IMPELL CORPORATION

PROJECT INSTRUCTION

TITLE: DESIGN VERIFICATION OF BASE PLATES, BASE ANGLE AND EMBEDMENT PLATE

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1.0 PURPOSE

This instruction provides the criteria and methodology for qualification of cable tray support anchorages. The methodology for analysis and qualification of base plates, base angles and embedded plates is presented.

2.0 SCOPE

The anchorages used to provide structural support to cable tray hangers located in CPSES Unit 1 are within the scope of this instruction. A cable tray support is typically fillet welded to a structural angle and/or plate which in turn is fastened to the supporting concrete structure via embedded bolts and/or inserts with bolts. Alternatively, the cable tray support can be welded directly to an embedded plate. This document covers the qualification of the above anchorage system. In particular, the following items are addressed: (1) Embedded bolts and/or inserts and bolts; (2) Structural steel angles, embedded plates and baseplates; and, (3) Fillet welds relevant to anchorage system.

3.0 REFERENCES DOCUMENTS

3.1 General

	Title	Revision	Date
3.1.1	Seismic Design Criteria for Cable Tray Hangers for CPSES Unit No. 1 (Ebasco)	1	12/20/85
3.1.2	General Instructions for Cable Tray Hanger Analysis for CPSES Units 1 and 2 (Ebasco)	2	12/20/85
3.1.3	Structural Embedments, Specification No. 2323-SS-30 (Gibbs and Hill)	1	02/10/84
3.1.4	CPSES Field Verification Method, TNE-FVM-CS-001 (TUGCo)	4	03/20/86
3.1.5	Not used		

3.1.6	Dynamic Analysis of Cable Tray Systems, PI-02 (Impell)	3	05/21/86
3.1.7	Qualification of Cable Tray Supports PI-03 (Impell)	2	05/20/86
3.1.8	Procedure for Obtaining Site Information PI-04 (Impell)	2	02/07/86
3.1.9	Finite Element Analysis Computer Program, Base Plate II (Control Data Corporation)	2.5	05/28/82
3.1.10	Base Angle and Base Plate Interaction Diagram Generation, Calculation No. M-15 (Impell)	2	06/02/86
3.1.11	Engineering Calculation Checking Procedure, QAP 3.6 (Impell)	4	07/15/85
3.1.12	Prying Action Factors for Two Bolt Base Plate, Calculation No. M-25 (Impell)	2	06/02/86

3.2 Codes of Record

3.2.1	American Institute of Steel Construction - Manual of Steel Construction, Supplements No. 1, 2, and 3 inclusive (AISC)	7th Edition	-
3.2.2	American Welding Society - Structural Welding Code - AWS D1.1 - 75 (AWS)	-	1975

4.0 METHODOLOGY

4.1 General

The main elements of the cable tray support anchorage are: (1) Embedded bolts and/or inserts and bolts, (2) Structural steel angles, embedded plates, and baseplates; and, (3) Fillet welds. The verification process for each of the above is discussed in this section.

Attachment I contains the general limitations of the analysis methodology. Consult with your lead engineer for conditions outside the general limitations.

4.1.1 Objective

The objective is to present a verification procedure that ensures the support anchorage meets all acceptance criteria as well as being consistent and efficient in terms of calculation method and quality assurance criteria. The procedure is presented in the following.

4.1.2 Verification Methodology

The flow chart shown in Figure F.1 illustrates the process. An anchorage connection is first classified as either: (1) common configuration that appears often (Fig. F.2); or (2) uncommon type that is usually infrequent. Each type is handled differently. Since it is anticipated that most anchorage connections can be classified as common, special consideration is given to procedures (or steps) that expedite the verification process. The objective is to qualify common anchorages. Should a common anchorage fail a simplified qualification procedure, a more refined process is then pursued. An overview of this methology for the common type anchorages is presented below.

The common anchorages are: (1) Structural angle having one bolt; (2) Angle having two bolts; (3) Angle having more than two bolts with one or more attachments; (4) Base plate having two bolts; (5) Base plate having four bolts; and, (6) Embedded plate. The first verification process for types 1 and 2 involves the use of interaction diagrams (Fig. F.1, Step 1). Anchorage qualification is performed by locating the key support reactions on the appropriate interaction diagram. If the reactions correspond to the "pass" region, the anchorage is qualified. If not, the anchorage cannot be qualified by using the interaction diagrams. 2

A hand calculation solution using existing project criteria (Ref. 3.1.2) is the next procedure (Fig. F.1, Step 2). This step encompasses all the common anchorages. The actual anchorage geometry and reaction forces for the particular anchorage in question are used thereby eliminating the conservatism inherent to the interaction diagrams.

Should the equations yield interaction ratios greater than unity, the anchorage cannot be qualified by standard hand calculations. However, the standard equations (Ref. 3.1.2) may have sufficient conservatism that the anchorage may still be qualified using the more refined approach given below.

For all types of common anchorages that cannot be qualified by the simplified procedures, the most refined verification is to perform a finite element analysis (Fig. F.1, Step 3) using the particular anchorage geometry and reactions. The BASEPLATE II (Ref. 3.1.9) computer code shall be used. Should the anchorage fail to pass this procedure, corrective design changes are required.

It should be noted that support modification may be indicated upon failure in any of the above procedures. For example, when using the interaction diagrams, the reactions may graph excessively in the failure zone. For these cases, it may be apparent that more refined procedures will not qualify the anchorage, and therefore they should not be pursued.

The uncommon anchorage configurations are to be verified using Baseplate II finite element analysis.

4.1.3 Source of Input Forces

In general, the load combinations (Ref. 3.1.1) containing the SSE condition yields forces that are larger than the load combinations containing the OBE condition. However, the SSE condition allowable values are greater than those for the OBE condition. Thus, it is not certain as to which case will govern for any anchorage, and both OBE and SSE loading combinations must be considered. An engineer may conservatively envelop all OBE and SSE reactions, and use these reactions along with the OBE allowables to qualify the anchorage. The necessary anchorage reactions are found in the SUPERPIPE results. (2)

Note forces and moments are considered absolute in value. There is to be no reduction in magnitude for the algebraic summation of forces.

4.1.4 Qualification Work flow

Support anchorage verification may commence as soon as a final support anchorage loads are available. The cable tray analysis yields the relevant forces/moments. The anchorage geometries are defined by the support drawings. The verification procedures are outlined herein. This constitutes the data necessary for the qualification procedure.

4.2 Interaction Diagrams

Step 1 (Fig. F.1) verification of common support anchorages involves the use of interaction diagrams. These are graphs having pass (or qualify) and fail regions as a function of the major support reactions. They are generated for various common support anchorage geometries.

4.2.1 Basis of Curves

Families of interaction diagrams for two types of common anchorages are generated using the existing criteria (see Section 4.3 below). These are the common angle anchorage having one or two bolts. The standard equations (Ref. 3.1.2) address the bolt/insert shear and tension forces. These equations do not address evaluation of base angle/plate stresses. In order to qualify an anchorage using these existing equations an additional calculation evaluating these stresses is performed.

The interaction diagrams are used to qualify an entire common anchorage. This includes the load carrying capacity of the bolts/inserts and the base angle/plate. It is shown (Ref. 3.1.10) that the stresses in the base angles/plates of common anchorages do not control qualification of the anchorages. Rather the capacity of the bolts/inserts are the forces limiting the qualification of a common anchorage. The common anchorages that meet the limiting parameters of a particular interaction diagram do <u>not</u> require additional calculation evaluating stress levels of the base angle/plate. The interaction diagram provides this evaluation.

4.2.2 Use of Curves

Given the support geometry and support forces/moments, qualification can be performed by entry into the appropriate interaction diagram. If the reactions plot in the pass region, the anchorage is qualified. If not, the anchorage cannot be qualified using interaction diagrams. Detailed instructions regarding the proper use and limitations of the interaction diagrams are presented in Attachment A. The supporting calculations are found in Ref. 3.1.10.

To minimize the number of interaction diagrams, it is necessary to set various parameters regarding the anchorage geometry and relation of several anchorage forces and/or moments. It is possible that an actual anchorage will have a geometry and/or reaction relationship that does not exactly match the generic diagrams. For these cases, the anchorage is checked by use of the closest conservative interaction diagram. Attachment A indicates how to select a conservative diagram.

4.3 Solution by Equations

Step 2, (Fig. F.1) verification is based on procedure as recommended by existing criteria (Ref. 3.1.2). Angles having one, two, or more than two bolts, and baseplates having two or four bolts are addressed below. 2

4.3.1 Basis of Equations

The procedure involves the determination of bolt/insert tension and shear forces via equations based on statics, prior empirical studies (Ref. 3.1.1) and additional prying action factors for the two bolt plates (Ref. 3.1.12). The bolt forces in conjunction with the bolt allowables are used in an interaction equation. The value of the interaction ratio determines whether the anchorage qualifies. Note that angle/plate stresses are not directly calculated. Fundamental methodologies for evaluation of plate/angle stresses are included as Attachment F. Evaluation of stress is required for each anchorage verified using these equations.

4.3.2 Use of Equations

The following provides for five common type anchorage connections. Each equation is described in the following:

2

4.3.2.1 Base Angle Having Two (2) Bolts

Bolt Tension T:

$$T = a_1 \times \left[\frac{Mx}{2\ H}\right] + a_2 \times \left[\frac{M'z}{dx}\right] + a_3 \times \left[\frac{C\ Fy}{2\ (C-G)}\right]$$

2

Bolt Shear S:

$$S = \left[\left[\frac{Fx}{2}\right]^2 + \left[\frac{Fz}{2} + \frac{M'y}{dx}\right]^2\right]^{1/2}$$

where,

F_x, F_y, F_z = Reaction forces in kips

M_x, M_y, M_z = Reaction moments in kip-inches

$M'_y = M_y + F_xG + F_zE_x$

$M'_z = M_z + F_yE_x$

a_1, a_2, a_3 = Prying action coefficients (see Table T.1)

C, G, E_x, d_x: See Fig. F.3a

H = minimum (C - G, G)

2

4.3.2.2 Base Angle Having One (1) Bolt

Bolt Tension T:

$$T = a_1 \times \left[\frac{Mx}{H}\right] + a_2 \times \left[\frac{M'z}{e'}\right] + a_3 \times \left[\frac{C\ Fy}{(C-G)}\right]$$

2

Bolt Shear S:

$$S = \left[Fx^2 + Fz^2\right]^{1/2}$$

where,

F_x, F_y, F_z = Reaction forces in kips

M_x, M_z = Reaction moments in kip-inches

$M'_z = M_z + F_y E_x$

a_1, a_2, a_3 = Prying action coefficients (see Table T.2)

e' = min[e1, e2] (See Fig. F.3b) $\leqslant$ 4.5 in.

C, G, E_x, L: See Fig. F.3b

H = (C - G, G)

2

4.3.2.3 Base Angle Having More than Two (2) Bolts and One or More Attachments

Maximum Bolt Tension T_{max}:

$$T_{max} = T_{FY} + T_{mx} + T_{mz}$$

$$T_{FY} = \max\left[\left[a_3 \times \frac{C \times (F_{yi} + F_{y(i+1)})}{2(C - G_{max})}\right] + a_2 \times \left[\frac{F_{yi} E_{xi}}{d_{xi}} + \frac{F_{y(i+1)} \times E_{x(i+1)}}{d_{x(i+1)}}\right]\right]_{\substack{i = 1 \\ \text{to} \\ i = (m-1)}}$$

$$T_{mx} = a_1 \times \max\left[\frac{M_{xi}}{2(H)} + \frac{M_{x(i+1)}}{2(H)}\right]_{\substack{i = 1 \\ \text{to} \\ i = (m-1)}}$$

$$T_{mz} = a_2 \times \sum_{i=1}^{m} \frac{M_{zi}}{d_{xi}}$$

Maximum Bolt Shear S_{max}:

$$S_{max} = S_{Fx} + S_{Fz} + S_{my}$$

$$S_{Fx} = \sum_{i=1}^{m} \frac{F_{xi}}{n} + \frac{\sum_{i=1}^{m} F_{xi} \times G_{max}}{\sum_{i=1}^{m} d_{xi}}$$

$$S_{my} = \frac{\sum_{i=1}^{m} M_{yi}}{\sum_{i=1}^{m} d_{xi}}$$

$$S_{Fz} = \max \left[\frac{F_{zi}}{2} + \frac{F_{zi} \times E_{xi}}{d_{xi}} + \frac{F_{z(i+1)}}{2} + \frac{F_{z(i+1)} \times E_{x(i+1)}}{d_{x(i+1)}} \right]_{i = 1 \text{ to } (m-1)}$$

where,

F_{xi}, F_{yi}, F_{zi} = Reaction forces in kips (See Fig. F.5)

M_{xi}, M_{yi}, M_{zi} = Reaction moments in kips-inches (See Fig. F.5)

a_1, a_2, a_3 = Prying action coefficients (see Table T.1)
(Same as two bolt angles)

C, G_{max}, E_{xi}, d_{xi} (See Fig. F.5)

H = minimum (C - G, G)

Simplified maximum shear and tension loads, for a three bolt angle with two attachments.

$$S_{max} = \left[(F_{x1} + F_{x2}) \left[\frac{1}{3} + \frac{G_{max}}{d_{x1} + d_{x2}} \right] + \frac{F_{z1} + F_{z2}}{2} + \frac{F_{z1} \times E_{x1}}{d_{x1}} + \frac{F_{z2} \times E_{x2}}{d_{x2}} + \frac{M_{y1} + M_{y2}}{d_{x1} + d_{x2}} \right]$$

$$T_{max} = \left[a_3 \times \left[\frac{C \times (F_{y1} + F_{y2})}{2(C - G_{max})} \right] + a_2 \times \left[\frac{F_{y1} \times E_{x1}}{d_{x1}} + \frac{F_{y2} \times E_{x2}}{d_{x2}} \right] + a_1 \times \left[\frac{M_{x1} + M_{x2}}{2H} \right] + a_2 \times \left[\frac{M_{z1}}{d_{x1}} + \frac{M_{z2}}{d_{x2}} \right] \right]$$

4.3.2.4 Base Plate Having Four (4) Anchor Bolts (2)

Bolt Tension T:

$$T = a_1 \times \left[\frac{M'_x}{2dz}\right] + a_2 \times \left[\frac{M'_z}{2dx}\right] + a_3 \times \left[\frac{F_y}{4}\right]$$

Bolt Shear S:

$$S = \left[\left[\frac{Fx}{4} + \frac{M'_y \, dz}{2\,(dx^2 + dz^2)}\right]^2 + \left[\frac{Fy}{4} + \frac{M'_y \, dx}{2\,(dx^2 + dz^2)}\right]^2\right]^{1/2}$$

where,

F_x, F_y, F_z = Reaction forces in kips (Fig. F.4a)

M_x, M_y M_z = Reaction moments in kip-inches (Fig. F.4a)

$M'_x = M_x + F_y E_z$

$M'_y = M_y + F_x E_z + F_z E_x$

$M'_z = M_z + F_y E_x$

dx, dz, Ex, Ez = See Fig. F.4a

a_1, a_2, a_3 = Prying Action Coefficients (See Table T.3)

4.3.2.5 Base Plate Having (2) Anchor Bolts

Bolt Tension T:

$$T = a_1 \times \left[\frac{M_x'}{2\,G'}\right] + a_2 \times \left[\frac{M_z'}{dx}\right] + a_3 \times \left[\frac{F_y}{2}\right]$$

Bolt Shear S:

$$S = \left[\left[\frac{F_x}{2}\right]^2 + \left[\frac{F_z}{2} + \frac{M_y'}{dx}\right]^2\right]^{1/2}$$

where,

F_x, F_y, F_z = Reaction forces in kips

M_x, M_y, M_z = Reaction moments in kip-inches

$M_x' = M_x + F_y E_z$

$M'_y = M_y + F_x E_z + F_z E_x$

$M'_z = M_z + F_y E_x$

a_1, a_2, a_3 = Prying action coefficients (see Table T.4)

$G' = \text{Min}\ [G_1, G_2]$

dx, G_1, G_2, E_x, E_z: See Fig. F.4b

4.4 General Finite Element Solution

Step 3 (Fig. F.1) verification utilizes finite element techniques as implemented in the Baseplate II computer program. This is the most versatile solution procedure since any geometry and reaction set can be handled. Description of this program can be found in Ref. 3.1.9.

4.4.1 Finite Element Model Inputs

Sample BASEPLATE II input listings for L6 x 6 x 3/4 are contained in Attachment G. FEM's shall be constructed using these inputs as guidelines. In no case shall the element length to width ratio exceed 3.

Variables used in the FEM of baseplates that require standardization for this project are given below:

Bolt Stiffnesses:

Bolt Type	Diameter (in)	Tension Stiffness K_T (K/in)	Shear Stiffness K_V (K/in)
Hilti Kwik	1/4 to 1 1/4	461	111
Hilti Super Kwik	1/4 to 1 1/4	461	111
Richmond Insert	1 1/2	3460	652
Richmond Insert	1	2175	485

Values for K_T and K_V are reproduced from Attachment G6, Reference 3.1.2. 2

The values of K_C, representing the stiffness of the concrete subgrade, are defined by the equation:

$$K_C = 1.077E_C \sqrt{A} \ (\#/") \quad (A \text{ is as defined in Ref. 3.1.9})$$

4.5 Embedment Plates Solution

Prior verification of embedment plates were done following the procedures outlined in Appendix 4 of Reference 3.1.3. Currently this specification is under reivew, therefore embeded plate qualification procedures will be provided at a later date.

4.5.1 Sources of Input Forces

Will be provided at a later date.

4.5.2 Basis of Interaction Curves

Will be provided at a later date.

4.5.3 Use of Interaction Curves

Will be provided at a later date.

4.5.4 Finite Element Solution Method

Will be provided at a later date.

5.0 ACCEPTANCE CRITERIA

The acceptance criteria utilized to verify adequacy of support anchorages are detailed below. Failure of any of the acceptance criteria will require modification of the support to the extent that the criteria can be met.

5.1 Bolt and Insert Qualification

There are two methods of securing base angles or plates to concrete surfaces via threaded fasteners. One method utilizes Hilti Kwik or Super Kwik expansion anchors. The second utilizes embedded, Richmond inserts, and standard ASTM bolts. The qualification of either fastening type requires the evaluation of a stress ratio that incorporates the actual and allowable forces. The equations must be used for both the loading combinations containing the OBE and SSE conditions (Ref. 3.1.1).

5.1.1 Hilti Expansion Anchors - Kwik and Super Kwik

$$\frac{T}{T'(S.R.)} + \frac{S}{S'(S.R.)} \leq 1.0$$

5.1.2 Richmond Inserts and Bolts

For Inserts:

$$\left[\frac{T}{T'(S.R.)}\right]^{4/3} + \left[\frac{S}{S'(S.R.)}\right]^{4/3} \leq 1.0$$

For Bolts:

$$\left[\frac{T}{T'}\right]^{2} + \left[\frac{S}{S'}\right]^{2} \leq 1.0$$

5.1.3 Definition of Terms

The definition of the variables used in the above equations are given below:

T = Actual Tension

T' = Allowable Tension

S = Actual Shear

S' = Allowable Shear

S.R. = Separation Ratio

The actual tension and shear values are those calculated using methodologies prescribed in Section 4.0 of this instruction.

The allowable tension and shear values are given in the appropriate tables of Attachment B of this instruction. Attachment E describes methodology for determining embedded length.

The separation ratio is a value calculated by dividing the separation distance provided by the minimum separation distance required. The appropriate required separation distances are provided in the tables of Attachment C of this instruction. The separation ratio must be greater than or equal to 0.500, and not greater than 1.0.

5.2 Embedment Plate Qualification

5.2.1 General

Will be provided at a later date.

5.2.2 Acceptance Criteria

Will be provided at a later date.

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5.3 Steel Qualification

General

Attachment F contains anchorage plate/angle stress calculations to be used in conjunction with the hand calculations in Step 2 (See Section 4.3).

Reference 3.1.7 provides criteria for the verification of steel support component adequacy. Qualification of steel anchorage components shall be in accordance with Attachment F and Ref. 3.1.7 as appropriate.

5.4 Weld Qualification

Reference 3.1.7 provides verification and acceptance criteria for anchorage component weldment adequacy.

6.0 QUALITY ASSURANCE

The checking requirements of the work performed in accordance with this instruction shall be in accordance with Impell Quality Assurance Procedures (Ref. 3.1.11).

Attachment H contains checklists that may be incorporated in the anchorage verification calculation as appropriate for the anchor type.

ATTACHMENT A

INTERACTION DIAGRAMS

A.1 General

This attachment contains the interaction diagrams for three types of regular anchorages. These are (1) angle having two bolts; (2) angle having single bolt; and, (3) base plate having two bolts. The following sections contain discussions regarding the derivation and use of the procedure. 2

A.2 Interaction Equations

The interaction diagrams are based on the equations presented in Section 4.3. To limit the number of diagrams, it is necessary to make various assumptions regarding anchorage geometry and the relation of several anchorage reaction forces and/or moments. It is possible that an actual analysis will have a geometry and/or reactions that do not exactly match the generic diagrams. For these cases, the support is checked by use of the closest conservative interaction diagram.

The following tables explain each variable used in the generation of the diagrams. In addition, what constitutes a conservative variable value is presented. This can be used to assist in the selection of a conservative interaction diagram.

Angle Having One Bolt

Variable (Section 4.3)	Comment	Conservative Value
F_x	Reaction force parallel to longitudinal axis (Fig. F.3b).	
F_y	Reaction force parallel to free leg (Fig. F.3b).	
F_z	Assumed equal to αF_x (Note 1) (Fig. F.3b).	Actual $F_z \leq \alpha F_x$
M_x	Assumed equal to αM_z (Note 1)	Actual $M_x \leq \alpha M_z$
M_z	Reaction moment (Fig. F.3b).	
a_1, a_2, a_3	Prying coefficients according to angle type. (Table T.2)	
C	Angle leg size L6 as specified on diagram.	
G	Bolt gage length	
E_x	Eccentricity of channel with respect to mid-length of angle. Assumed equal to 10% of C (Fig. F.3b).	Actual eccentricity $\leq$ .10C
e'	Shortest distance from bolt to end of angle. Assumed to be equal to 3.6".	Actual e' $\geq$ assumed value = 3.6"
L	Equals overall length of angle.	

2

2

Notes:

1. α equals an arbitrary constant - 0.25, 0.5, or 1.0

Angle Having Two Bolt

Variable (Section 4.3)	Comment	Conservative Value
F_x	Reaction force parallel to longitudinal axis (Fig. F.3a).	
F_y	Reaction force parallel to the free leg (Fig. F.3a).	
F_z	Assumed equal to αF_x (Note 1) (Fig. F.3a).	Actual $F_z \leq \alpha F_x$
M_x	Assumed equal to αM_z (Note 1)	Actual $M_x \leq \alpha M_z$
M_y	Reaction torsion assumed to be the maximum allowable pure torsion from connecting channel.	Actual $M_y \leq$ assumed max.
M_z	Reaction moment (Fig. F.3a).	
a_1, a_2, a_3	Prying coefficients according to angle type (Table T.1).	
C	Angle leg size, L6 as specified on diagram.	
G	Bolt gage length	
e'	Shortest distance from bolt.	1.75" $\leq$ e'
dx	Distance between bolts. Various values assumed for different diagrams.	
E_x	Eccentricity of channel with respect to center of bolts (Fig. F.3a). Assumed equal to 1.6".	Actual eccentricity $\leq$ 1.6"

2

Notes:

1. α equals an arbitrary constant - 0.25, 0.5., or 1.0

Baseplate Having Two Bolts

Variable (Section 4.3)	Comment	Conservative Value
F_x	Reaction force parallel to longitudinal axis (Fig. F.4b).	
F_y	Reaction force perpendicular to the bolted surface (Fig. F.4b).	
F_z	Assumed equal to αF_x (Note 1) (Fig. F.4b).	Actual $F_z \leq \alpha F_x$
M_x	Assumed equal to αM_z (Note 1) (Fig. F.4b).	Actual $M_x \leq \alpha M_z$
M_y	Reaction torsion assumed to be the maximum allowable pure torsion from connecting channel.	Actual $M_y \leq$ assumed max.
M_z	Reaction moment (Fig. F.4b)	
a_1, a_2, a_3	Prying coefficients. (Table T.4)	
C	Plate width (Fig. F.4b)	C = 8"
G'	Shortest distance from plate edge to bolts (Fig. F.4b).	G' = 3.5"
e'	Shortest distance from plate edge to end of plate.	e' = 1.75"
dx	Distance between bolts. Various values assumed.	21" $\geq$ dx $\geq$ 13" value
E_x	Eccentricity of connecting channel centroid from the mid-point between bolts. (Fig. 7.4b). Assumed to be equal to 1.6".	Actual $E_x \leq$ 1.6"
E_z	Eccentricity of connecting channel centroid from the bolt centerline (Fig. 7.4b). Assumed to be 1.6".	Actual $E_z \leq$ 1.6"

Notes:

1. α equals an arbitrary constant - 0.25, 0.5, or 1.0

A.3 Interaction Diagram Use

Data regarding anchorage geometry and support analysis reactions are required for the selection and use of the interaction diagrams. The following steps outline the appropriate procedure.

Step 1: Obtain anchorage reactions. Both OBE and SSE conditions must be considered. (Section A.2, Fx, Fy, Fz, Mx, My, Mz. These are in the local anchorage coordinate system).

Step 2: Record actual anchorage geometry. Make note of actual angle type and relevant dimension. The relevant data are indicated in Section A.2 for the particular common anchorage type.

Step 3: Select interaction diagram. Refer to the interaction diagram cover index (Section A.5) and select the interaction diagram having properties that are most similar to the actual geometry (from Step 2). Note the geometry used to generate the interaction diagram must be conservative with respect to the actual geometry. The notes under the column heading Conservative Values in Section A.2 indicate conservative values. 2

Step 4: Calculate force ratios. For both OBE and SSE conditions, calculate the following ratios:

$$\alpha F = \frac{Fz}{Fx} \quad ; \quad \alpha M = \frac{Mx}{Mz}$$

and, $\alpha R = \max [\alpha F, \alpha M]$. Also, check that My is less than that assumed in the particular diagram. If αR exceeds 1.0, F_x and, or M_z may be conservatively increased to lower αR. The new F_x and, or M_z values must be kept for the rest of the evaluation. 2

Step 5: Locate appropriate interaction curve. Given the interaction diagram from Step 3, select the proper curve according to the αR ratio (Step 4) and actual Fx (Step 1 or 4) as follows. Select the closest curve having a value such that $\alpha R < \alpha$; and, $Fx < V$ (the curves are labeled having several values of α and V). V can be linearly interpolated such that a constant α is maintained. 2

Step 6: Plot the key reactions and determine if the anchorage qualifies. Graph the point represented by the actual Fy and Mz. (Step 1). If this point lies below the curve selected in Step 5, the anchorage qualifies. If the point lies above the curve, the anchorage cannot be qualified via interaction diagrams (proceed to a refined verification procedure).

A.4 Example

The following numerical example illustrates the process. Consider a common angle anchorage having two bolts under the SSE condition.

Step 1: Obtain anchorage reactions.

Fx = .80 kips Mx = 4.4 k-in.
Fy = 1.0 kips My = 3.0 k-in.
Fz = .16 kips Mz = 20.0 k-in.

Step 2: Record relevant anchorage geometry.

