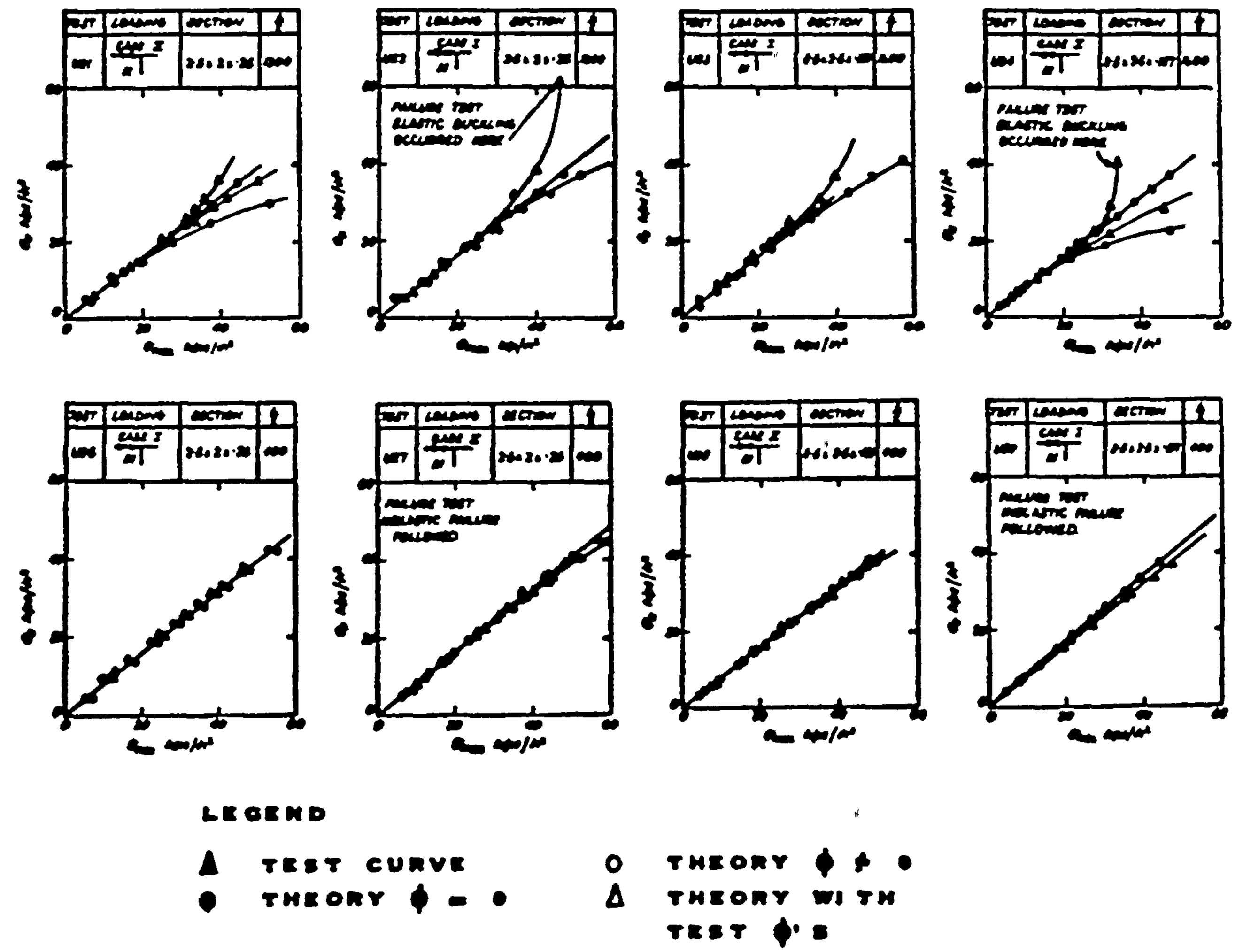

Fig. 7.—Relationship between Nominal Applied Stress σ_a and Maximum Stress σ_{max} for Tests on Unequal Angle Sections.