Connecting channel: C6 X 8.2
Base Angle L6 X 6 X 3/4
Hilti quick bolts (dia. = 1.25 in.)
Embedment length = 9.0 inches
dx = 20. inches
C = 6 inches
G = 2.6 inches
E_x = 0 inches (Channel centered between bolts)
e' = 2 inches

Step 3: Select interaction diagram. Figure A.4 is selected using the index in Section A.5. Check that it is conservative. i.e., 2

Actual G = 2.5 in. $\leq$ 3.75 in., thus OK
Actual dw = 20 in. 21" $\geq$ dx $\geq$ 16" OK
Actual E_x = 0 $\leq$ 1.6", OK
Actual e' = 2" 1.75" $\leq$ e'
Actual embedment = 9.0 in. $\geq$ 7.5 in., OK 2

Step 4: Calculate force ratios.

$$\alpha F = \frac{Fz}{Fx} = \frac{.16}{.80} = .20 \quad ; \quad \alpha M = \frac{Mx}{Mz} = \frac{4.4}{20.0} = .22$$

Use αR = .22. My = 3 k-in. < 7.74 k-in. thus OK.

Step 5: Locate appropriate interaction curve.

αR = .22 , Fx = .80

Thus, use curve labeled α = .25, and V = 2.0 kip. See Figure A.4.

Step 6: Plot the key reactions. Plot Fy = 1.0 kip and Mz = 20.0 k-in.

The point plots below the interaction curve in the pass region. The anchorage is qualfied.

A.5 Interaction Diagrams

Anchorage	Bolt Type	Allowable Force Conditions	Min. embed.	Figure Number
Angle 6 X 6 X 3/4 having one bolt	1.25 inch Hilti Kwik Bolt	OBE	7.5"	A.1
		SSE	7.5"	A.2
Angle 6 X 6 X 3/4 having two bolts	1.25 inch Hilti Kwik Bolt	OBE	7.5"	A.3
		SSE	7.5"	A.4
Angle 6 X 6 X 3/4 having one bolt	1.0 inch Richmond Insert	OBE	-	A.5
		SSE	-	A.6
Angle 6 X 6 X 3/4 having two bolts	1.0 inch Richmond Insert	OBE	-	A.7
		SSE	-	A.8
Angle 6 X 6 X 3/4 having one bolt	1.25 inch Hilti Kwik Bolt	OBE	10.5"	A.9
		SSE	10.5"	A.10
Angle 6 X 6 X 3/4 having two bolts	1.25 inch Hilti Kwik Bolt	OBE	10.5"	A.11
		SSE	10.5"	A.12
Angle 6 X 4 X 3/4 having one bolt	1.25 inch Hilti Kwik Bolt	OBE	7.5"	A.13
		SSE	7.5"	A.14
Angle 6 X 4 X 3/4 having two bolts	1.25 inch Hilti Kwik Bolt	OBE	7.5"	A.15
		SSE	7.5"	A.16
Angle 6 X 4 X 3/4 having one bolt	1.0 inch Richmond Insert	OBE	-	A.17
		SSE	-	A.18
Angle 6 X 4 X 3/4 having two bolts	1.0 inch Richmond Insert	OBE	-	A.19
		SSE	-	A.20
Angle 6 X 4 X 3/4 having one bolt	1.25 inch Hilti Kwik Bolt	OBE	10.5"	A.21
		SSE	10.5"	A.22
Angle 6 X 4 X 3/4 having two bolts	1.25 inch Hilti Kwik Bolt	OBE	10.5"	A.23
		SSE	10.5"	A.24
Angle 5 X 5 X 3/4 having one bolt	1.25 inch Hilti Kwik Bolt	OBE	7.5"	A.25
		SSE	7.5"	A.26
Angle 5 X 5 X 3/4 having two bolts	1.25 inch Hilti Kwik Bolt	OBE	7.5"	A.27
		SSE	7.5"	A.28
Angle 5 X 5 X 3/4 having one bolt	1.0 inch Richmond Insert	OBE	-	A.29
		SSE	-	A.30

Anchorage	Bolt Type	Allowable Force Conditions	Min. embed.	Figure Number
Angle 5 X 5 X 3/4 having two bolts	1.0 inch Richmond Insert	OBE	—	A.31
		SSE	—	A.32
Angle 5 X 5 X 3/4 having one bolt	1.25 inch Hilti Kwik Bolt	OBE	10.5"	A.33
		SSE	10.5"	A.34
Angle 5 X 5 X 3/4 having two bolts	1.25 inch Hilti Kwik Bolt	OBE	10.5"	A.35
		SSE	10.5"	A.36
Plate 1 X 8 having two bolts	1.25 inch Hilti Kwik Bolt	OBE	7.5"	A.37
		SSE	7.5"	A.38
Plate 1 X 8 having two bolts	1.0 inch Richmond Insert	OBE	—	A.39
		SSE	—	A.40
Plate 1 X 8 having two bolts	1.25 inch Hilti Kwik Bolt	OBE	10.5"	A.41
		SSE	10.5"	A.42

(2)

The interaction diagrams may be used for values of Separation Ratio (S.R.) less than 1.0 provided the input forces and moments are increased by the reciprocol of the S.R.:

Examples:

F_x = .80 kips F'_x = .80/SR = 1.0 kips

F_y = 1.0 kips F'_y = 1.0/SR = 1.25 kips

F_z = .16 kips F'_z = .16/SR = .20 kips

M_x = 4.4 in-kips M'_x = 4.4/SR = 5.5 in-kips

M_y = 3.0 in-kips M'_y = 3.0/SR = 3.75 in-kips

M_z = 20.0 in-kips M'_z = 20.0/SR = 25.0 in-kips

where SR = .80

The interaction diagrams may then be entered using the increased values.

TENSION FORCE Fy (kips)

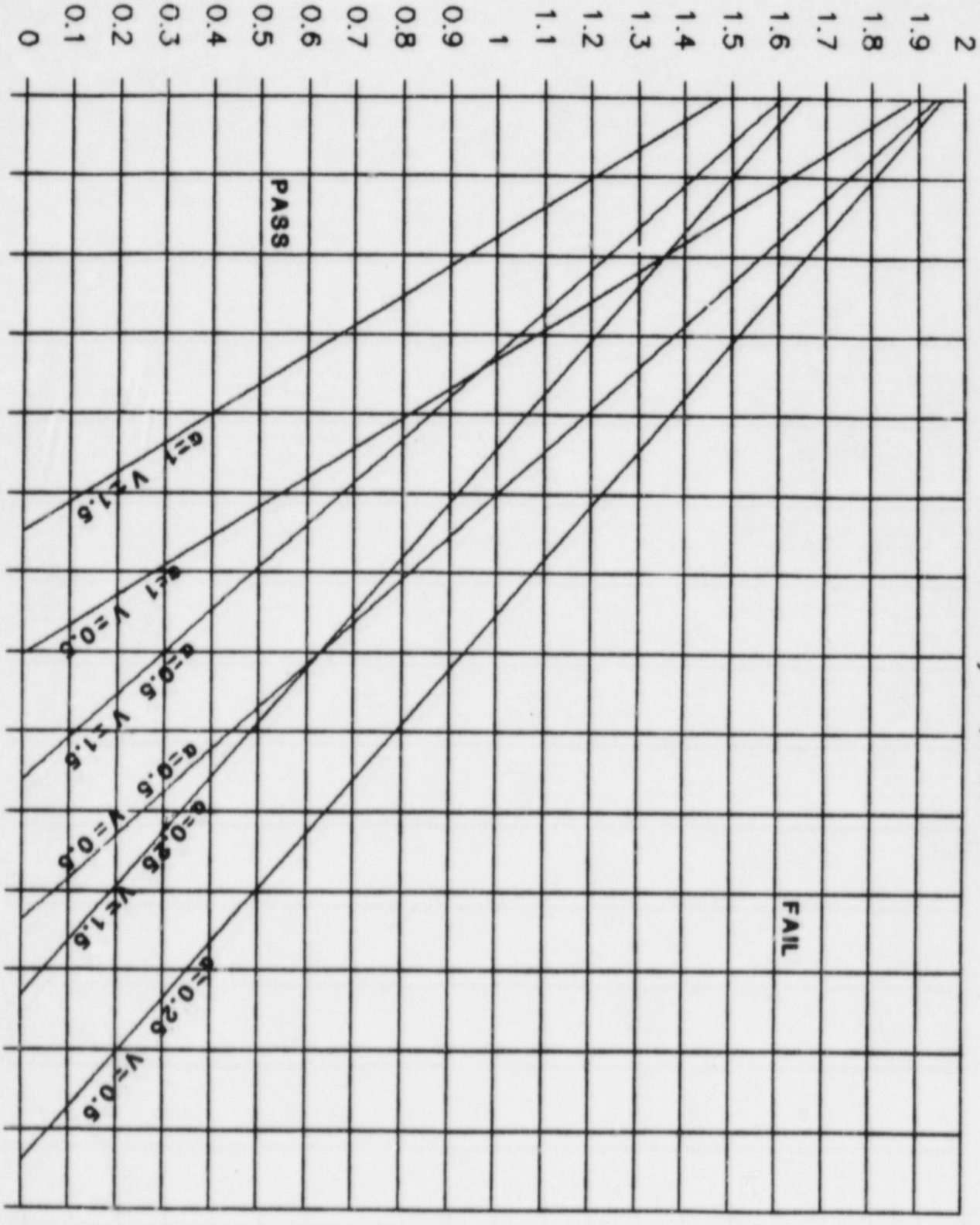
BOLT: DIAMETER = 1.25 "
EMBED LENGTH = 7.5 " min
GAGE DISTANCE (G) = 3.75"max/2.25 "min
EDGE DISTANCE (e') = 3.6 " min
ECCENTRICITY (Ex) = 0.6 " max
NUMBER OF BOLTS = 1

My : NOT REQUIRED

ANGLE: L6x6x3/4

BOLTS: HKB & KSKB
T' = 6.22 kips
S' = 7.136 kips

2



MOMENT Mz (kip-in)

FIGURE A.1 TENSION FORCE vs MOMENT OBE

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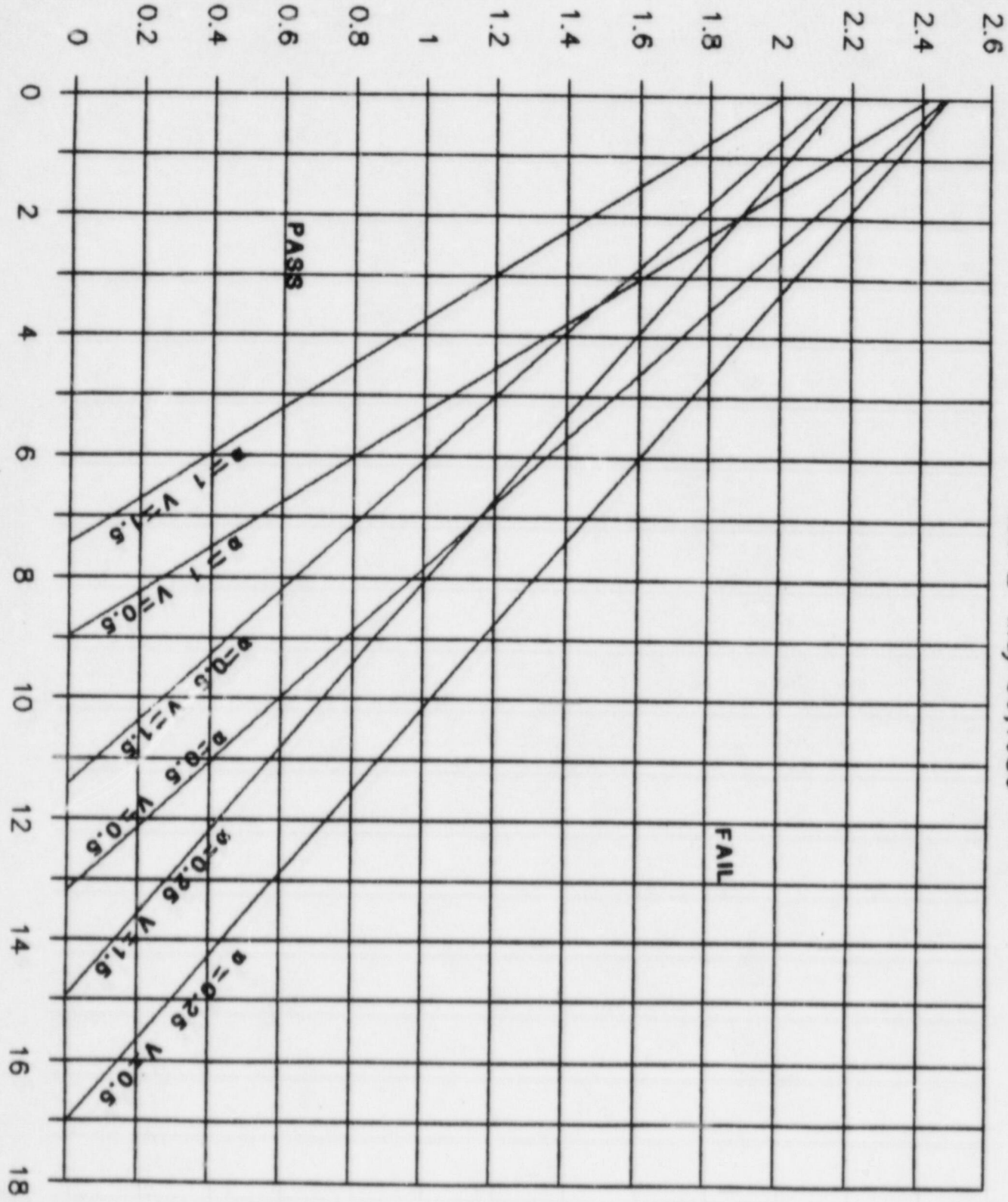
TENSION FORCE Fy (kips)

BOLT: DIAMETER = 1.25 "
EMBED LENGTH = 7.5 " min
GAGE DISTANCE (G) = 3.75"max/2.25 "min
EDGE DISTANCE (e') = 3.6 " min
ECCENTRICITY (Ex) = 0.6 " max
NUMBER OF BOLTS = 1

My : NOT REQUIRED

ANGLE: L6x6x3/4

BOLTS: HKB & KSKB
T' = 7.775 kips
S' = 8.92 kips



MOMENT Mz (kip-in)

FIGURE A.2 TENSION FORCE vs MOMENT SSE

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TENSION FORCE Fy (kips)

BOLT: DIAMETER = 1.25 "
EMBED LENGTH = 7.5 " min
C TO C SPACING (dx) = 26"max/16" min
GAGE DISTANCE (G) = 3.75"max/2.25"min
EDGE DISTANCE (e') = 1.75 " min
ECCENTRICITY (Ex) = 1.6 " max
NUMBER OF BOLTS = 2

My ≤ 7.739 kip-in

ANGLE: L6x6x3/4

BOLTS: HKB & HSKB
T' = 6.22 kips
S' = 7.136 kips

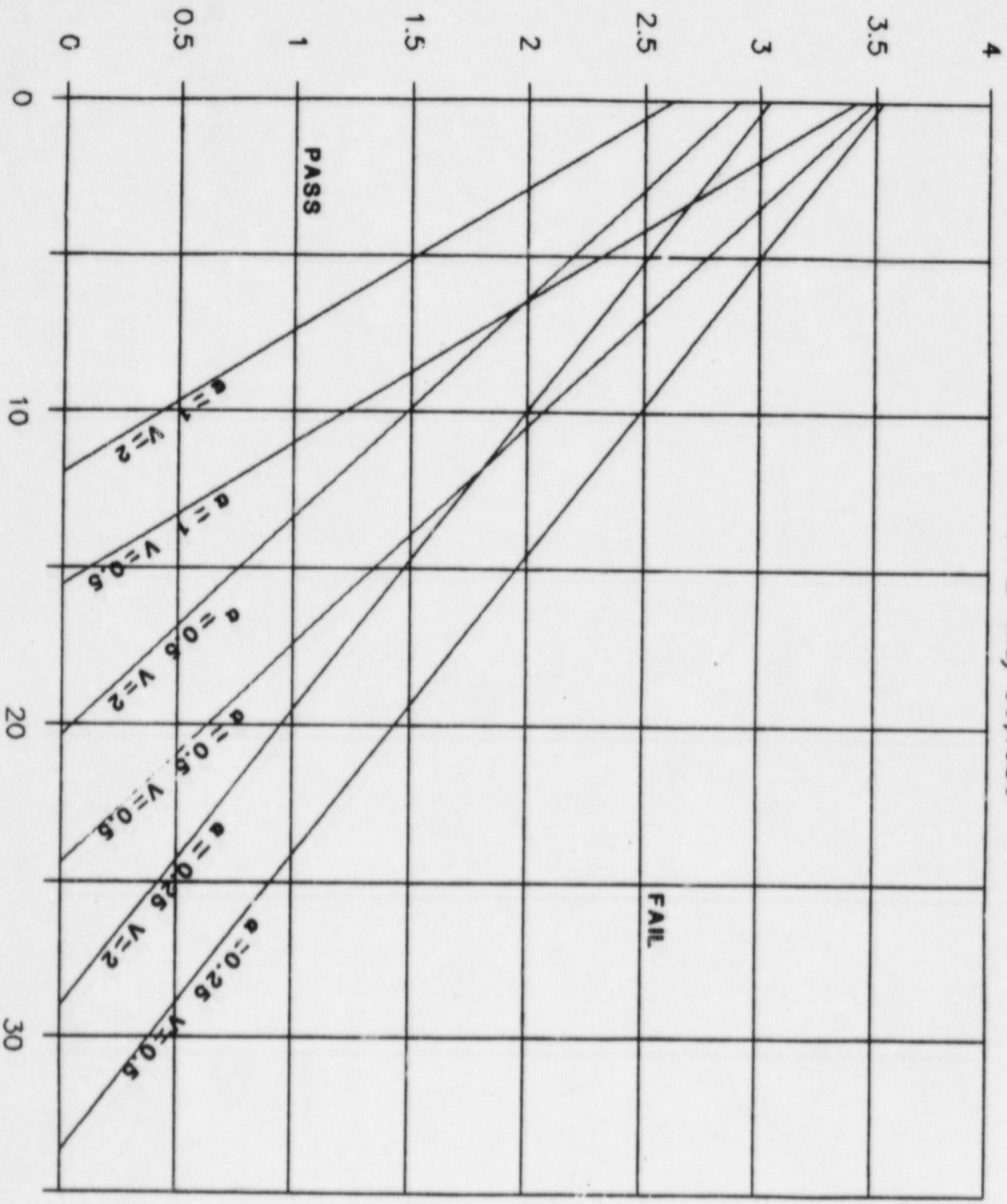

FIGURE A.3 TENSION FORCE vs MOMENT OBE

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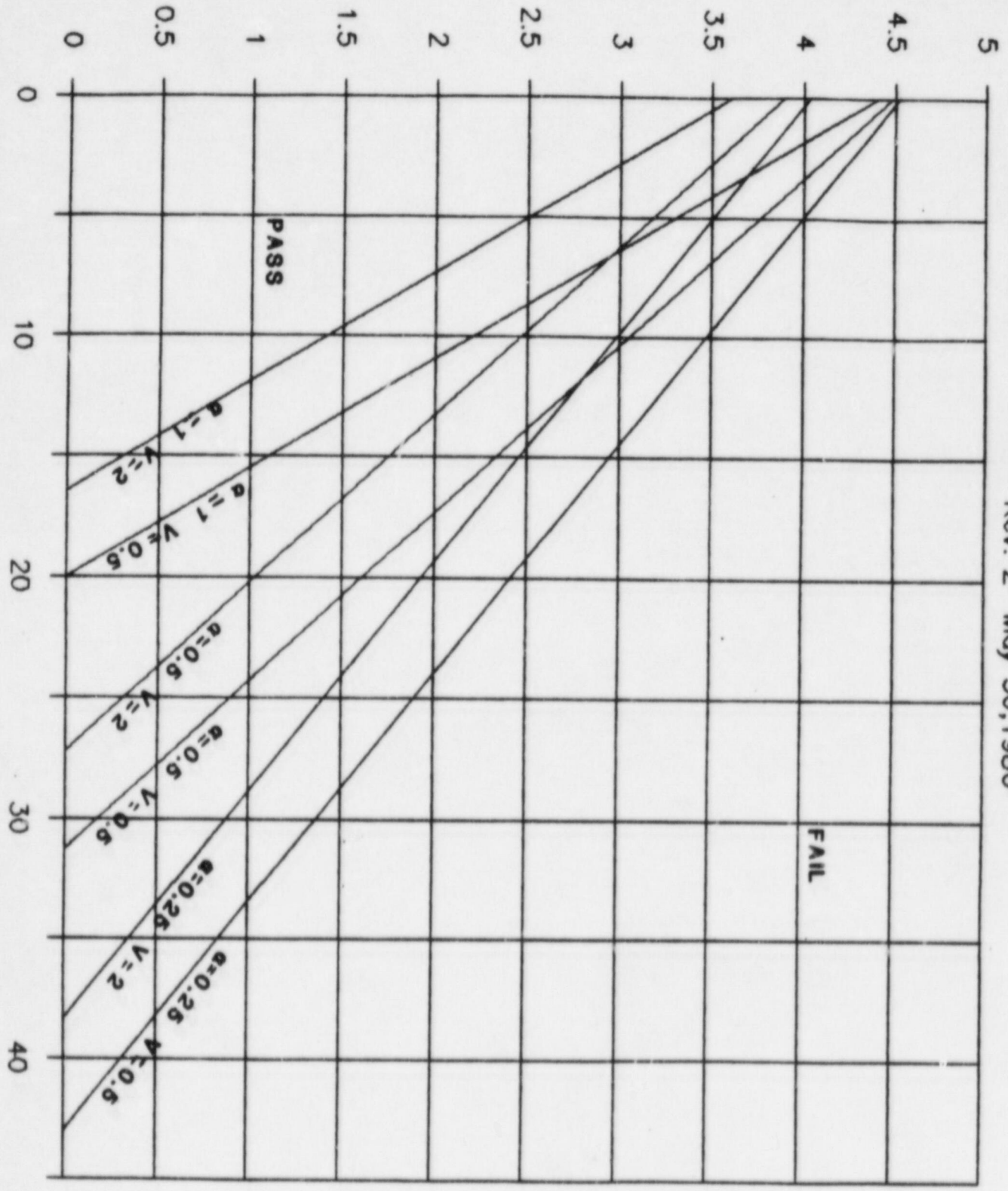
TENSION FORCE Fy (kips)

BOLT:	DIAMETER =	1.25 "
	EMBED LENGTH =	7.5 " min
C TO C SPACING (dx) =		26"max/16" min
GAGE DISTANCE (G) =		3.75"max/2.25"min
EDGE DISTANCE (e') =		1.75 " min
ECCENTRICITY (Ex) =		1.6 " max
NUMBER OF BOLTS =		2

My ≤ 7.739 kip-in

ANGLE: L6x6x3/4

BOLTS: HKB & HSKB
T' = 7.775 kips
S' = 8.92 kips



MOMENT Mz (kip-in)

FIGURE A.4 TENSION FORCE vs MOMENT SSE

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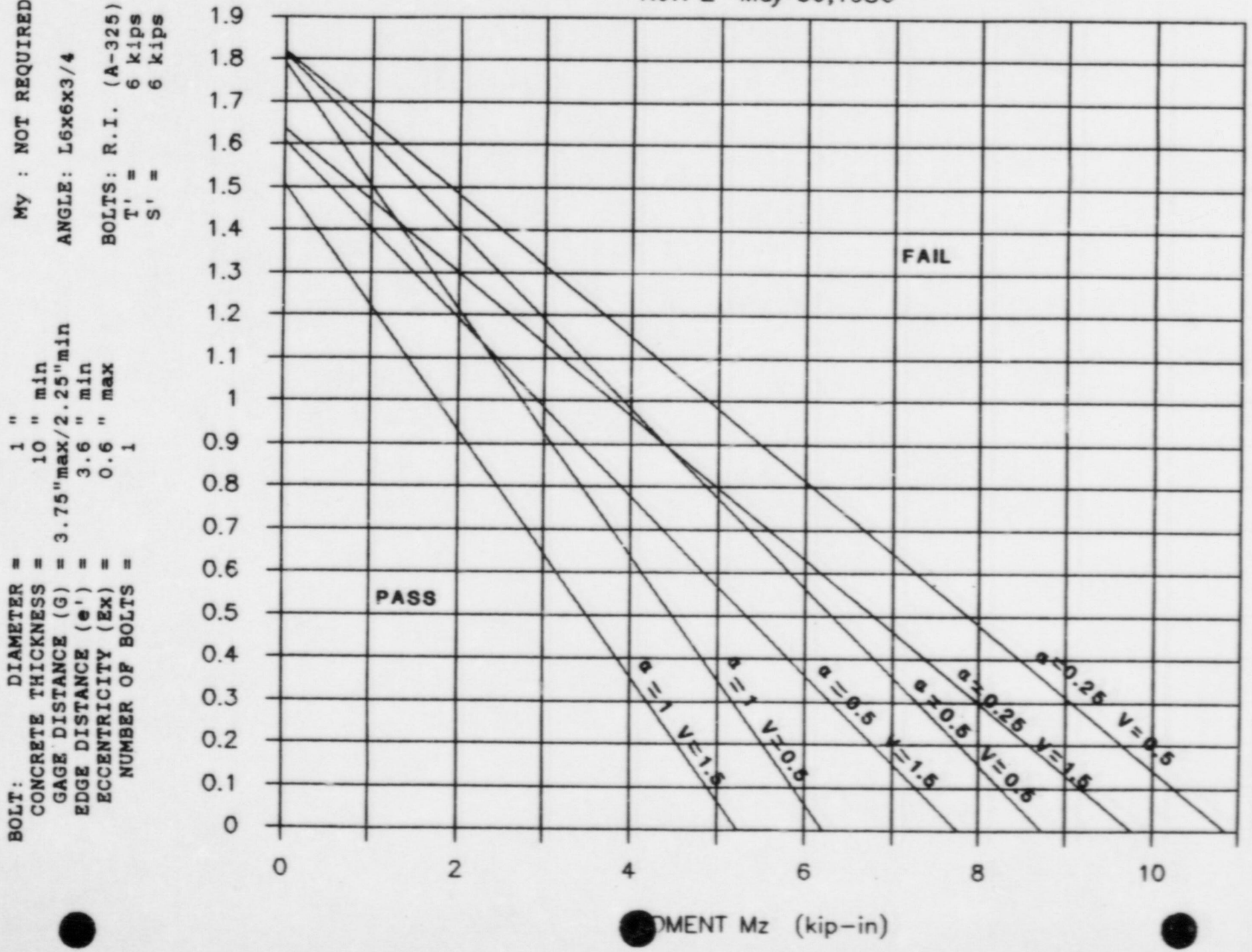
FIGURE A.5 TENSION FORCE vs MOMENT OBE

Rev. 2 May 30,1986



TENSION FORCE Fy (kips)

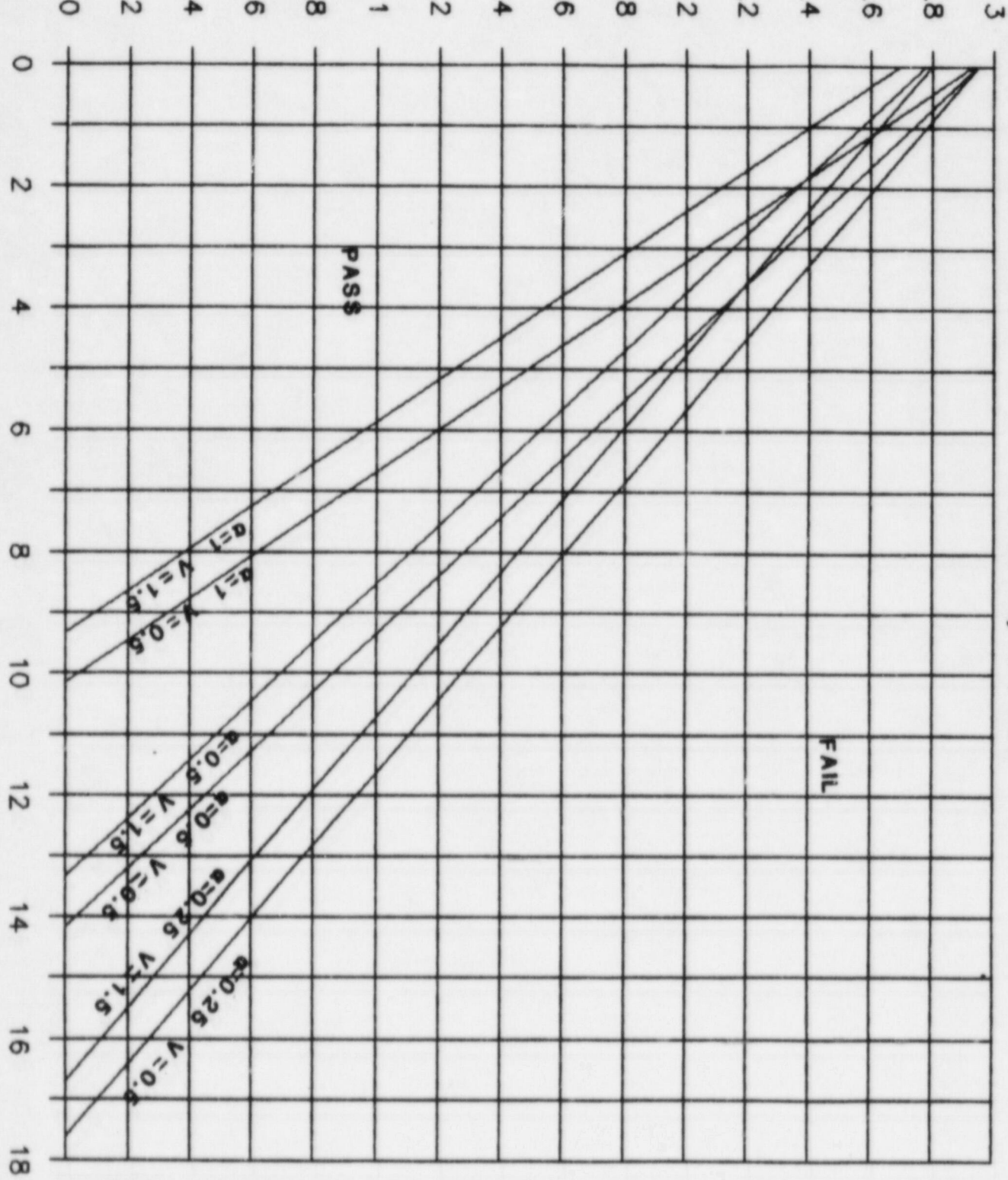
BOLT: DIAMETER = 1 "
CONCRETE THICKNESS = 10 " min
GAGE DISTANCE (G) = 3.75"max/2.25"min
EDGE DISTANCE (e') = 3.6 " min
ECCENTRICITY (Ex) = 0.6 " max
NUMBER OF BOLTS = 1

My : NOT REQUIRED

ANGLE: L6x6x3/4

BOLTS: R.I. (A-325)
T' = 9.6 kips
S' = 9.6 kips

2



MENT Mz (kip-in)

FIGURE A.6 TENSION FORCE vs MOMENT SSE

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2

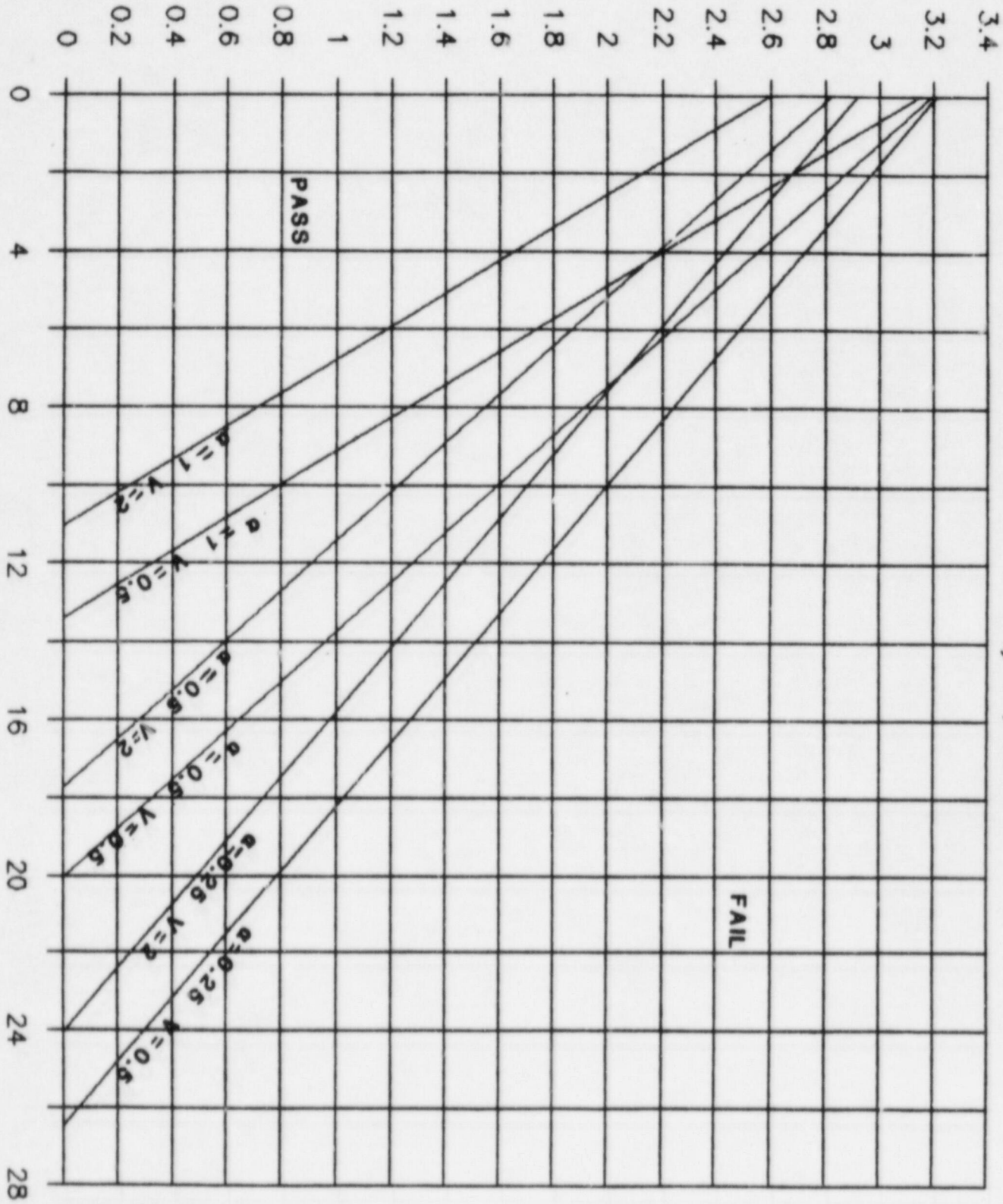
TENSION FORCE Fy (kips)

BOLT: DIAMETER = 1 "
CONCRETE THICKNESS = 10 " min
C TO C SPACING (dx) = 26"max/16" min
GAGE DISTANCE (G) = 3.75"max/2.25"min
EDGE DISTANCE (e') = 1.75 " min
ECCENTRICITY (Ex) = 1.6 " max
NUMBER OF BOLTS = 2

My ≤ 7.739 kip-in

ANGLE: L6x6x3/4

BOLTS: R.I. (A-325)
T' = 6 kips
S' = 6 kips



MOMENT Mz (kip-in)

FIGURE A.7 TENSION FORCE vs MOMENT OBE

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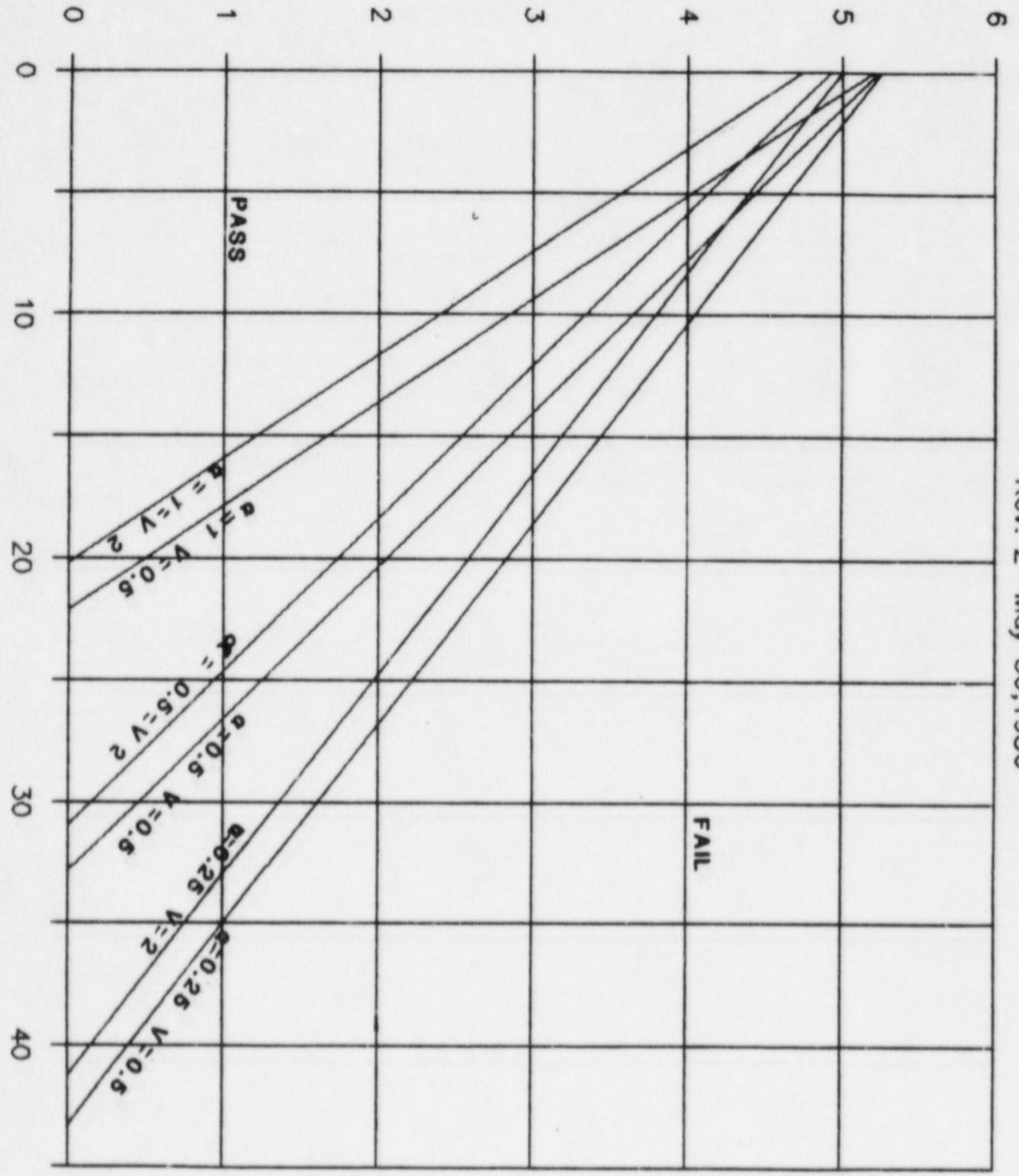
TENSION FORCE Fy (kips)

BOLT: DIAMETER =	1 "	My ≤ 7.739 kip-in
CONCRETE THICKNESS =	10 " min	
C TO C SPACING (dx) =	26"max/16" min	ANGLE: L6x6x3/4
GAGE DISTANCE (G) =	3.75"max/2.25"min	
EDGE DISTANCE (e') =	1.75 " min	BOLTS: R.I. (A-325)
ECCENTRICITY (Ex) =	1.6 " max	T' = 9.6 kips
NUMBER OF BOLTS =	2	S' = 9.6 kips

2



MOMENT Mz (kip-in)

FIGURE A.8 TENSION FORCE vs MOMENT SSE

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TENSION FORCE Fy (kips)

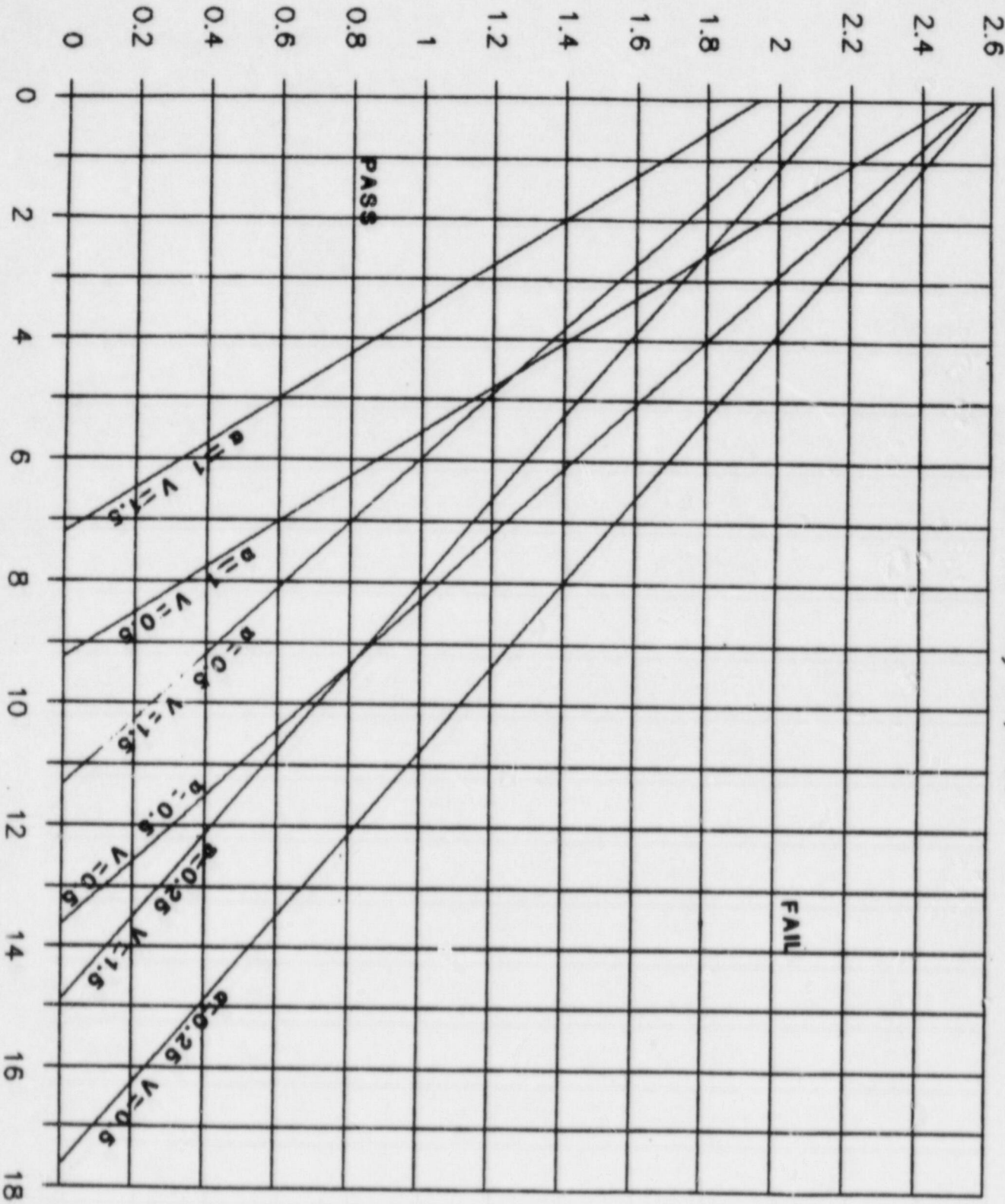
BOLT: DIAMETER = 1.25 "
EMBED LENGTH = 10.5 " min
GAGE DISTANCE (G) = 3.75"max/2.25 "min
EDGE DISTANCE (e') = 3.6 " min
ECCENTRICITY (Ex) = 0.6 " max
NUMBER OF BOLTS = 1

My : NOT REQUIRED

ANGLE: L6x6x3/4

BOLTS: HKB & KSKB
T' = 8.18 kips
S' = 7.136 kips

2



MOMENT Mz (kip-in)

FIGURE A.9 TENSION FORCE vs MOMENT OBE

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2

TENSION FORCE Fy (kips)

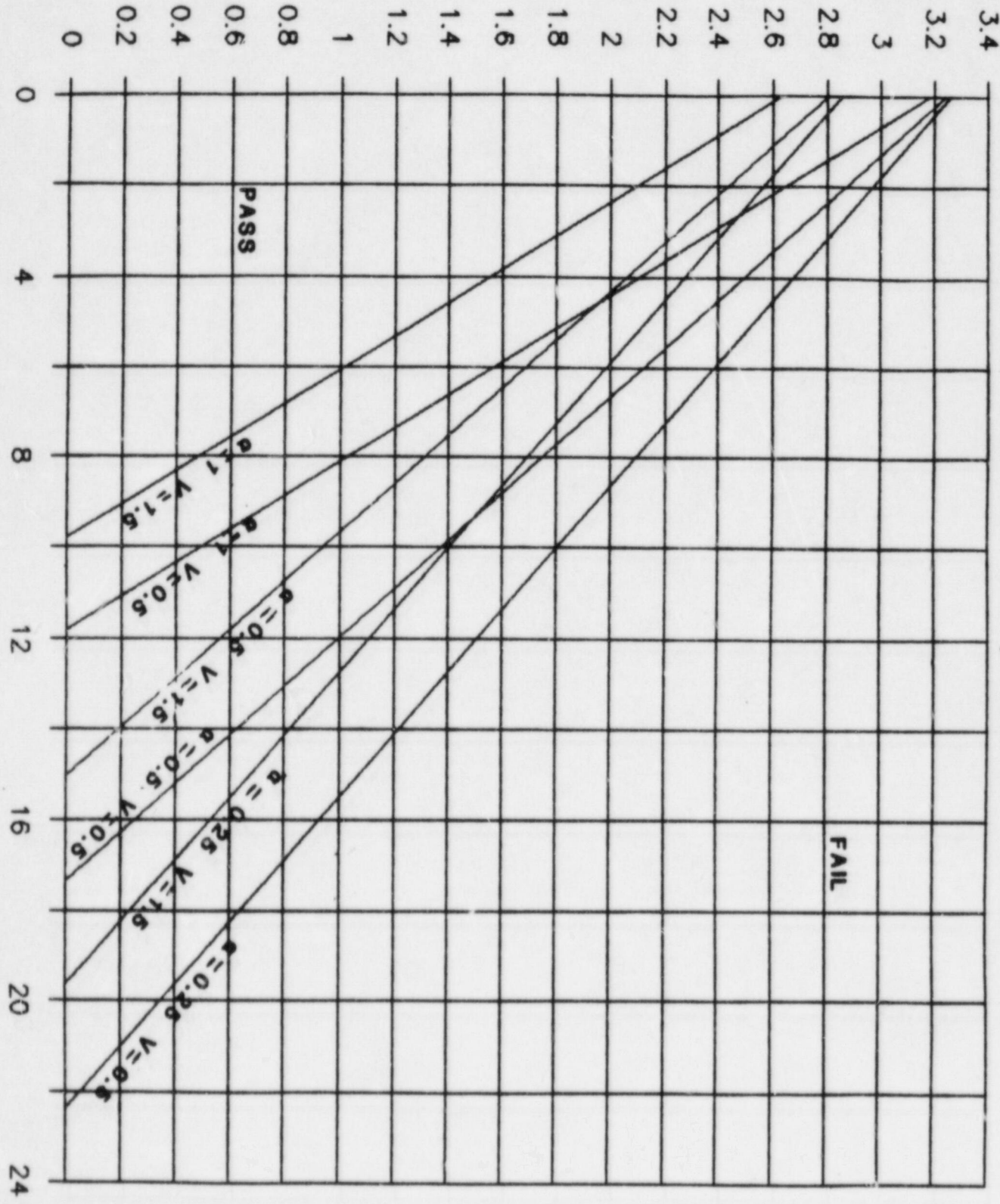
BOLT: DIAMETER = 1.25 "
EMBED LENGTH = 10.5 " min
GAGE DISTANCE (G) = 3.75"max/2.25 "min
EDGE DISTANCE (e') = 3.6 " min
ECCENTRICITY (Ex) = 0.6 " max
NUMBER OF BOLTS = 1

My : NOT REQUIRED

ANGLE: L6x6x3/4

BOLTS: HKB & KSKB
T' = 10.225 kips
S' = 8.92 kips

2



MOMENT Mz (kip-in)

FIGURE A.10 TENSION FORCE vs MOMENT SSE

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2

TENSION FORCE Fy (kips)

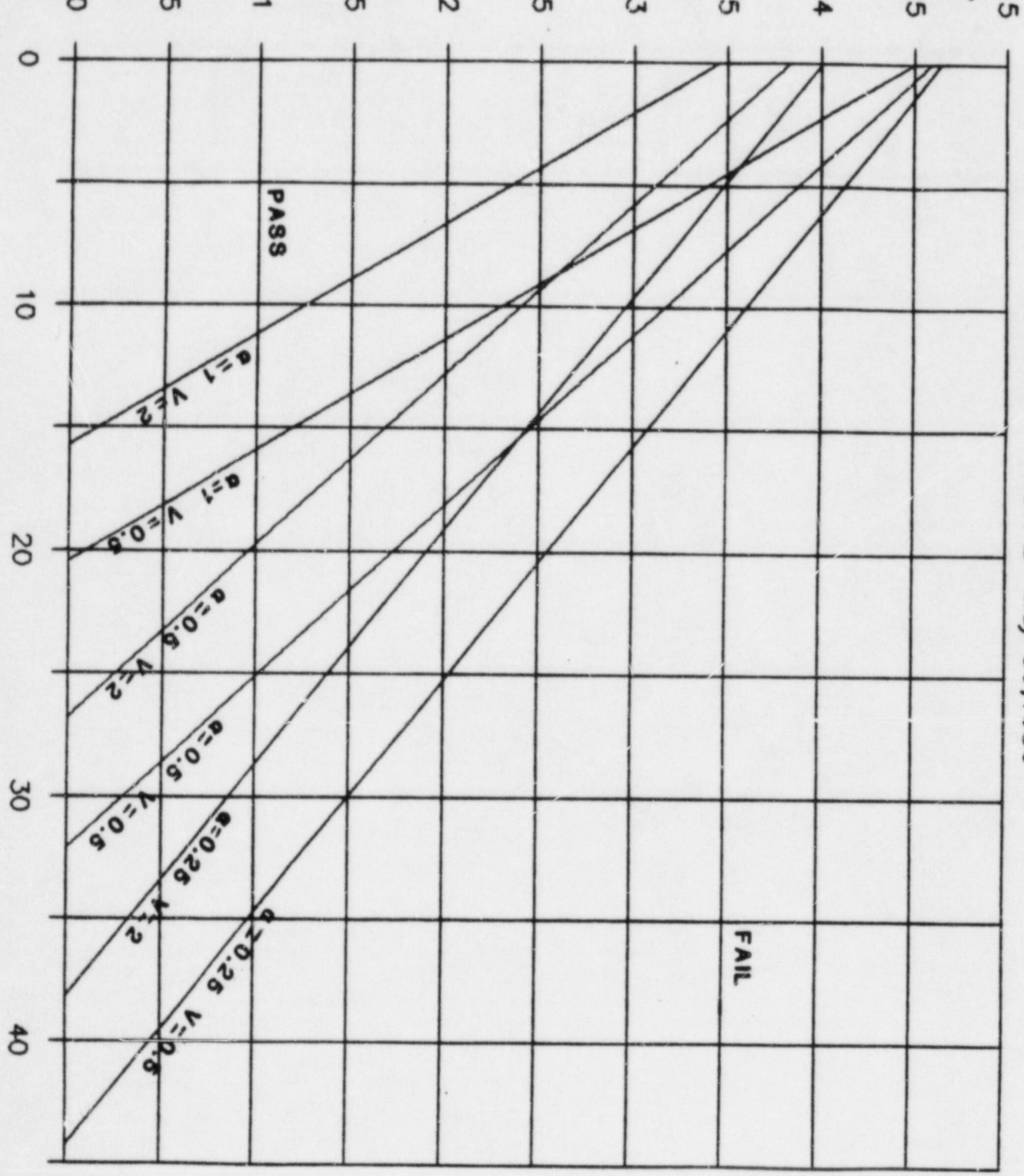
BOLT: DIAMETER = 1.25 "
EMBED LENGTH = 10.5 " min
C TO C SPACING (dx) = 26"max/16" min
GAGE DISTANCE (G) = 3.75"max/2.25"min
EDGE DISTANCE (e') = 1.75 " min
ECCENTRICITY (Ex) = 1.6 " max
NUMBER OF BOLTS = 2

My ≤ 7.739 kip-in

ANGLE: L6x6x3/4

BOLTS: HKB & HSKB
T' = 8.18 kips
S' = 7.136 kips

2



MOMENT Mz (kip-in)

FIGURE A.11 TENSION FORCE vs MOMENT OBE

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2

TENSION FORCE Fy (kips)

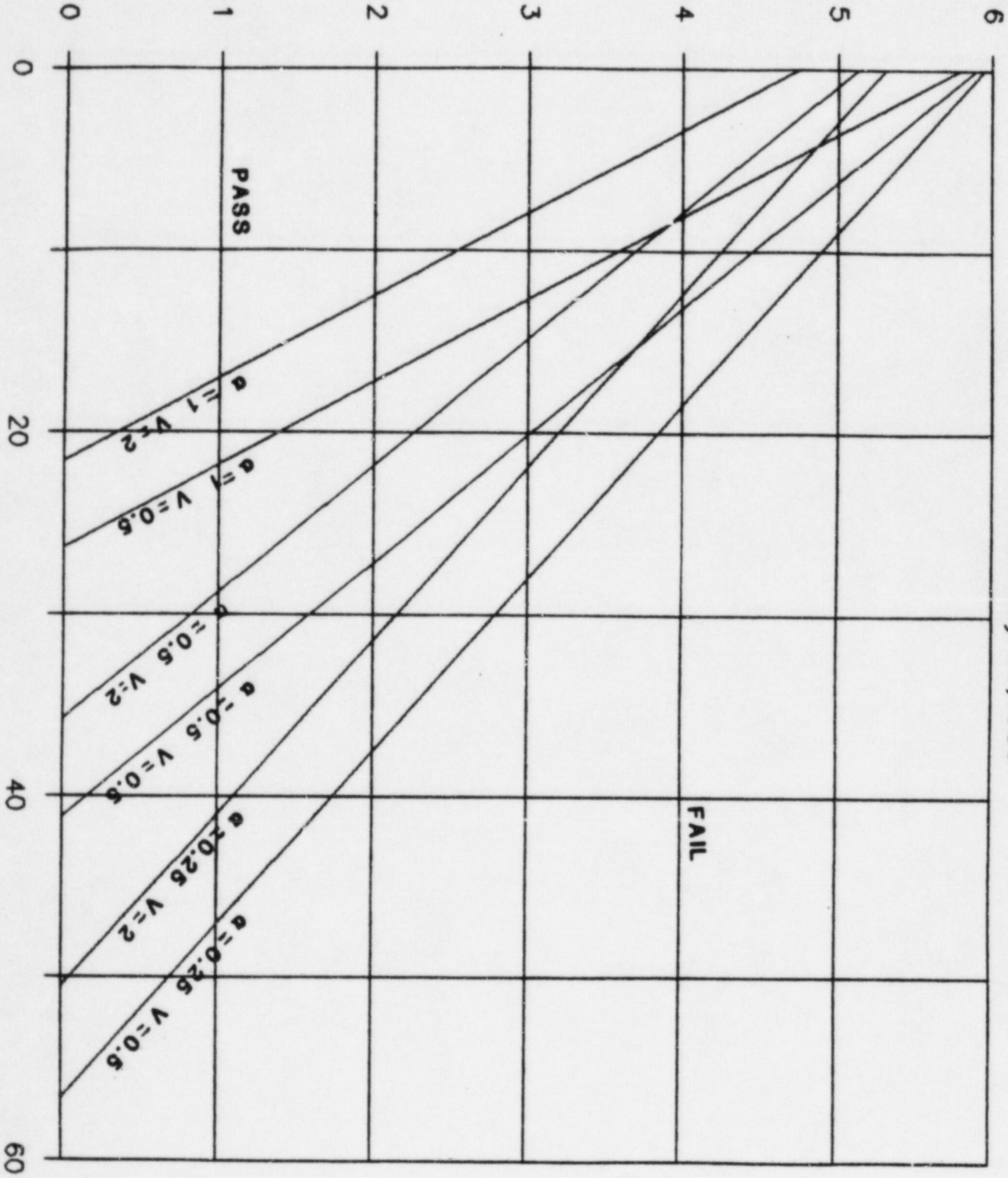
BOLT: DIAMETER = 1.25 "
EMBED LENGTH = 10.5 " min
C TO C SPACING (dx) = 26"max/10" min
GAGE DISTANCE (G) = 3.75"max/2.25"min
EDGE DISTANCE (e') = 1.75 " min
ECCENTRICITY (Ex) = 1.6 " max
NUMBER OF BOLTS = 2