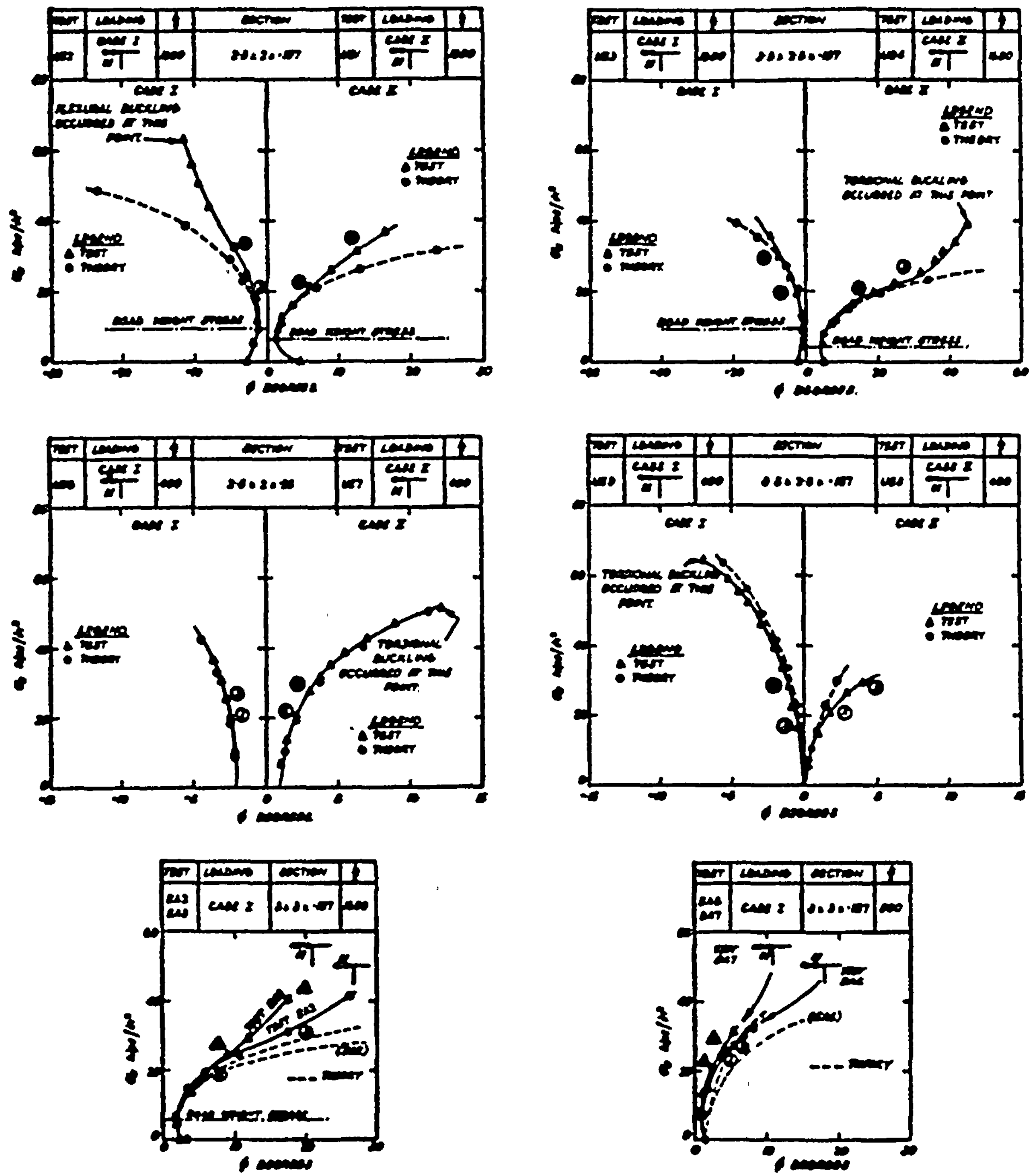

Fig. 9.—Relationship between Nominal Applied Stress σ_a and Maximum Angle of Twist ϕ.