My ≤ 7.739 kip-in

ANGLE: L6x6x3/4

BOLTS: HKB & HSKB
T' = 10.225 kips
S' = 8.92 kips

2



MOMENT Mz (kip-in)

FIGURE A.12 TENSION FORCE vs MOMENT SSE

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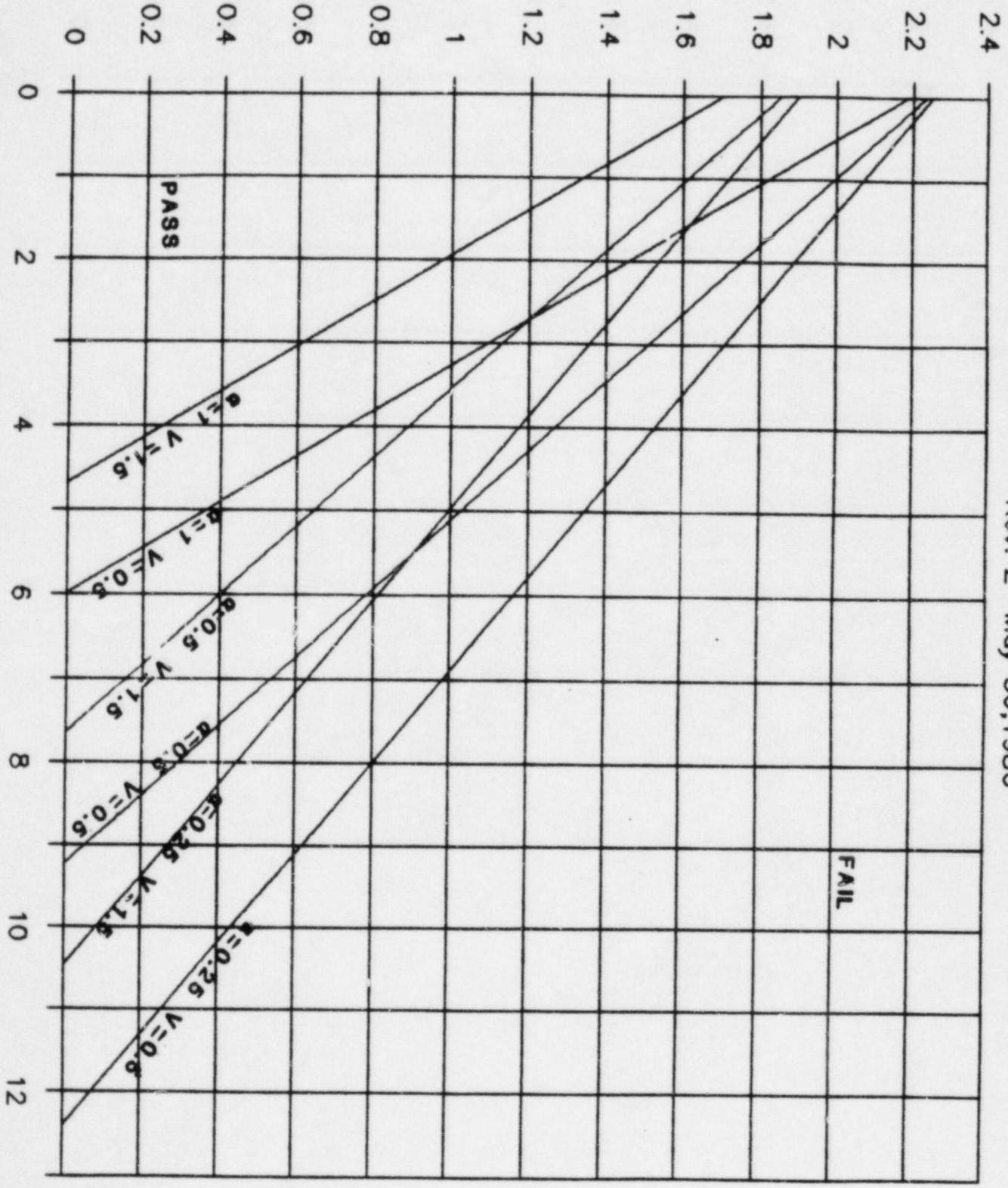
TENSION FORCE Fy (kips)

BOLT:	DIAMETER =	1.25 "	
	EMBED LENGTH =	7.5 " min	
	GAGE DISTANCE (G) =	2.25"max/1.75 "min	
	EDGE DISTANCE (e') =	3.6 " min	
	ECCENTRICITY (Ex) =	0.6 " max	
	NUMBER OF BOLTS =	1	

My : NOT REQUIRED

ANGLE: L6x4x3/4

BOLTS: HKB & KSKB
T' = 6.22 kips
S' = 7.136 kips



MOMENT Mz (kip-in)

FIGURE A.13 TENSION FORCE vs MOMENT OBE

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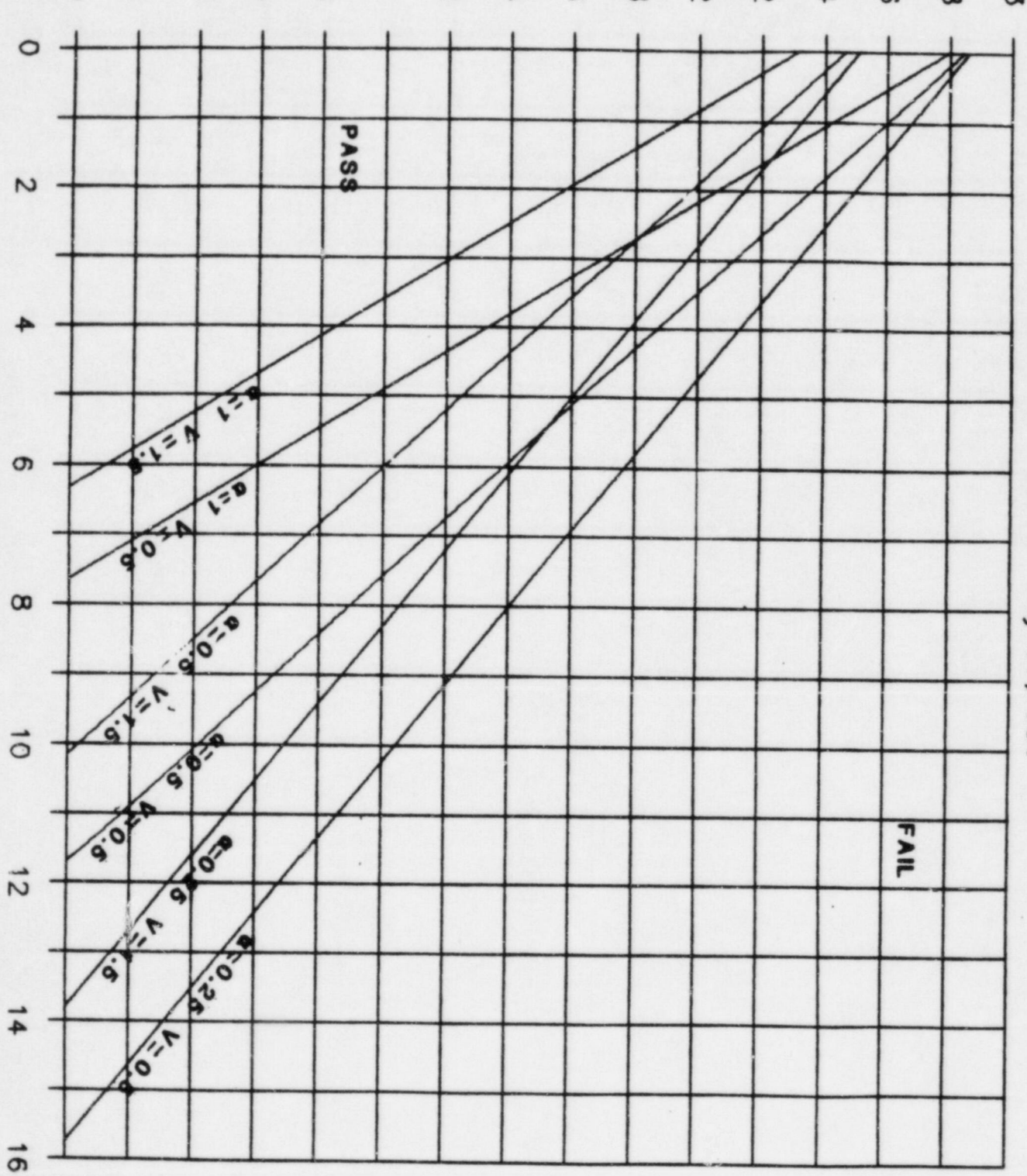
TENSION FORCE Fy (kips)

BOLT: DIAMETER = 1.25 "
EMBED LENGTH = 7.5 " min
GAGE DISTANCE (G) = 2.25"max/1.75 "min
EDGE DISTANCE (e') = 3.6 " min
ECCENTRICITY (Ex) = 0.6 " max
NUMBER OF BOLTS = 1

My : NOT REQUIRED

ANGLE: L6x4x3/4

BOLTS: HKB & KSKB
T' = 7.775 kips
S' = 8.92 kips



MOMENT Mz (kip-in)

FIGURE A.14 TENSION FORCE vs MOMENT SSE

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TENSION FORCE Fy (kips)

BOLT: DIAMETER = 1.25 "
EMBED LENGTH = 7.5 " min
C TO C SPACING (dx) = 26"max/16" min
GAGE DISTANCE (G) = 2.25"max/1.75"min
EDGE DISTANCE (e') = 1.75 " min
ECCENTRICITY (Ex) = 1.6 " max
NUMBER OF BOLTS = 2

My ≤ 7.739 kip-in

ANGLE: L6x4x3/4

BOLTS: HKB & HSKB
T' = 6.22 kips
S' = 7.136 kips

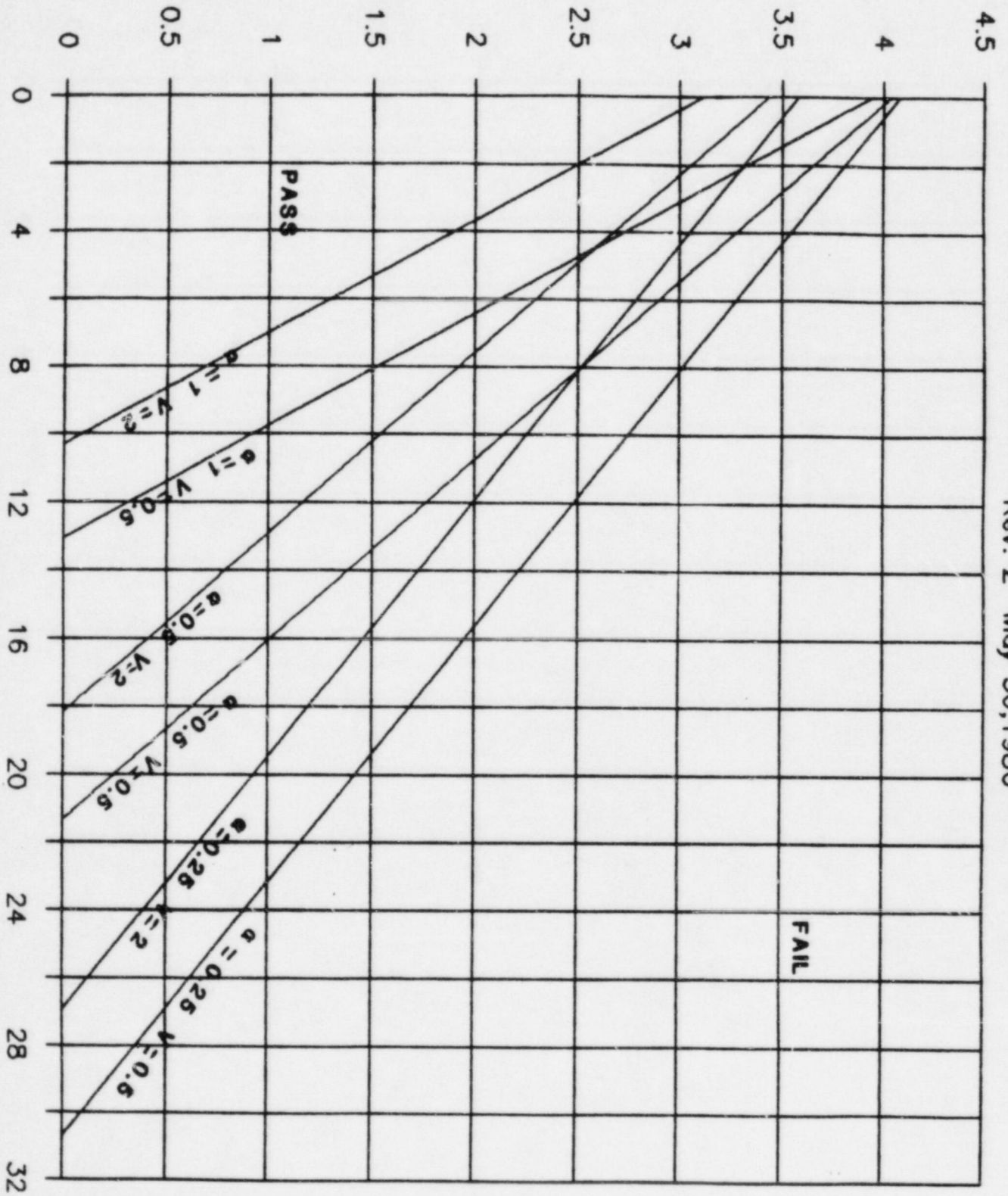


FIGURE A.15 TENSION FORCE vs MOMENT OBE

Rev. 2 May 30,1986

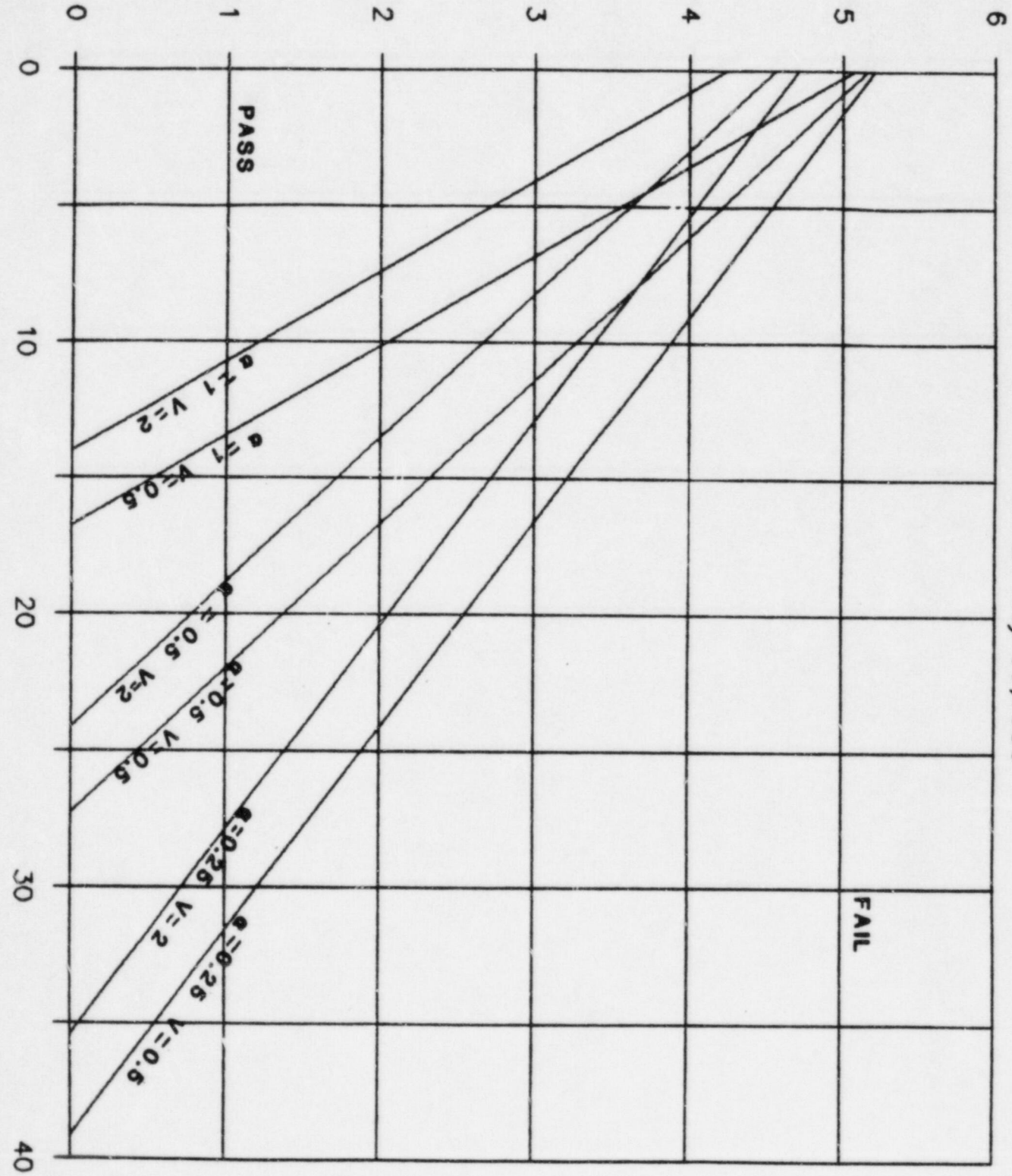
TENSION FORCE Fy (kips)

BOLT: DIAMETER = 1.25 "
EMBED LENGTH = 7.5 " min
C TO C SPACING (dx) = 26"max/16" min
GAGE DISTANCE (G) = 2.25"max/1.75"min
EDGE DISTANCE (e') = 1.75 " min
ECCENTRICITY (Ex) = 1.6 " max
NUMBER OF BOLTS = 2

My ≤ 7.739 kip-in

ANGLE: L6x4x3/4

BOLTS: HKB & HSKB
T' = 7.775 kips
S' = 8.92 kips



MOMENT Mz (kip-in)

FIGURE A.16 TENSION FORCE vs MOMENT SSE

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FIGURE A.17 TENSION FORCE vs MOMENT OBE

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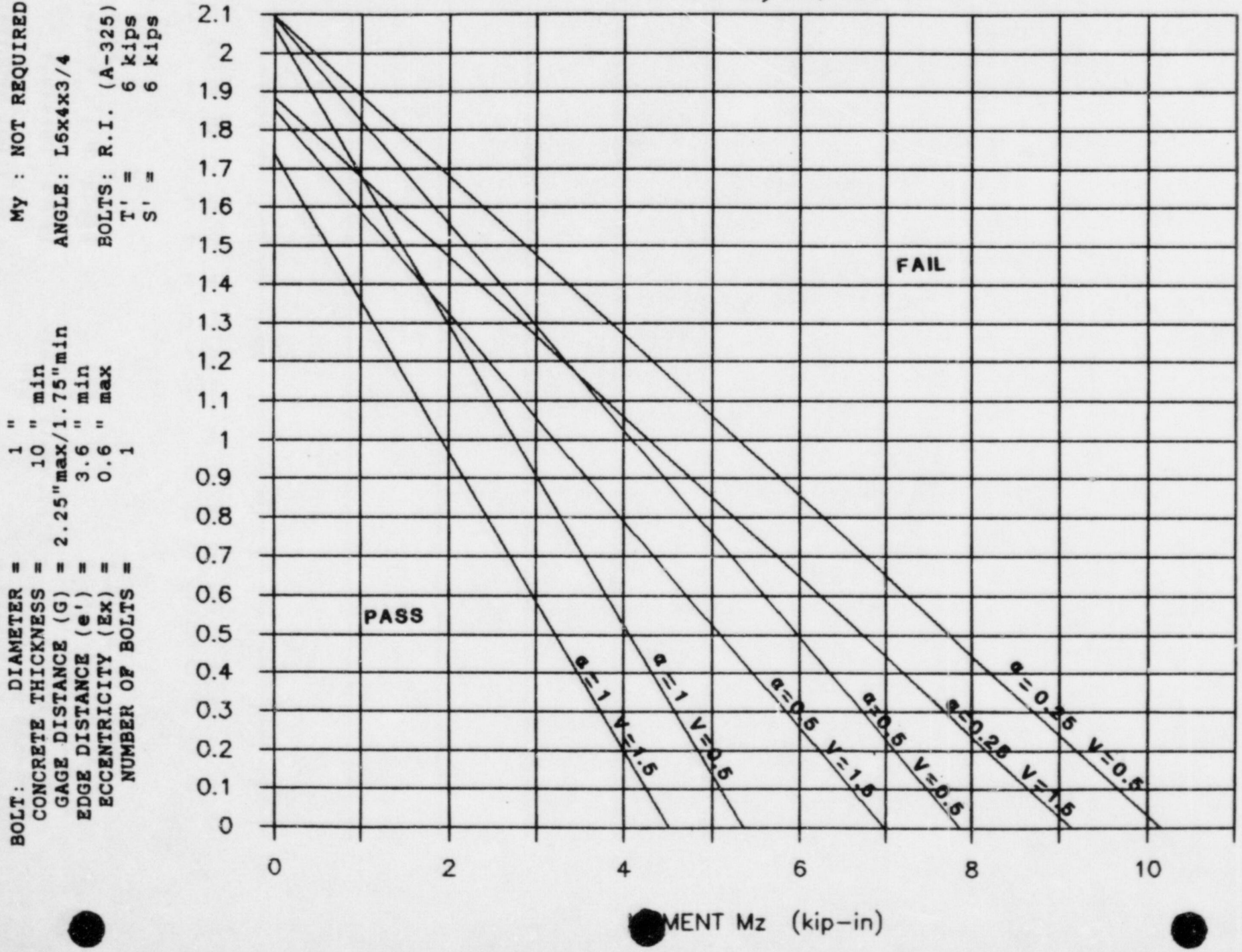


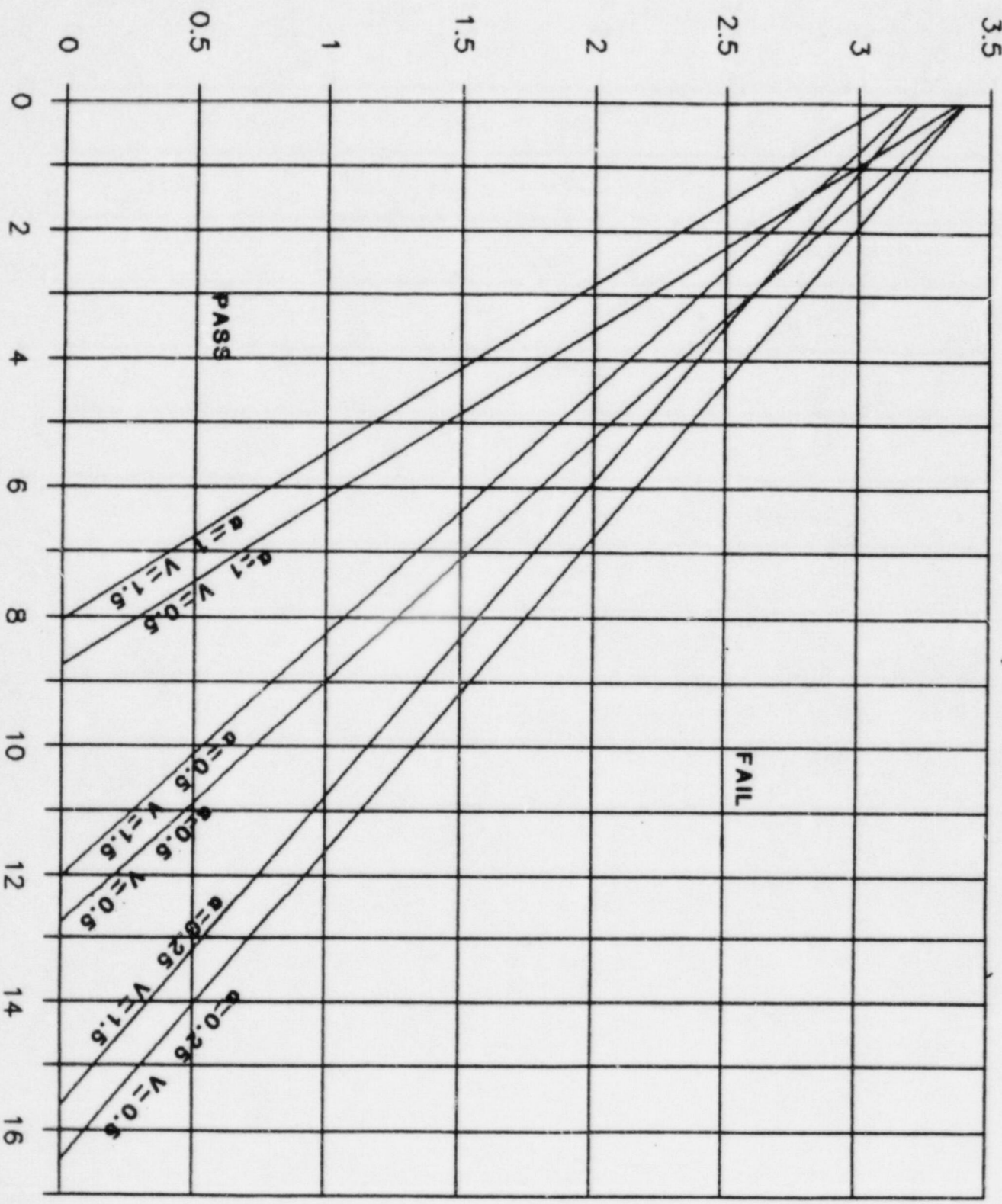
TENSION FORCE Fy (kips)

BOLT: DIAMETER = 1 "
CONCRETE THICKNESS = 10 " min
GAGE DISTANCE (G) = 2.25"max/1.75"min
EDGE DISTANCE (e') = 3.6 " min
ECCENTRICITY (Ex) = 0.6 " max
NUMBER OF BOLTS = 1

My : NOT REQUIRED

ANGLE: L6x4x3/4

BOLTS: R.I. (A-325)
T' = 9.6 kips
S' = 9.6 kips



MENT Mz (kip-in)

FIGURE A.18 TENSION FORCE vs MOMENT SSE

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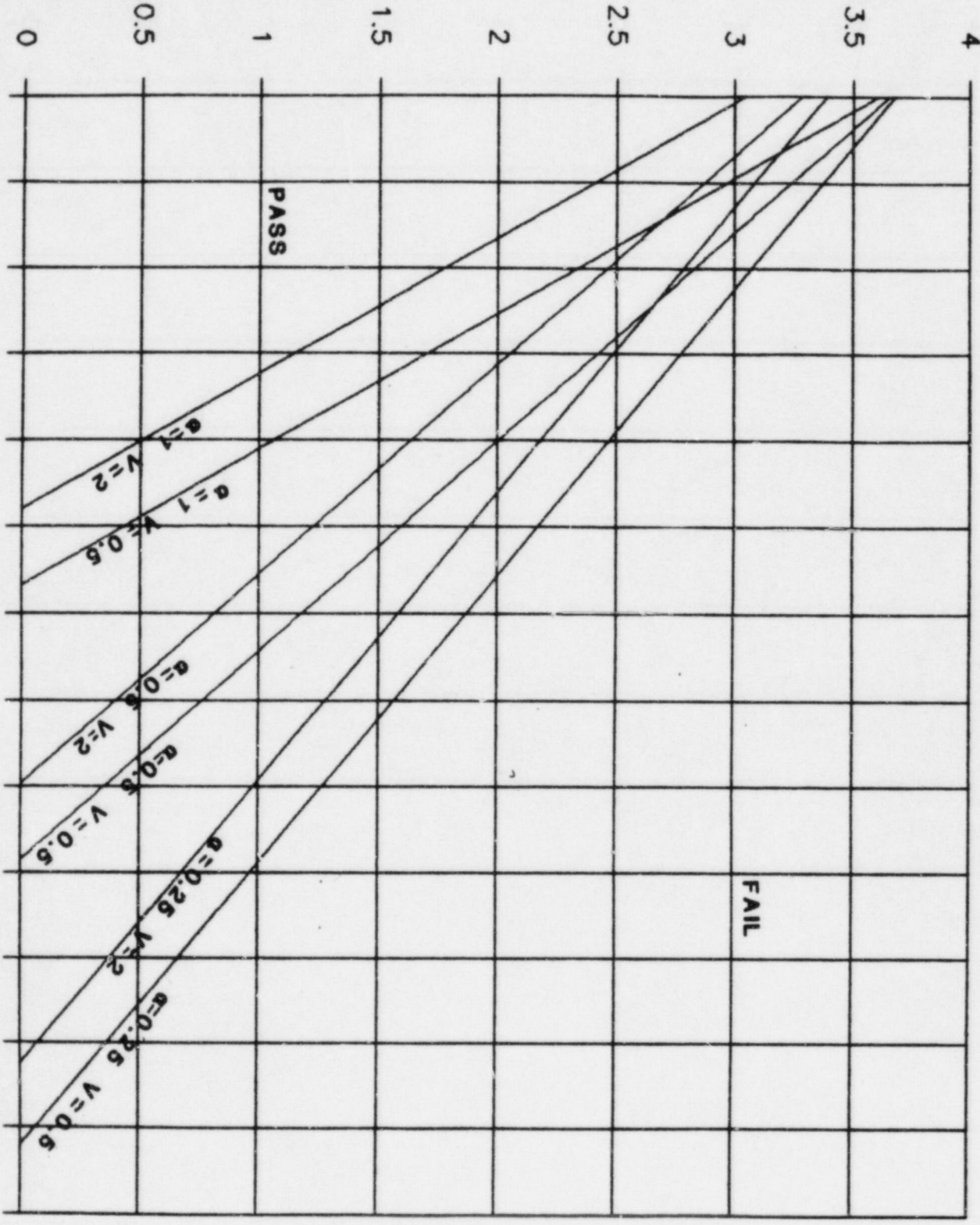
TENSION FORCE Fy (kips)

BOLT: DIAMETER = 1 "
CONCRETE THICKNESS = 10 " min
C TO C SPACING (dx) = 26"max/16" min
GAGE DISTANCE (G) = 2.25"max/1.75"min
EDGE DISTANCE (e') = 1.75 " min
ECCENTRICITY (Ex) = 1.6 " max
NUMBER OF BOLTS = 2

My ≤ 7.739 kip-in

ANGLE: L6x4x3/4

BOLTS: R.I. (A-325)
T' = 6 kips
S' = 6 kips



MENT Mz (kip-in)

FIGURE A.19 TENSION FORCE vs MOMENT OBE

Rev. 2 May 30,1986

2

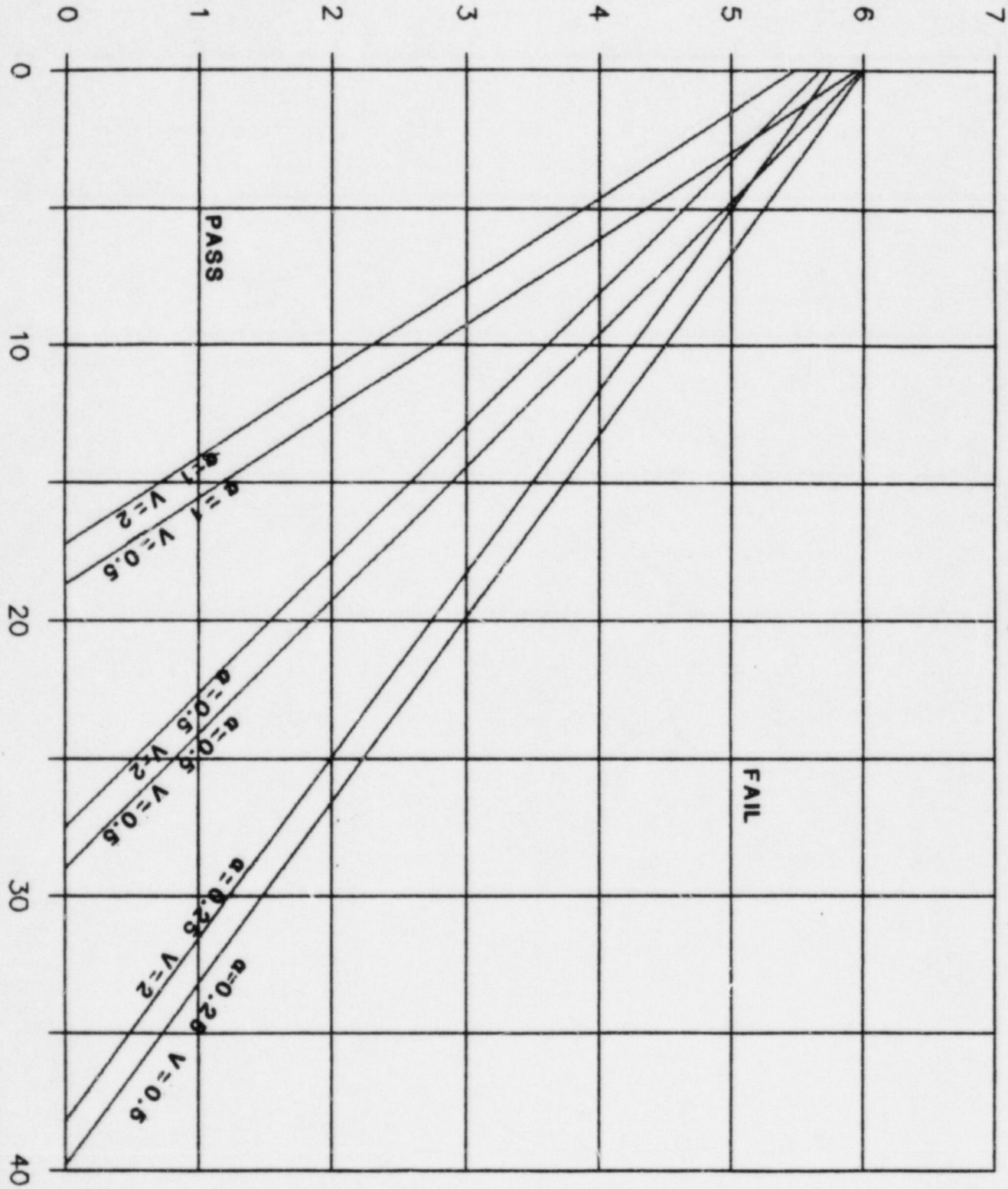
TENSION FORCE Fy (kips)

BOLT: DIAMETER	=	1 "
CONCRETE THICKNESS	=	10 " min
C TO C SPACING (dx)	=	26"max/16" min
GAGE DISTANCE (G)	=	2.25"max/1.75"min
EDGE DISTANCE (e')	=	1.75 " min
ECCENTRICITY (Ex)	=	1.6 " max
NUMBER OF BOLTS	=	2

My ≤ 7.739 kip-in

ANGLE: L6x4x3/4

BOLTS: R.I. (A-325)
T' = 9.6 kips
S' = 9.6 kips



MOMENT Mz (kip-in)

FIGURE A.20 TENSION FORCE vs MOMENT SSE

Rev. 2 May 30,1986

TENSION FORCE Fy (kips)

BOLT: DIAMETER = 1.25 "
EMBED LENGTH = 10.5 " min
GAGE DISTANCE (G) = 2.25"max/1.75 "min
EDGE DISTANCE (e') = 3.6 " min
ECCENTRICITY (Ex) = 0.6 " max
NUMBER OF BOLTS = 1

My : NOT REQUIRED

ANGLE: L6x4x3/4

BOLTS: HKB & KSKB
T' = 8.18 kips
S' = 7.136 kips

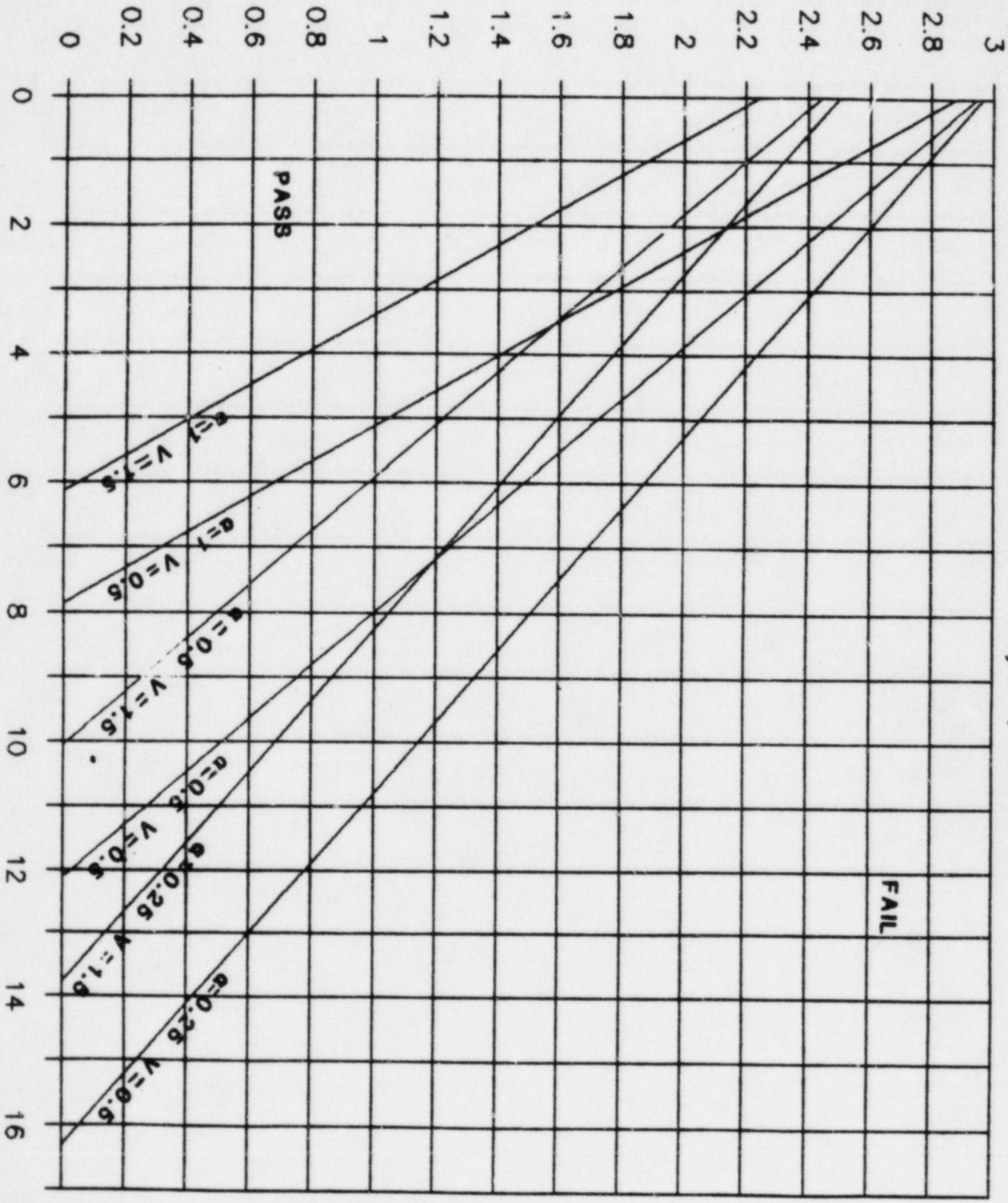


FIGURE A.21 TENSION FORCE vs MOMENT OBE

Rev. 2 May 30,1986

FIGURE A.22 TENSION FORCE vs MOMENT SSE

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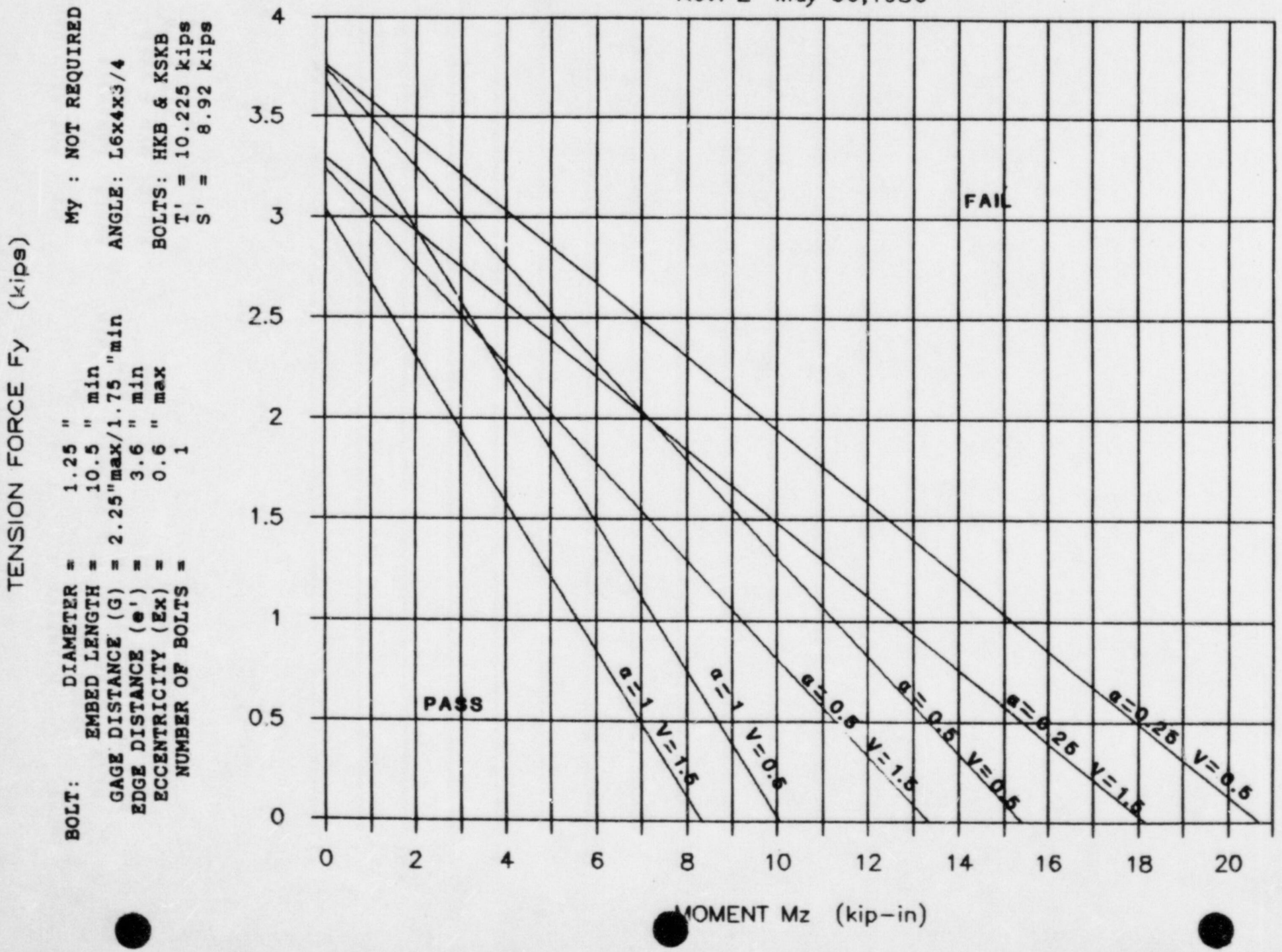

TENSION FORCE Fy (kips)

BOLT: DIAMETER = 1.25 "
EMBED LENGTH = 10.5 " min
C TO C SPACING (dx) = 26"max/16" min
GAGE DISTANCE (G) = 2.25"max/1.75"min
EDGE DISTANCE (e') = 1.75 " min
ECCENTRICITY (Ex) = 1.6 " max
NUMBER OF BOLTS = 2

My ≤ 7.739 kip-in

ANGLE: L6x4x3/4

BOLTS: HKB & HSKB
T' = 8.18 kips
S' = 7.136 kips

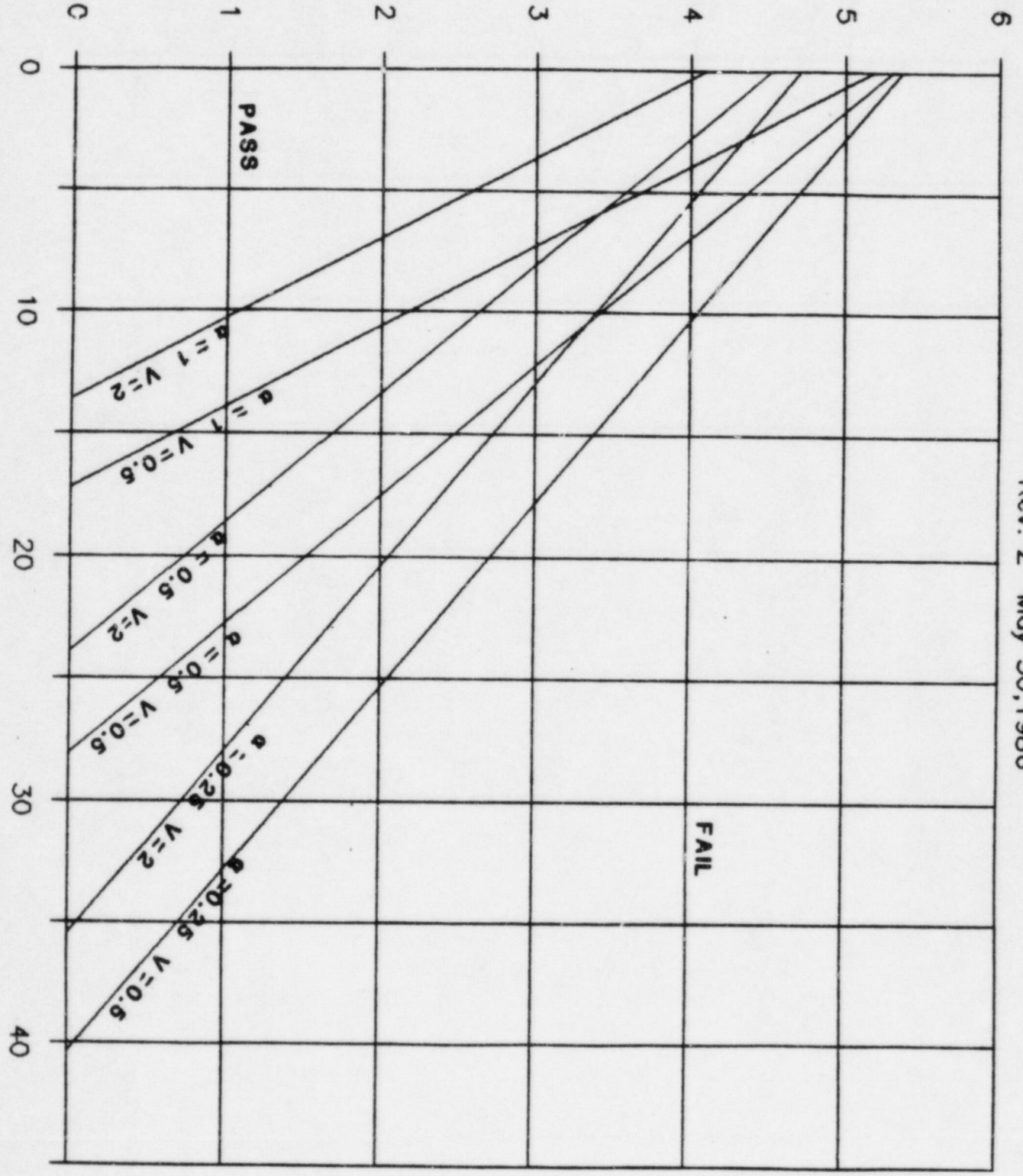

FIGURE A.23 TENSION FORCE vs MOMENT OBE

Rev. 2 May 30,1986

TENSION FORCE Fy (kips)

BOLT: DIAMETER =	1.25 "	My ≤ 7.739 kip-in
EMBED LENGTH =	10.5 " min	
C TO C SPACING (dx) =	26"max/16" min	ANGLE: L6x4x3/4
GAGE DISTANCE (G) =	2.25"max/1.75"min	
EDGE DISTANCE (e') =	1.75 " min	BOLTS: HKB & HSKB
ECCENTRICITY (Ex) =	1.6 " max	T' = 10.225 kips
NUMBER OF BOLTS =	2	S' = 8.92 kips

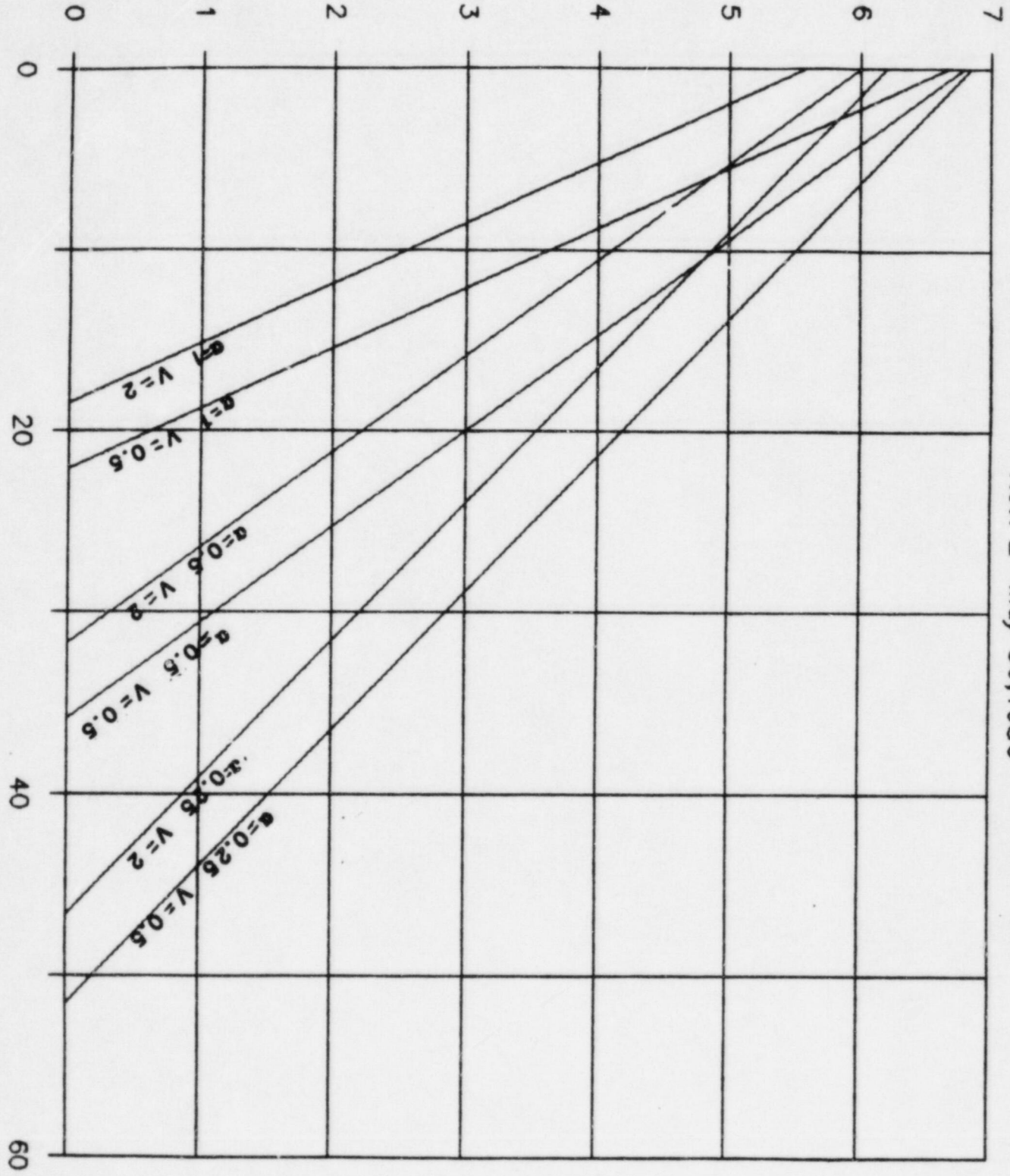

FIGURE A.24 TENSION FORCE vs MOMENT SSE

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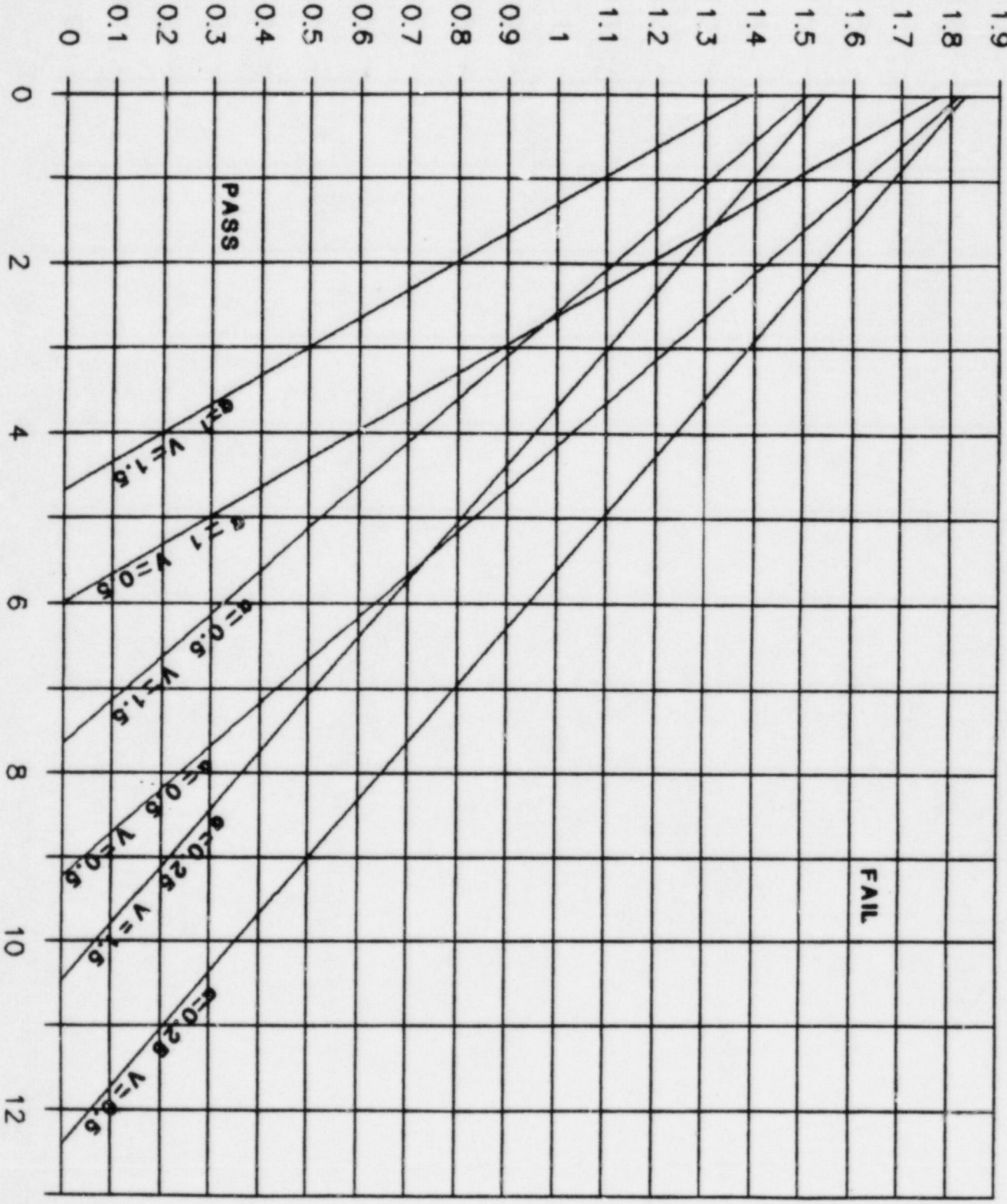
TENSION FORCE Fy (kips)