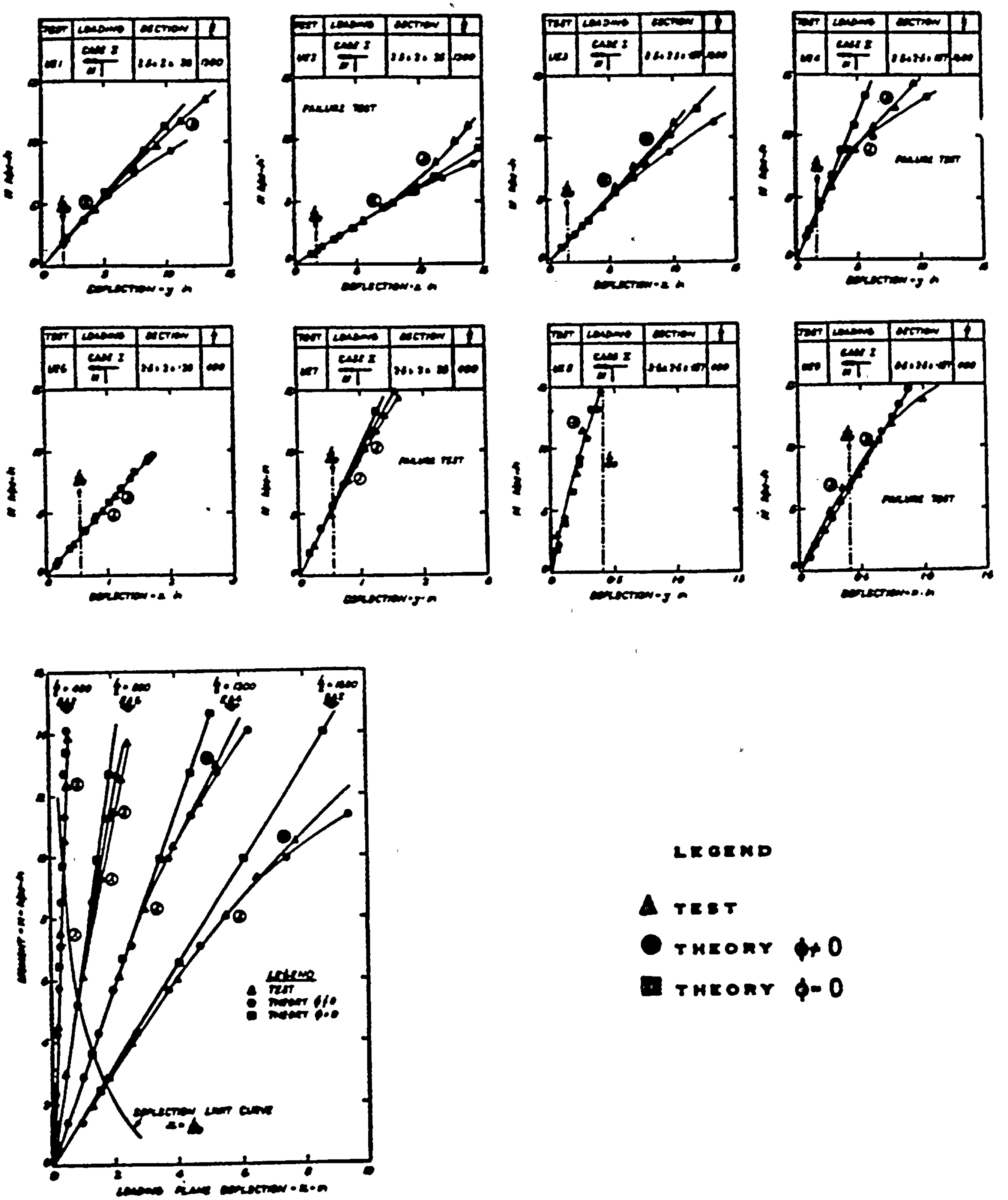

Fig. 11.—Relationship between Applied Moment M and Maximum Loading Plane Deflections.