BOLT:	DIAMETER =	1.25 "	
	EMBED LENGTH =	7.5 " min	
	GAGE DISTANCE (G) =	3.25"max/1.75 "min	
	EDGE DISTANCE (e') =	3.6 " min	
	ECCENTRICITY (Ex) =	0.6 " max	
	NUMBER OF BOLTS =	1	

My : NOT REQUIRED

ANGLE: L5x5x3/4

BOLTS: HKB & KSKB
T' = 6.22 kips
S' = 7.136 kips



MOMENT Mz (kip-in)

FIGURE A.25 TENSION FORCE vs MOMENT OBE

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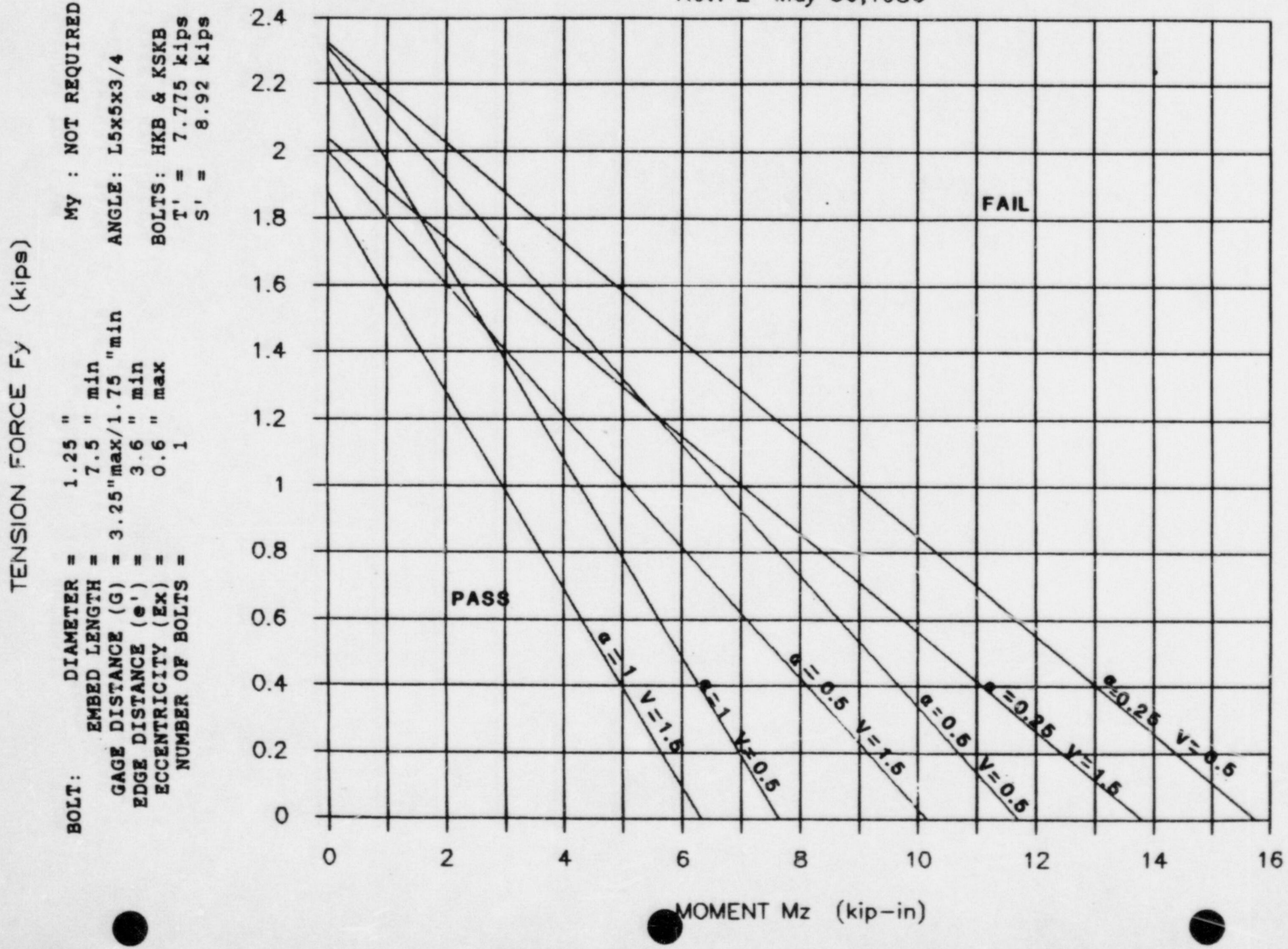
FIGURE A.26 TENSION FORCE vs MOMENT SSE

Rev. 2 May 30,1986



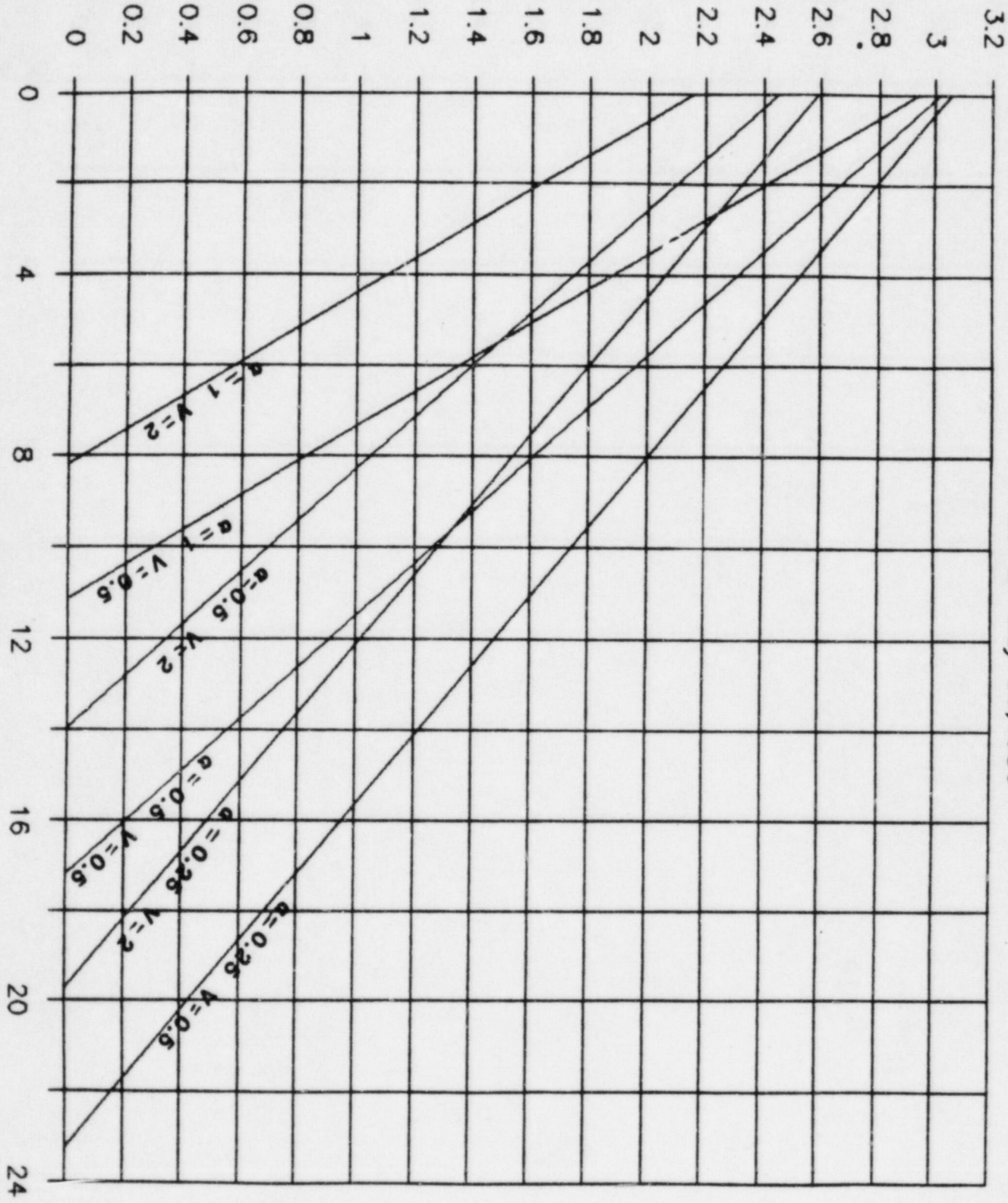
TENSION FORCE Fy (kips)

BOLT: DIAMETER = 1.25 "
EMBED LENGTH = 7.5 " min
C TO C SPACING (dx) = 21"max/13" min
GAGE DISTANCE (G) = 3.25"max/1.75"min
EDGE DISTANCE (e') = 1.75 " min
ECCENTRICITY (Ex) = 1.6 " max
NUMBER OF BOLTS = 2

My ≤ 7.739 kip-in

ANGLE: L5x5x3/4

BOLTS: HKB & HSKB
T' = 6.22 kips
S' = 7.136 kips



MOMENT Mz (kip-in)

FIGURE A.27 TENSION FORCE vs MOMENT OBE

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TENSION FORCE Fy (kips)

BOLT: DIAMETER = 1.25 "
EMBED LENGTH = 7.5 " min
C TO C SPACING (dx) = 21"max/13" min
GAGE DISTANCE (G) = 3.25"max/1.75"min
EDGE DISTANCE (e') = 1.75 " min
ECCENTRICITY (Ex) = 1.6 " max
NUMBER OF BOLTS = 2

My ≤ 7.739 kip-in

ANGLE: L5x5x3/4

BOLTS: HKB & HSKB
T' = 7.775 kips
S' = 8.92 kips

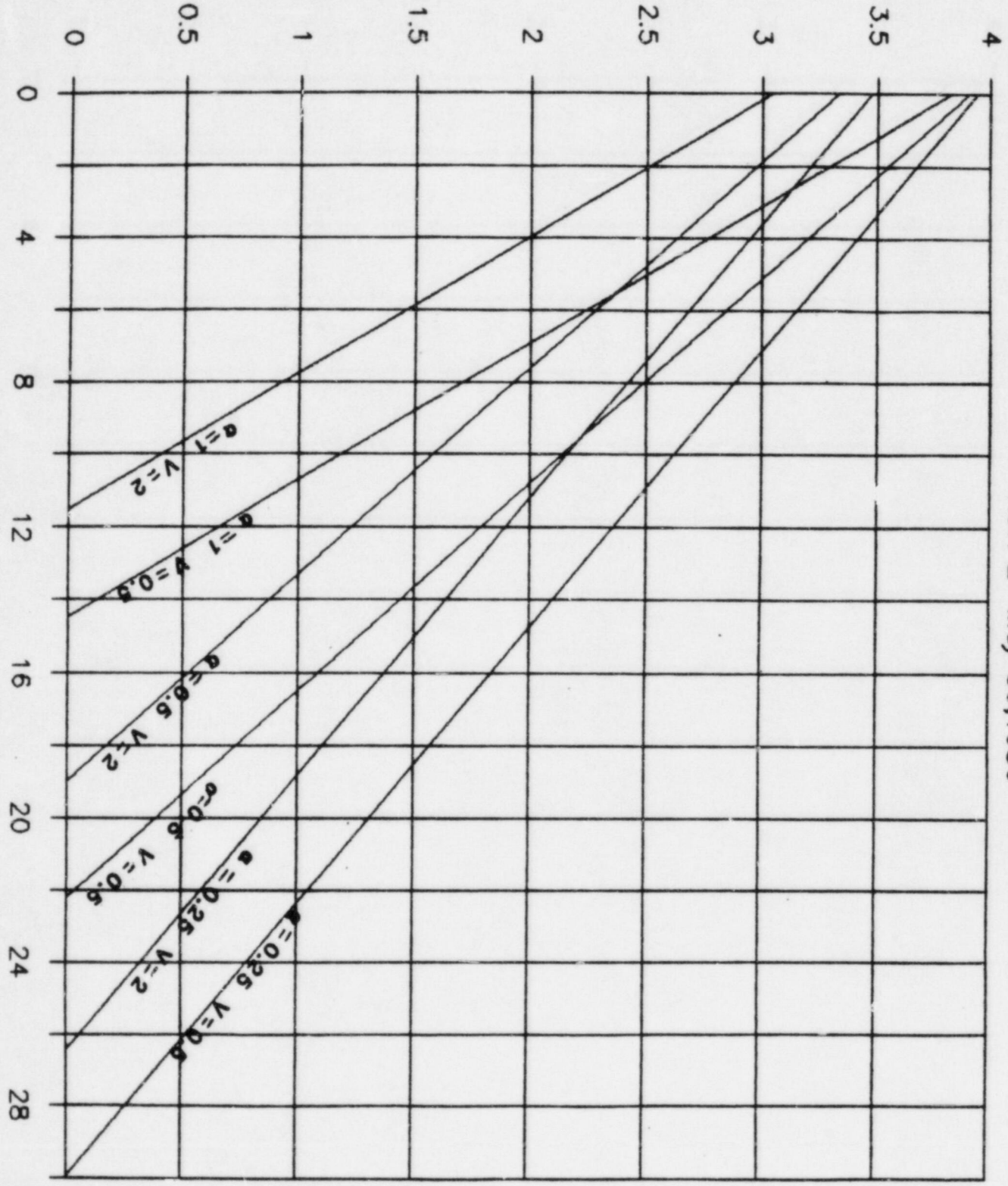

FIGURE A.28 TENSION FORCE vs MOMENT SSE

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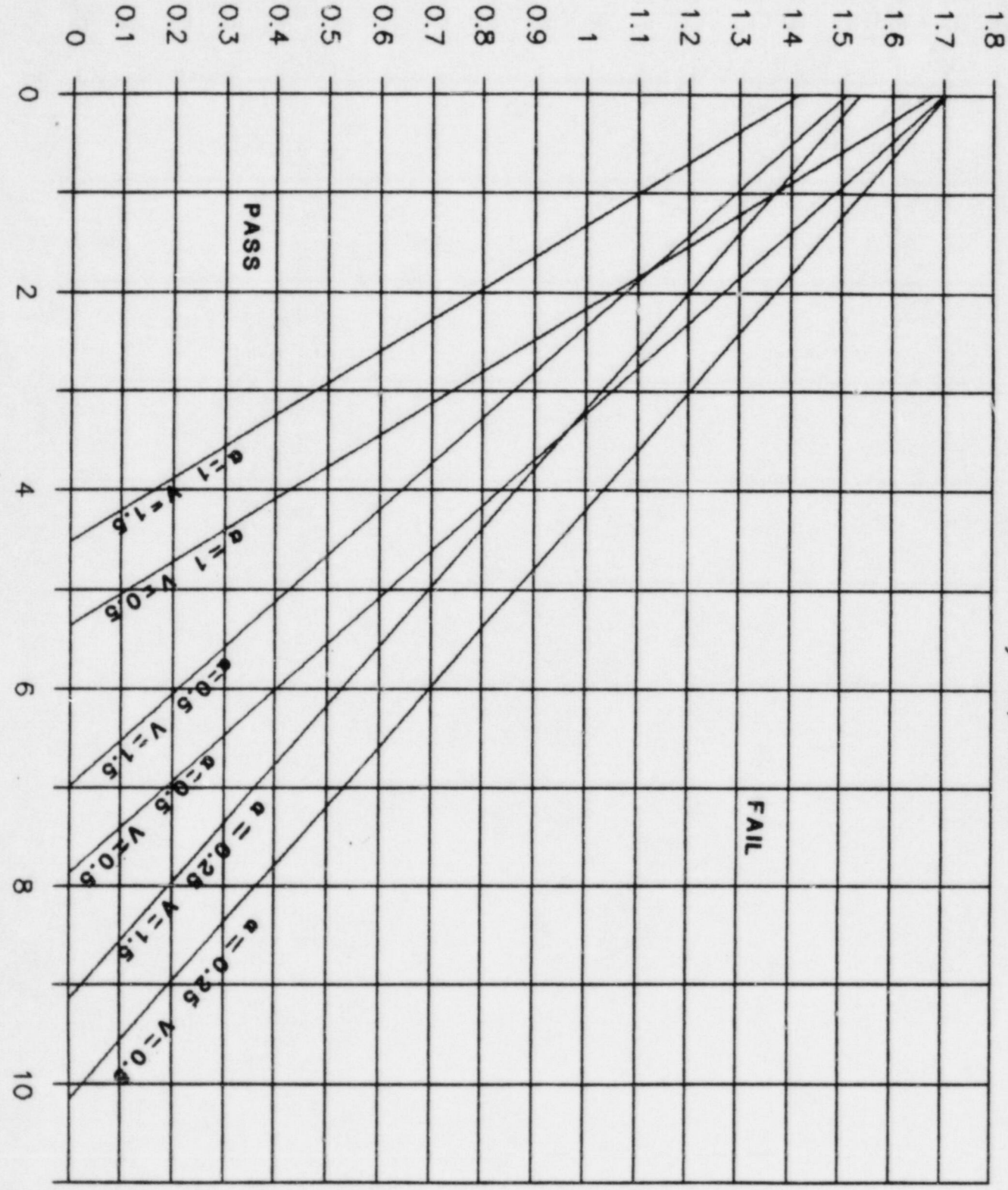
TENSION FORCE Fy (kips)

BOLT: DIAMETER = 1 "
CONCRETE THICKNESS = 10 " min
GAGE DISTANCE (G) = 3.25"max/1.75"min
EDGE DISTANCE (e') = 3.6 " min
ECCENTRICITY (Ex) = 0.6 " max
NUMBER OF BOLTS = 1

My : NOT REQUIRED

ANGLE: L5x5x3/4

BOLTS: R.I. (A-325)
T' = 6 kips
S' = 6 kips



MOMENT Mz (kip-in)

FIGURE A.29 TENSION FORCE vs MOMENT OBE

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FIGURE A.30 TENSION FORCE vs MOMENT SSE

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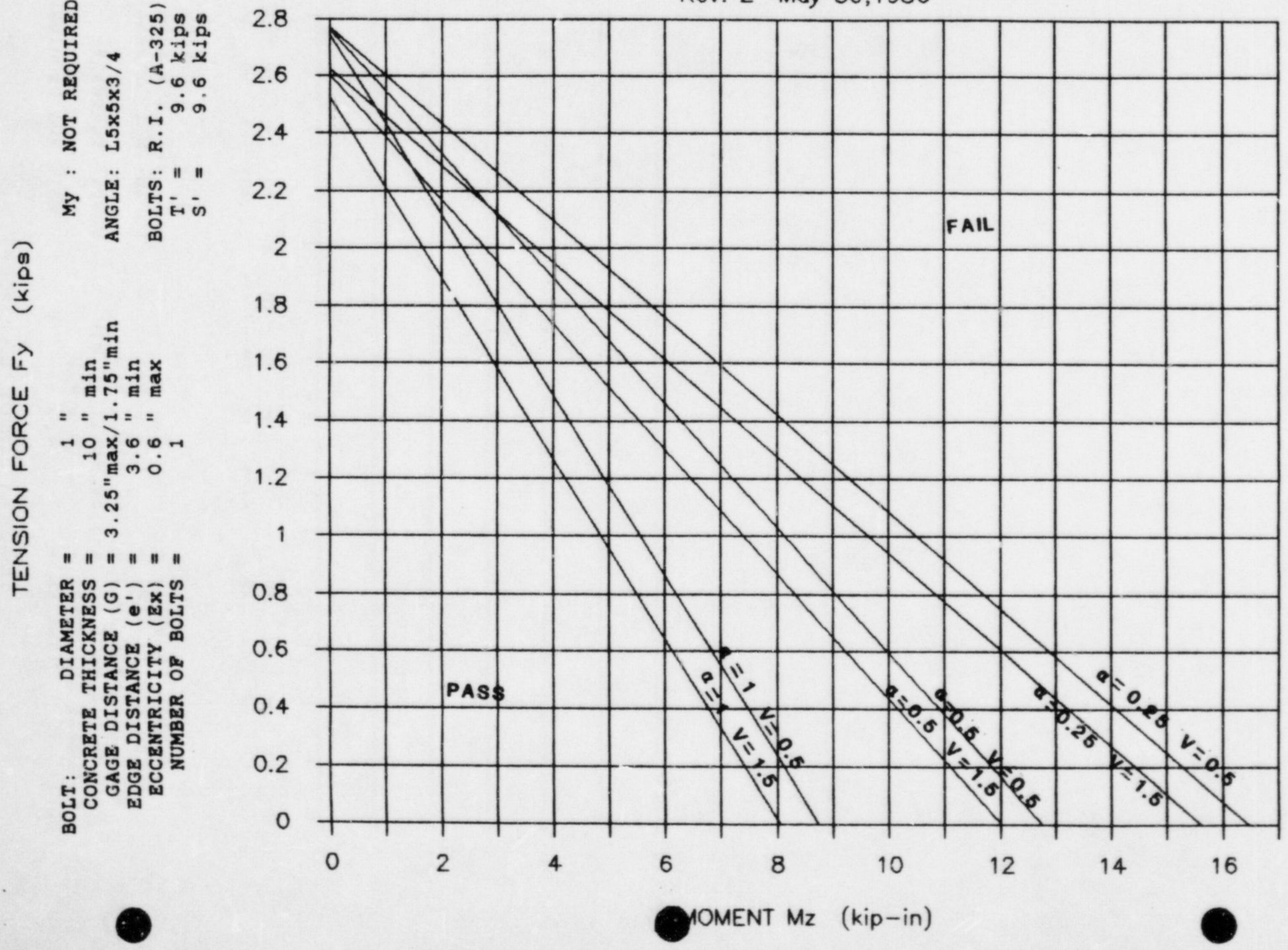


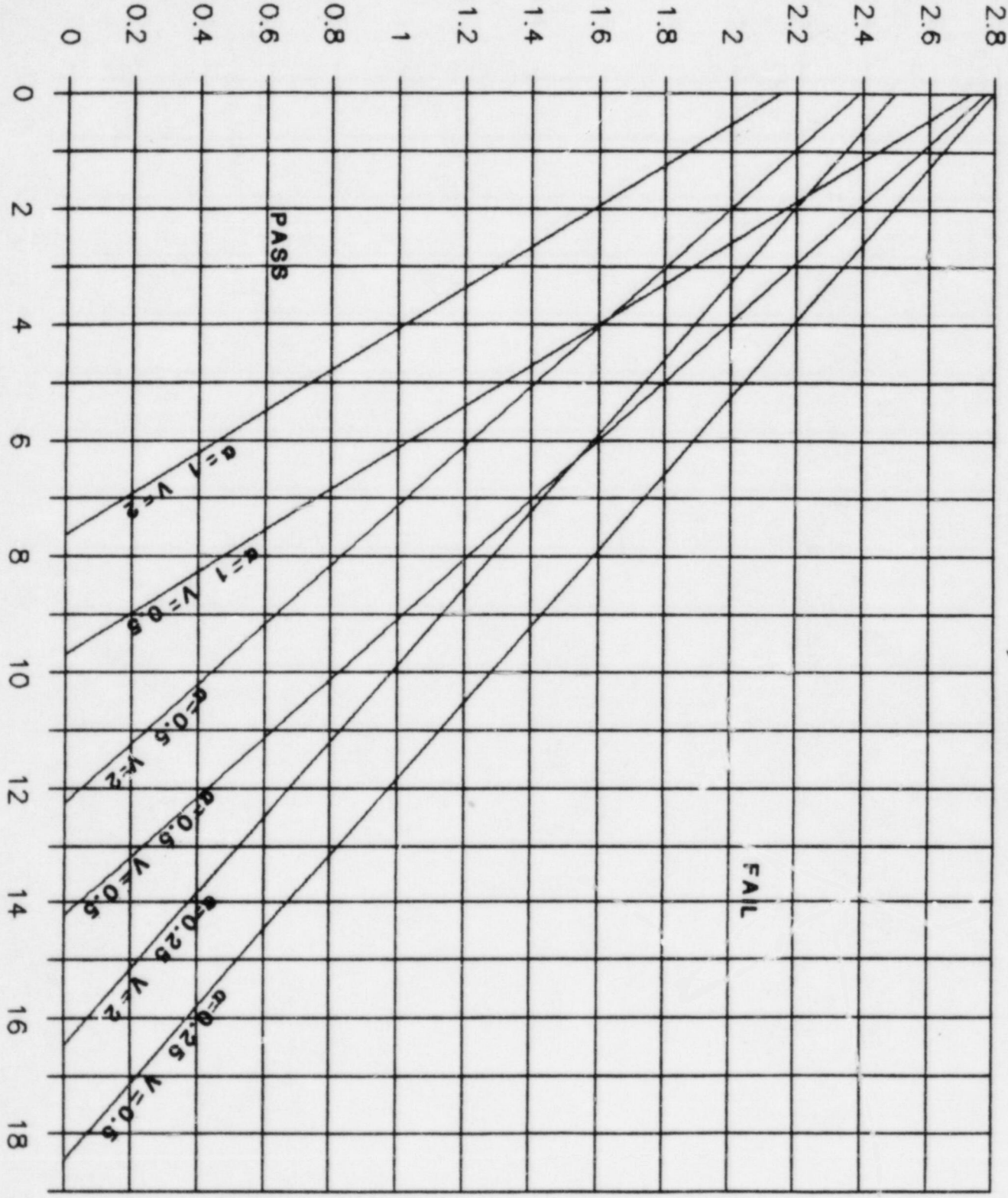
TENSION FORCE Fy (kips)

BOLT: DIAMETER = 1 "
CONCRETE THICKNESS = 10 " min
C TO C SPACING (dx) = 21"max/13" min
GAGE DISTANCE (G) = 3.25"max/1.75"min
EDGE DISTANCE (e') = 1.75 " min
ECCENTRICITY (Ex) = 1.6 " max
NUMBER OF BOLTS = 2

My ≤ 7.739 kip-in

ANGLE: L5x5x3/4

BOLTS: R.I. (A-325)
T' = 6 kips
S' = 6 kips



MOMENT Mz (kip-in)

FIGURE A.31 TENSION FORCE vs MOMENT OBE

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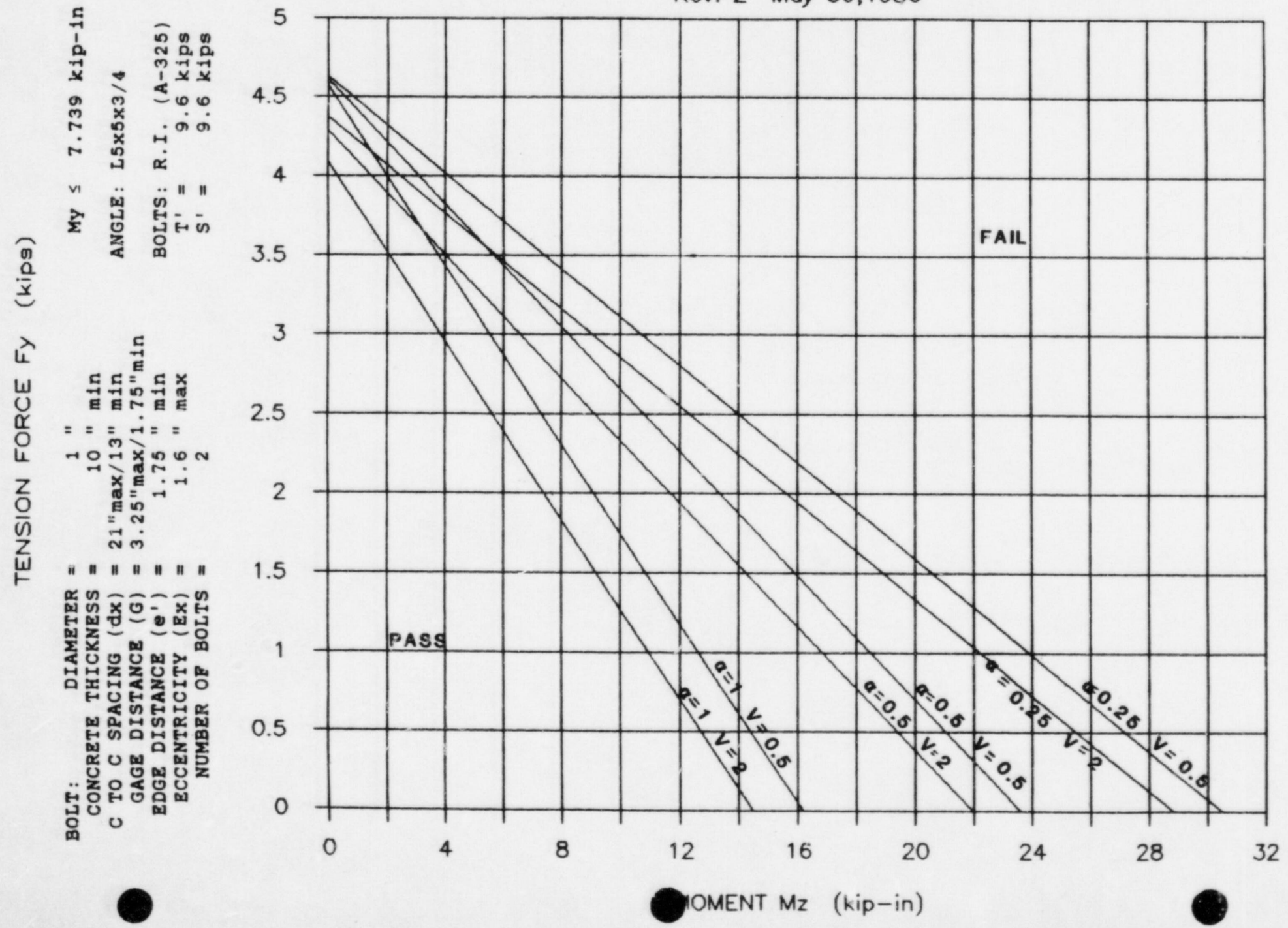
FIGURE A.32 TENSION FORCE vs MOMENT SSE

Rev. 2 May 30,1986



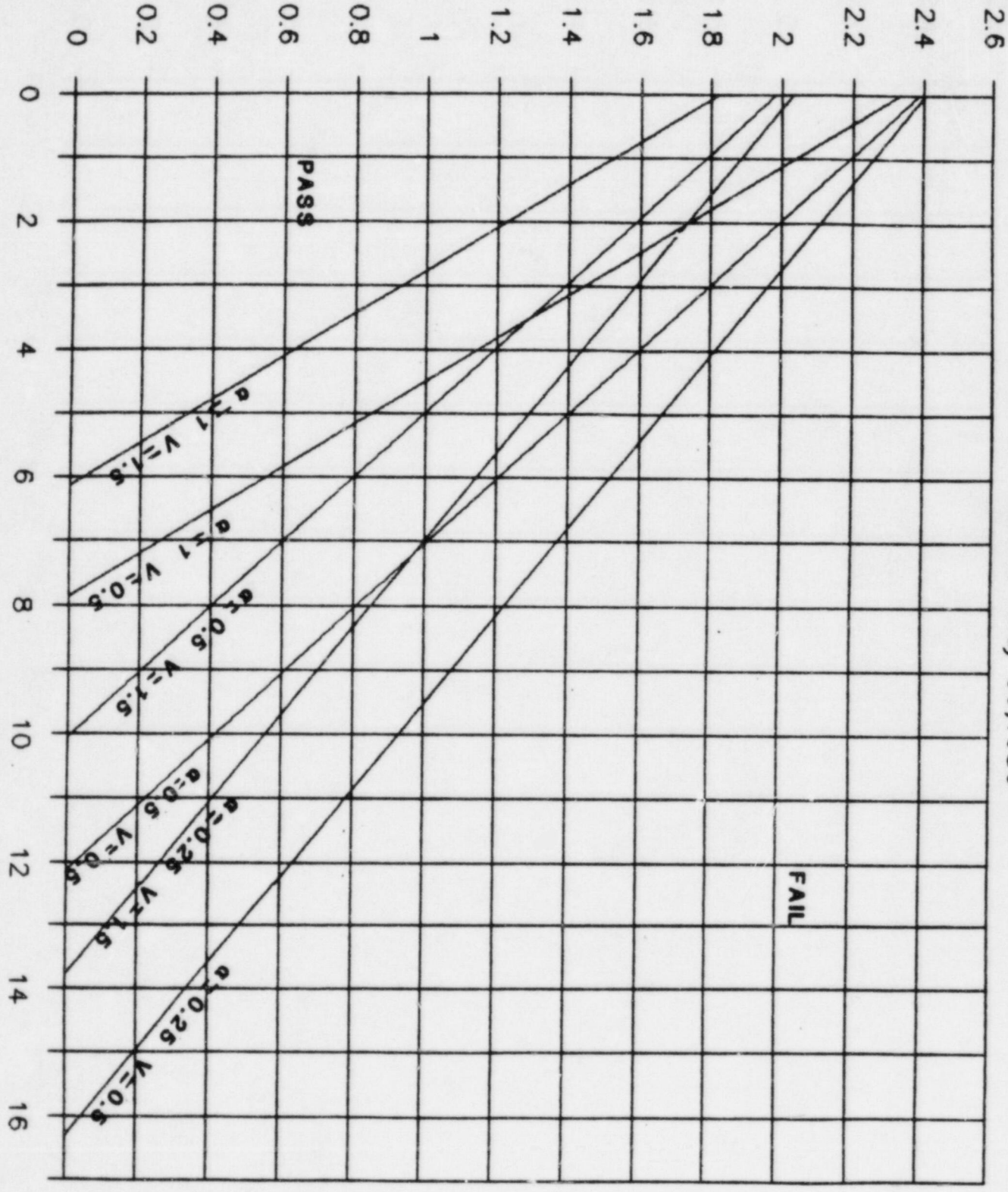
TENSION FORCE Fy (kips)

BOLT: DIAMETER = 1.25 "
EMBED LENGTH = 10.5 " min
GAGE DISTANCE (G) = 3.25"max/1.75 "min
EDGE DISTANCE (e') = 3.6 " min
ECCENTRICITY (Ex) = 0.6 " max
NUMBER OF BOLTS = 1

My : NOT REQUIRED

ANGLE: L5x5x3/4

BOLTS: HKB & KSKB
T' = 8.18 kips
S' = 7.136 kips



MOMENT Mz (kip-in)

FIGURE A.33 TENSION FORCE vs MOMENT OBE

Rev. 2 May 30,1986

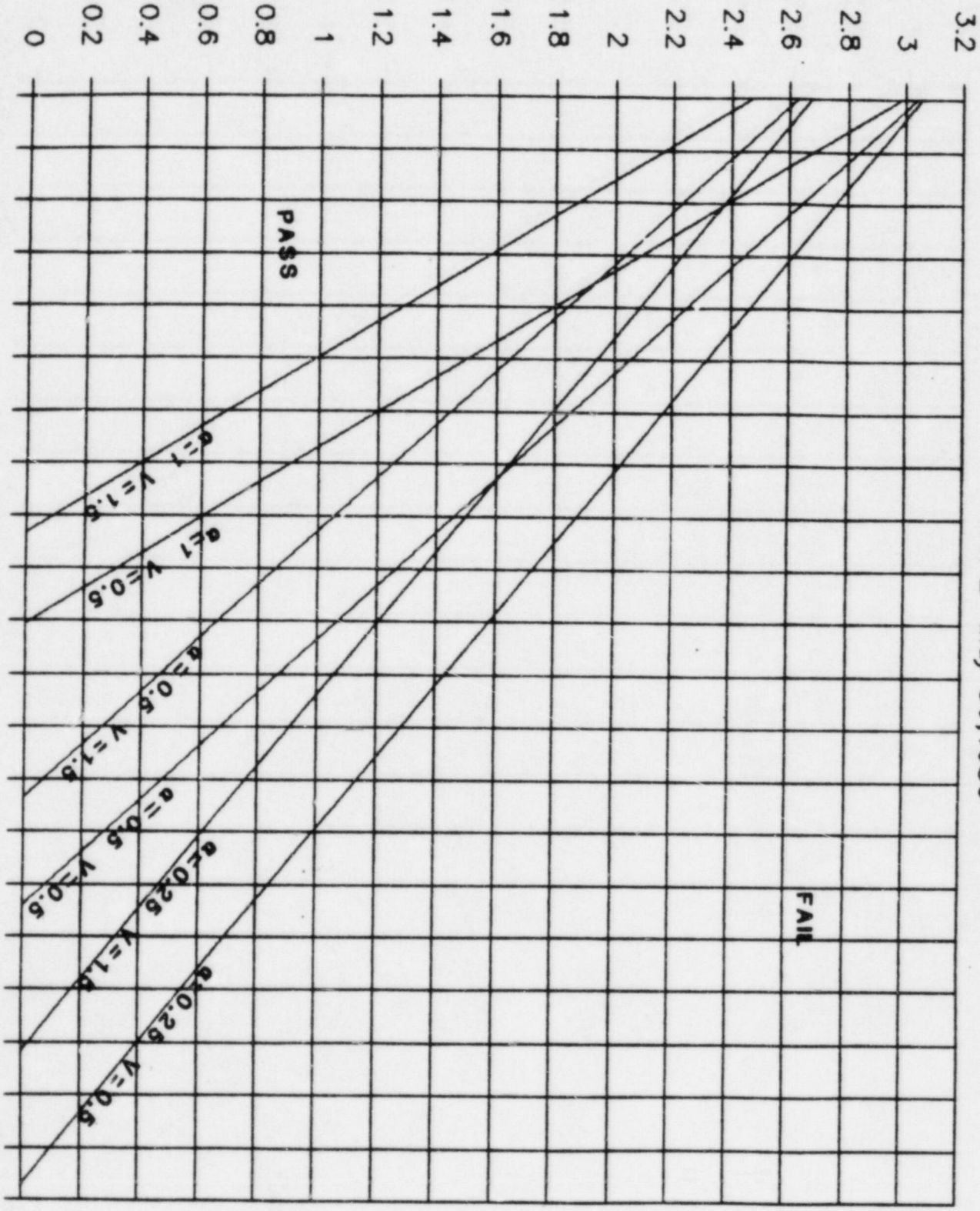
TENSION FORCE Fy (kips)

BOLT: DIAMETER = 1.25 "
EMBED LENGTH = 10.5 " min
GAGE DISTANCE (G) = 3.25"max/1.75 "min
EDGE DISTANCE (e') = 3.6 " min
ECCENTRICITY (Ex) = 0.6 " max
NUMBER OF BOLTS = 1

My : NOT REQUIRED

ANGLE: L5x5x3/4

BOLTS: HKB & KSKB
T' = 10.225 kips
S' = 8.92 kips



MOMENT Mz (kip-in)

FIGURE A.34 TENSION FORCE vs MOMENT SSE

Rev. 2 May 30,1986

TENSION FORCE Fy (kips)

BOLT:	DIAMETER =	1.25 "	My ≤ 7.739 kip-in
	EMBED LENGTH =	10.5 " min	
	C TO C SPACING (dx) =	21"max/13" min	ANGLE: L5x5x3/4
	GAGE DISTANCE (G) =	3.25"max/1.75"min	
	EDGE DISTANCE (e') =	1.75 " min	BOLTS: HKB & HSKB
	ECCENTRICITY (Ex) =	1.6 " max	T' = 8.18 kips
	NUMBER OF BOLTS =	2	S' = 7.136 kips

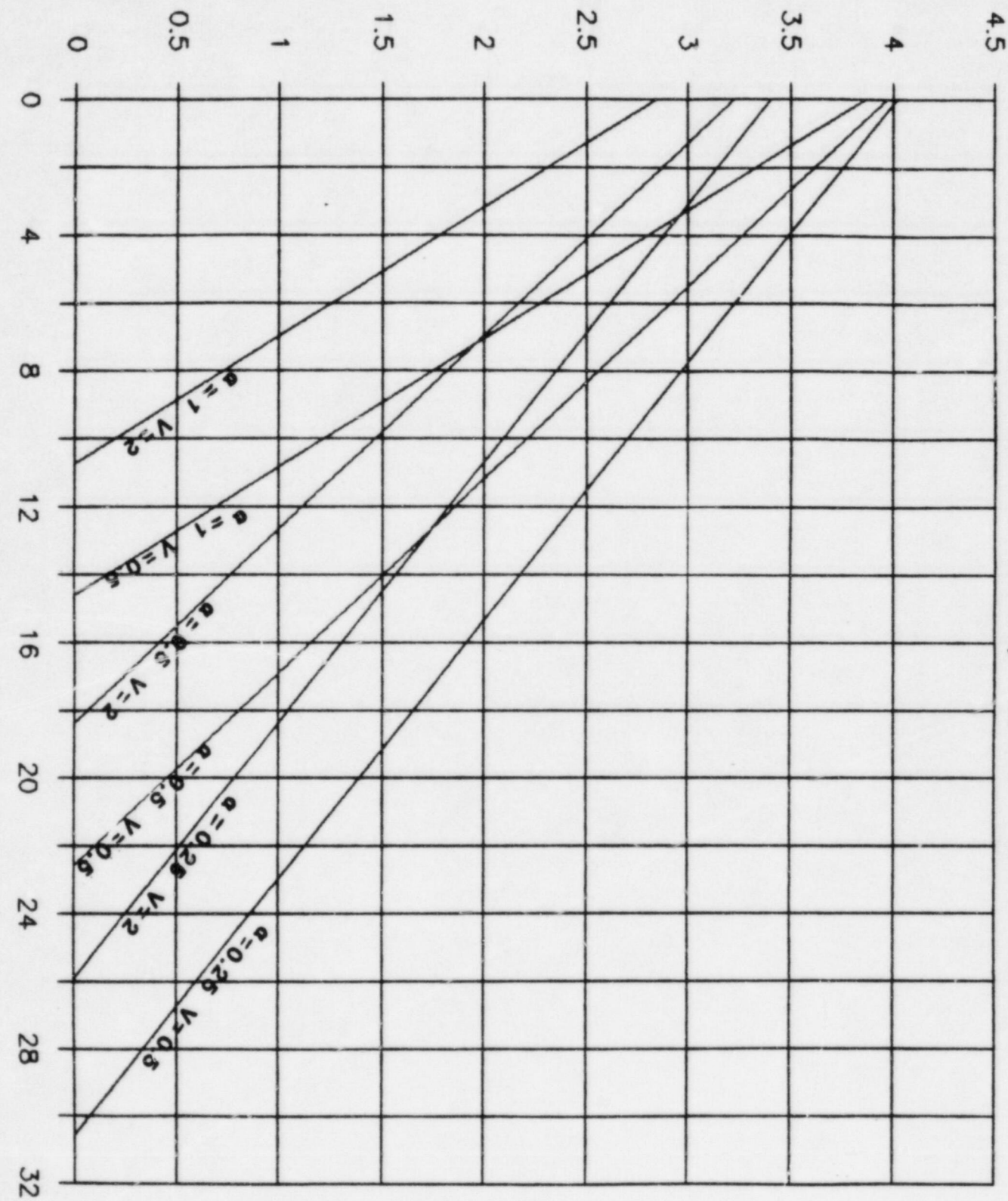


FIGURE A.35 TENSION FORCE vs MOMENT OBE

Rev. 2 May 30,1986

TENSION FORCE Fy (kips)

BOLT: DIAMETER =	1.25 "	My ≤ 7.739 kip-in
EMBED LENGTH =	10.5 " min	
C TO C SPACING (dx) =	21"max/13" min	ANGLE: L5x5x3/4
GAGE DISTANCE (G) =	3.25"max/1.75"min	
EDGE DISTANCE (e') =	1.75 " min	BOLTS: HKB & HSKB
ECCENTRICITY (Ex) =	1.6 " max	T' = 10.225 kips
NUMBER OF BOLTS =	2	S' = 8.92 kips

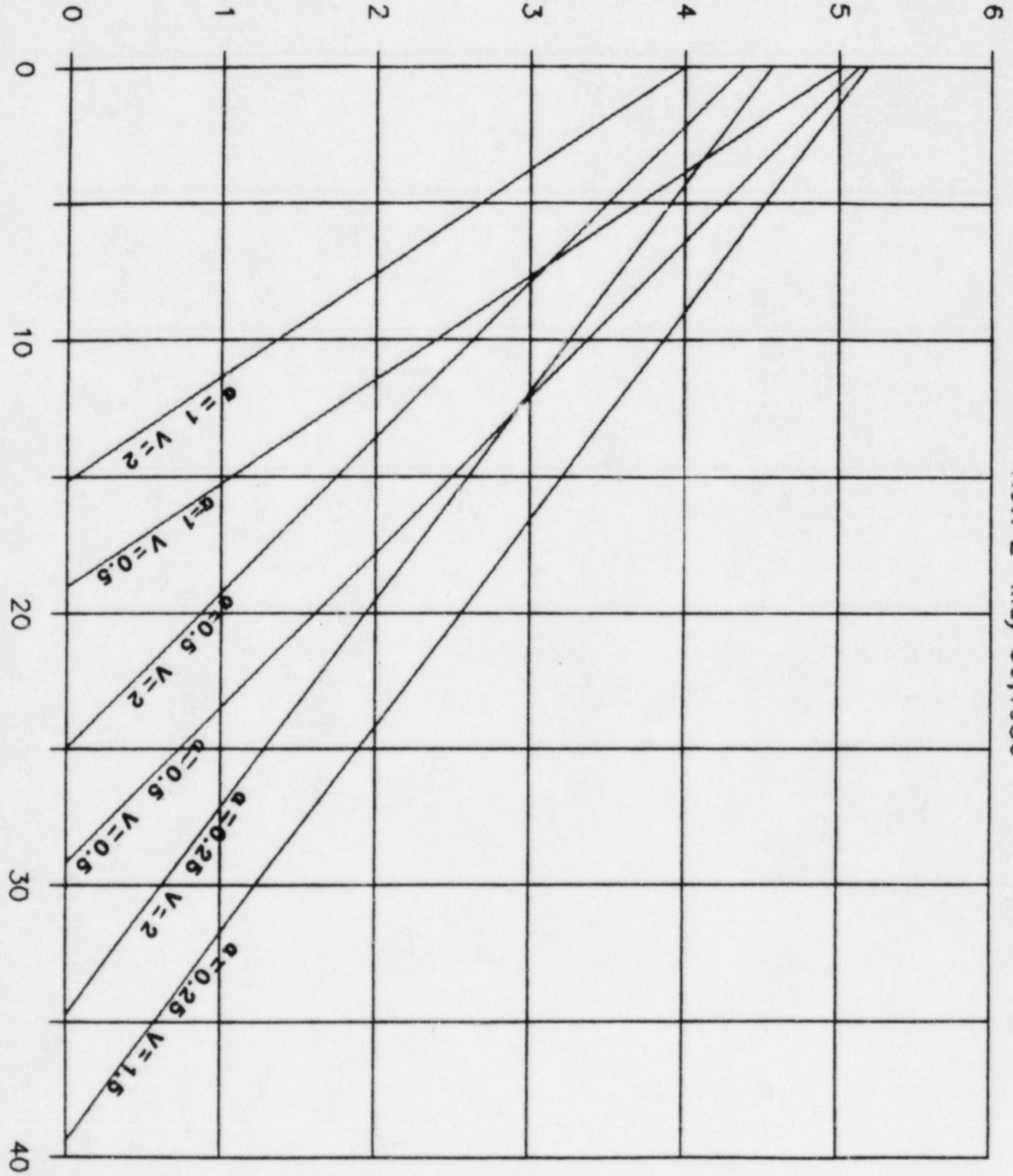


FIGURE A.36 TENSION FORCE vs MOMENT SSE

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FIGURE A.37 TENSION FORCE vs MOMENT OBE

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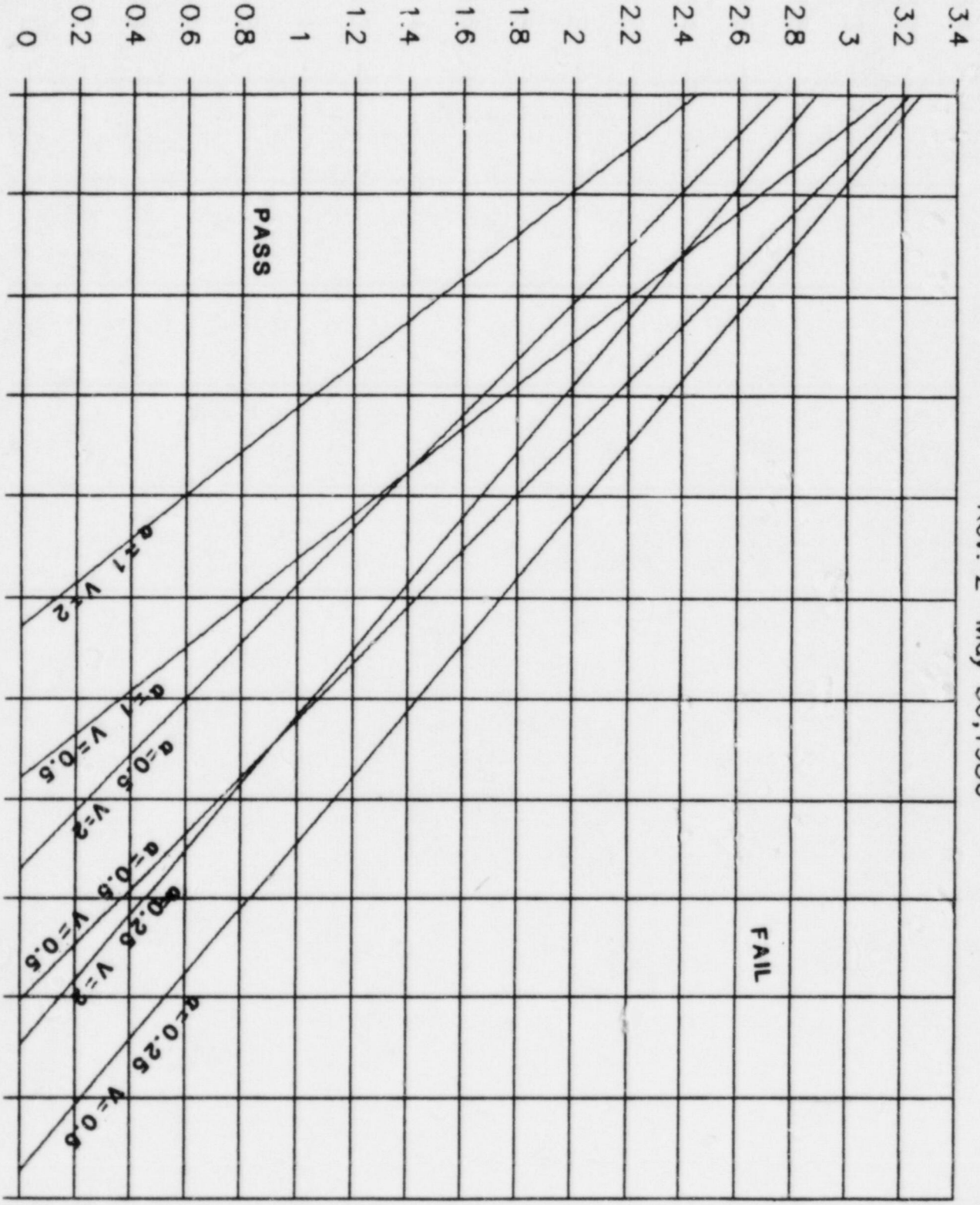
TENSION FORCE Fy (kips)

BOLT:	DIAMETER =	1.25 "	
	EMBED LENGTH =	7.5 "	min
C TO C SPACING (dx) =		21"max/13"	min
GAGE DISTANCE (G') =		3.5 "	min
EDGE DISTANCE (e') =		1.75 "	min
ECCENTRICITY (Ex) =		1.6 "	max
ECCENTRICITY (Ez) =		1.3 "	max
NUMBER OF BOLTS =		2	

My ≤ 7.739 kip-in

PLATE: 1x8

BOLTS: HKB & HSKB
T' = 6.22 kips
S' = 7.136 kips



MOMENT Mz (kip-in)

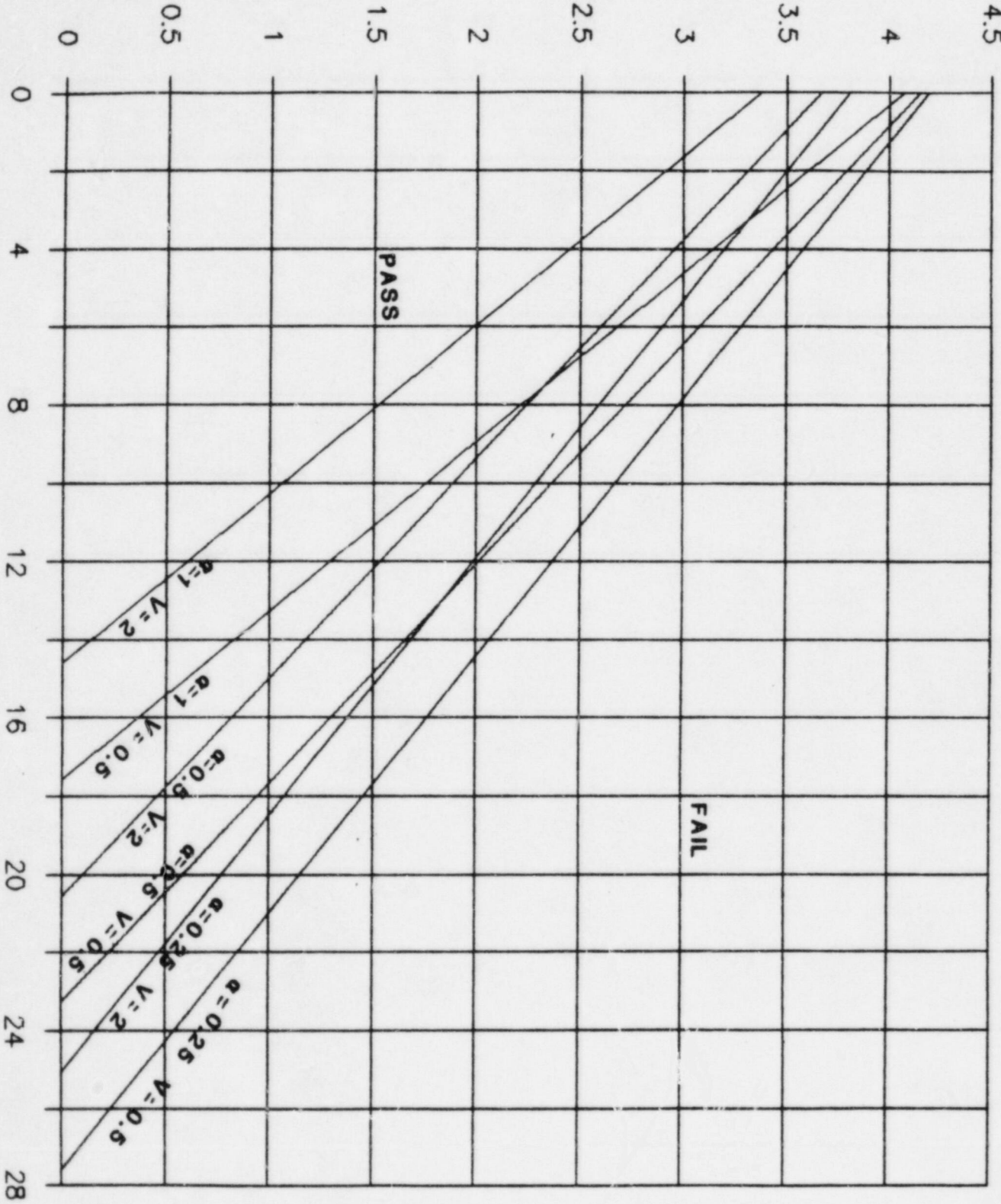
TENSION FORCE Fy (kips)

BOLT: DIAMETER	=	1.25 "	
EMBED LENGTH	=	7.5 "	min
C TO C SPACING (dx)	=	21"max/13"	min
GAGE DISTANCE (G')	=	3.5 "	min
EDGE DISTANCE (e')	=	1.75 "	min
ECCENTRICITY (Ex)	=	1.6 "	max
ECCENTRICITY (Ez)	=	1.3 "	max
NUMBER OF BOLTS	=	2	

My ≤ 7.739 kip-in

PLATE: 1x8

BOLTS: HKB & HSKB
T' = 7.775 kips
S' = 8.92 kips



MOMENT Mz (kip-in)

FIGURE A.38 TENSION FORCE vs MOMENT SSE

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TENSION FORCE Fy (kips)

BOLT:	DIAMETER =	1 "	
	CONCRETE THICKNESS =	10 "	min
	C TO C SPACING (dx) =	21"max/13"	min
	GAGE DISTANCE (G') =	3.5 "	min
	EDGE DISTANCE (e') =	1.75 "	min
	ECCENTRICITY (Ex) =	1.6 "	max
	ECCENTRICITY (Ez) =	1.3 "	max
	NUMBER OF BOLTS =	2	

My ≤ 7.739 kip-in

PLATE: 1x8

BOLTS: R.I. (A-325)
T' = 6 kips
S' = 6 kips

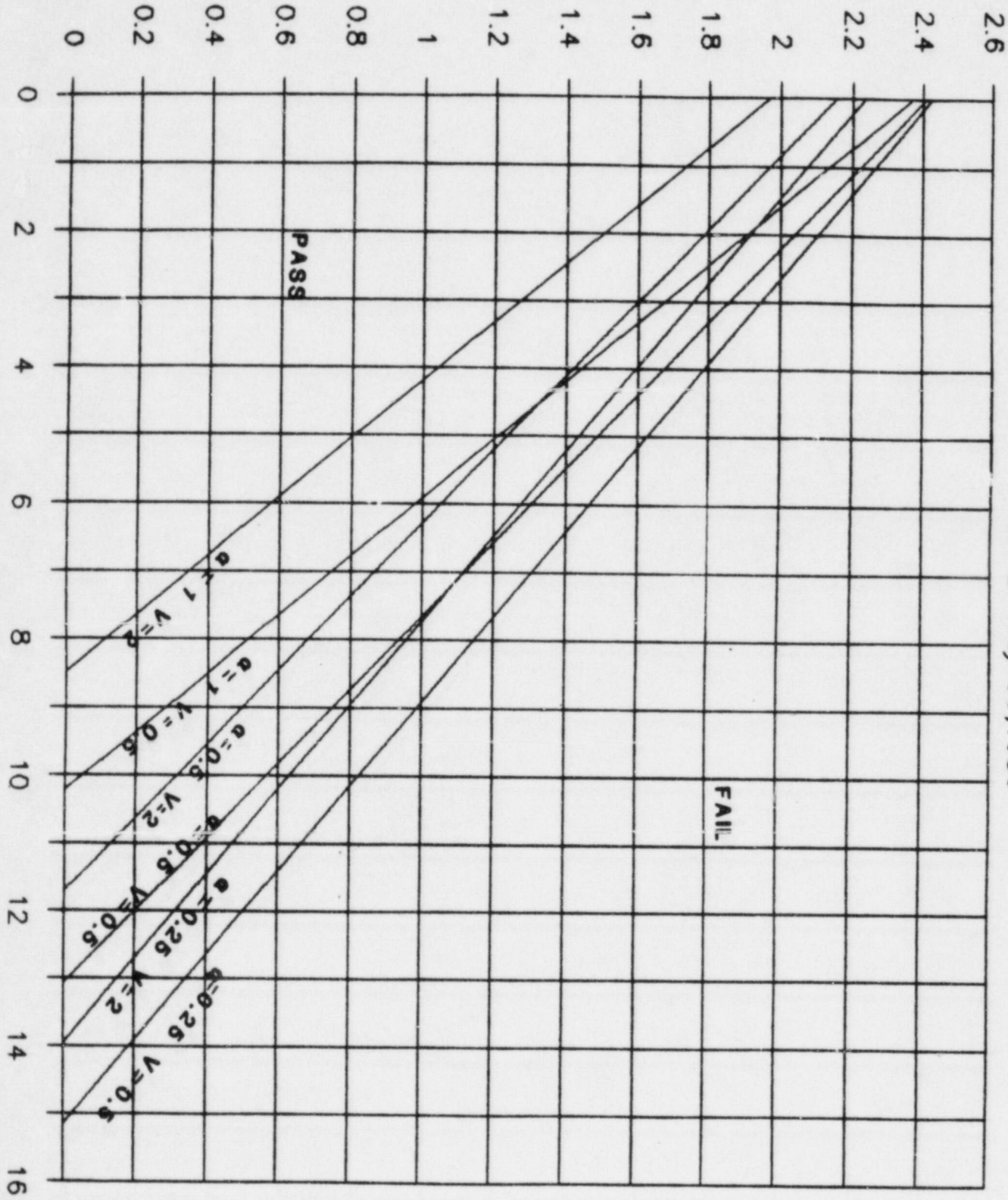


FIGURE A.39 TENSION FORCE vs MOMENT OBE

Rev. 2 May 30,1986

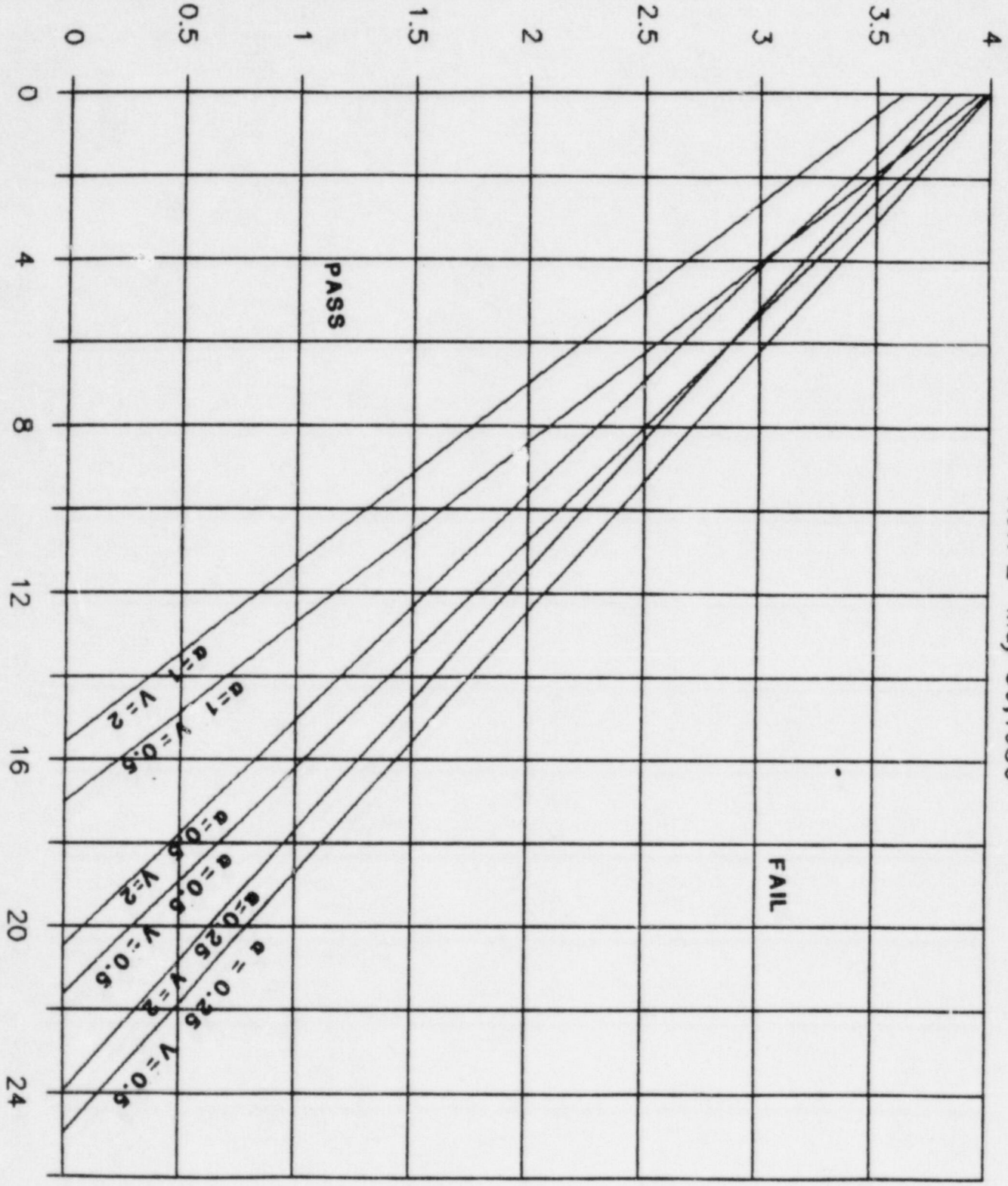
TENSION FORCE Fy (kips)

BOLT: DIAMETER =	1 "	
CONCRETE THICKNESS =	10 "	min
C TO C SPACING (dx) =	21"max/13"	min
GAGE DISTANCE (G') =	3.5 "	min
EDGE DISTANCE (e') =	1.75 "	min
ECCENTRICITY (Ex) =	1.6 "	max
ECCENTRICITY (Ez) =	1.3 "	max
NUMBER OF BOLTS =	2	

My ≤ 7.739 kip-in

PLATE: 1x8

BOLTS: R.I. (A-325)
T' = 9.6 kips
S' = 9.6 kips



MOMENT Mz (kip-in)

FIGURE A.40 TENSION FORCE vs MOMENT SSE

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FIGURE A.41 TENSION FORCE vs MOMENT OBE

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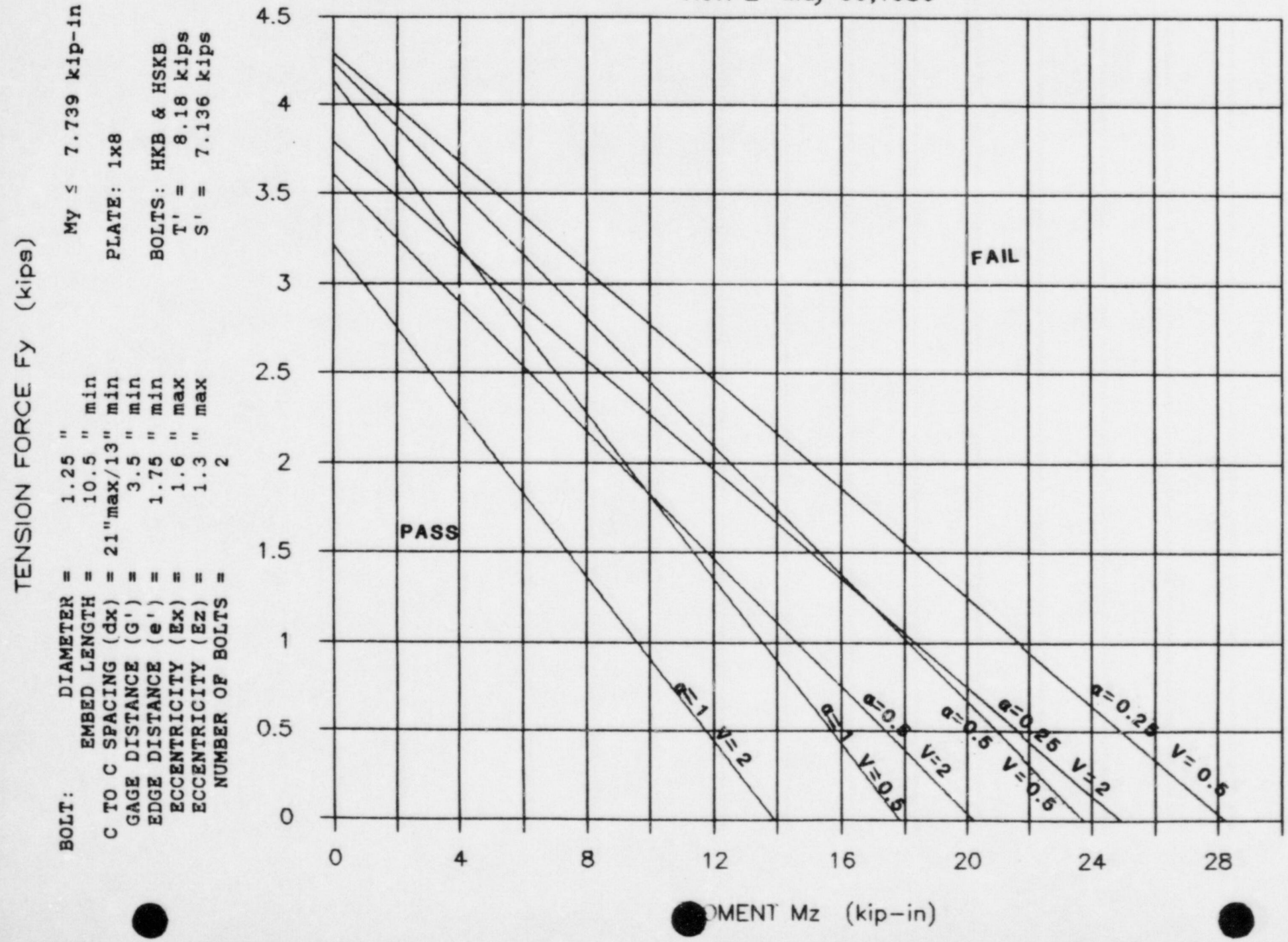


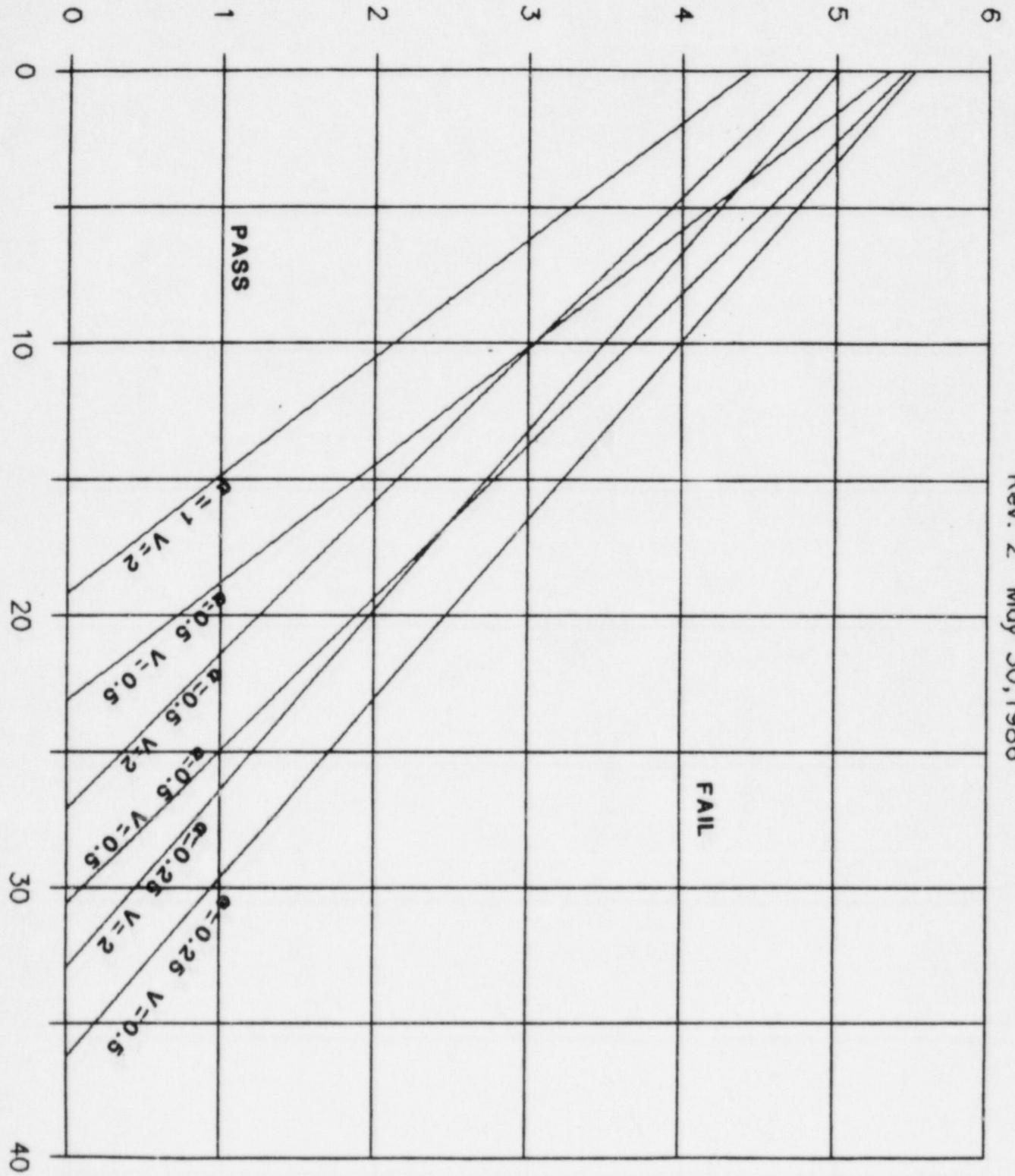
TENSION FORCE Fy (kips)

BOLT: DIAMETER =	1.25 "	
EMBED LENGTH =	10.5 "	min
C TO C SPACING (dx) =	21"max/13"	min
GAGE DISTANCE (G') =	3.5 "	min
EDGE DISTANCE (e') =	1.75 "	min
ECCENTRICITY (Ex) =	1.6 "	max
ECCENTRICITY (Ez) =	1.3 "	max
NUMBER OF BOLTS =	2	

My ≤ 7.739 kip-in

PLATE: 1x8

BOLTS: HKB & HSKB
T' = 10.225 kips
S' = 8.92 kips



MOMENT Mz (kip-in)

FIGURE A.42 TENSION FORCE vs MOMENT SSE

Rev. 2 May 30,1986

Attachment B
Bolt/Insert Capacities*

Table	Title	Reference
B.1	Kwik-Bolt Design Allowable Tensile and Shear Loads	3.1.1, Appendix 2
B.2	Super Kwik-bolt Design Allowable Tensile and Shear Loads	3.1.1, Appendix 2
B.3	OBE Allowable Loads For Richmond Inserts Attached To Beam Sides	3.1.1, Appendix 3
B.4	OBE Allowable Loads For 1" Richmond Inserts Attached to Walls, Slabs and Columns	3.1.1, Appendix 3
B.5	OBE Allowable Loads for 1 1/2" Richmond Inserts Attached To Walls, Slabs and Columns	3.1.1, Appendix 3
B.6	SSE Allowable Loads For Richmond Inserts Attached To Beam Sides	3.1.1, Appendix 3
B.7	SSE Allowable Loads For Richmond Inserts Attached To Walls, Slabs and Columns	3.1.1, Appendix 3
B.8	SSE Allowable Loads For Richmond Inserts Attached To Walls, Slabs and Columns	3.1.1, Appendix 3

* Note that all capacities given in these tables require evaluation with regard to the minimum spacing and clearance distances given in Attachment C to this instruction.

TABLE B.1

KWIK-BOLT
DESIGN ALLOWABLE TENSILE & SHEAR LOADS (lbs)

Factor of Safety		FS = 4.0 (SSE)		FS = 5.0 (OBE)	
DIAMETER	EMBEDMENT	TENSION	SHEAR	TENSION	SHEAR
3/4"	3 1/4"	2537	4183	2030	3426
	4"	3350	4283	2680	3426
	5"	4125	4283	3300	3426
	6"	4500	4616	3600	3693
	7"	5250	4616	4200	3693
	8"	5750	4616	4600	3693
	9"	5875	4616	4700	3693
1"	4 1/2"	4000	6719	3200	5375
	5"	4725	6719	3780	5375
	6"	5860	6719	4688	5375
	7"	5860	6719	4688	5375
	8"	5860	8622	4688	6898
	9"	5860	8622	4688	6898
	10"	5860	8622	4688	6898
1 1/4"	5 1/2"	5750	8920	4600	7136
	6 1/2"	6775	8920	5420	7136
	7 1/2"	7775	8920	6220	7136
	8 1/2"	8650	8920	6920	7136
	9 1/2"	9450	8920	7560	7136
	10 1/2"	10225	8920	8180	7136

Design allowables are based on 4000 psi concrete (fc = 4000 psi).

TABLE B.2

SUPER KWIK-BOLT
DESIGN ALLOWABLE TENSILE & SHEAR LOADS (lbs)

Factor of Safety		FS = 4.0 (SSE)		FS = 5.0 (OBE)	
DIAMETER	EMBEDMENT	TENSION	SHEAR	TENSION	SHEAR
1"	6 1/2"	8741	6884	6993	5507
	8 1/2"	12452	6884	9962	5507
	10 1/2"	12439	6854	9951	5507
1 1/4"	8 1/8"	10675	10369	8540	8295
	10 5/8"	13420	10369	10736	8295
	13 1/8"	16230	10369	12984	8295

Design allowables are based on 4000 psi concrete (fc = 4000 psi).

TABLE B.3

OBE ALLOWABLE (KIPS)

ALLOWABLE LOADS OF RICHMOND INSERTS AND BOLTS TO BE USED IN INTERACTION FORMULAS FOR BEAM SIDES

I = Insert Capacity B = Bolt Capacity T = Tension S = Shear

INSERT TYPE AND SIZE		Insert Spacing on 6" C/C One Way and 20" C/C or Greater Other Way				Insert Spacing on 8" C/C One Way and 20" C/C or Greater Other Way			
		A-307 Bolts or A-36 THD. Rods used with insert		A-325 or Better Bolts used with insert		A-307 Bolts or A-36 THD. Rods used with insert		A-325 or Better Bolts used with insert	
		T	S	T	S	T	S	T	S
1" (EC2W)	I	8.9	8.9	8.9	8.9	10.05	10.05	10.05	10.05
	B	12.11	7.85	24.23	11.78	12.11	7.85	24.23	11.78
1 1/2" (EC6W)	I	11.53	11.53	11.53	11.53	12.85	12.85	12.85	12.85
	B	28.11	17.67	56.21	26.51	28.11	17.67	56.21	26.51

TABLE B.4

OBE ALLOWABLE (KIPS)

ALLOWABLE LOADS OF 1" RICHMOND INSERTS (EC2W) AND BOLTS TO BE USED IN INTERACTION FORMULAS FOR WALLS, SLABS AND COLUMNS

I = Insert Capacity B = Bolt Capacity T = Tension S = Shear

Concrete Thickness		Insert Spacing on 10" CC Both Ways				Insert Spacing on 12" Both Ways				Insert Spacing on 14" C/C or More Both Ways (Full Capacity)			
		A-307 Bolts or A-36 THD. Rods used with insert		A-325 or Better Bolts used with insert		A-307 Bolts or A-36 THD. Rods used with insert		A-325 or Better Bolts used with insert		A-307 Bolts or A-36 THD. Rods used with insert		A-325 or Better Bolts used with insert	
		T	S	T	S	T	S	T	S	T	S	T	S
10" or Thicker	I	6.00	6.00	6.00	6.00	8.85	8.85	8.85	8.85	11.5	11.5	11.5	11.5
	B	12.11	7.85	24.23	11.78	12.11	7.85	24.23	11.78	12.11	7.85	24.23	11.78

TABLE B.5

OBE ALLOWABLE

ALLOWABLE LOADS OF 1 1/2" RICHMOND INSERTS (EC6W) AND BOLTS TO BE USED IN INTERACTION FORMULAS FOR WALLS, SLABS AND COLUMNS

I = Insert Capacity B = Bolt Capacity T = Tension S = Shear

Concrete Thickness		Insert Spacing on 20" CC Both Ways				Insert Spacing on 22" C/C or More Both Ways (Full Capacity)			
		A-307 Bolts or A-36 THD. Rods used with insert		A-325 or Better Bolts used with insert		A-307 Bolts or A-36 THD. Rods used with insert		A-325 or Better Bolts used with insert	
		T	S	T	S	T	S	T	S
12" or Thicker	I	25	25	25	25	31.3	27	31.3	27
	B	28.11	17.67	56.21	26.51	28.11	17.67	56.21	26.51

TABLE B.6

SSE ALLOWABLE (KIPS)

ALLOWABLE LOADS OF RICHMOND INSERTS AND BOLTS TO BE USED IN INTERACTION FORMULAS FOR BEAM SIDES

I = Insert Capacity B = Bolt Capacity T = Tension S = Shear

INSERT TYPE AND SIZE		Insert Spacing on 6" C/C One Way and 20" C/C or Greater Other Way				Insert Spacing on 8" C/C One Way and 20" C/C or Greater Other Way			
		A-307 Bolts or A-36 THD. Rods used with insert		A-325 or Better Bolts used with insert		A-307 Bolts or A-36 THD. Rods used with insert		A-325 or Better Bolts used with insert	
		T	S	T	S	T	S	T	S
1" (EC2W)	I	14.24	14.24	14.24	14.24	16.08	16.08	16.08	16.08
	B	19.38	12.56	38.77	18.85	19.38	12.56	38.77	18.85
1 1/2" (EC6W)	I	18.45	18.45	18.45	18.45	20.56	20.56	20.56	20.56
	B	44.98	28.27	89.94	42.42	44.98	28.27	89.94	42.42

TABLE B.7

SSE ALLOWABLE (KIPS)

ALLOWABLE LOADS OF 1" RICHMOND INSERTS (EC2W) AND BOLTS TO BE USED IN INTERACTION FORMULAS FOR WALLS, SLABS AND COLUMNS

I = Insert Capacity B = Bolt Capacity T = Tension S = Shear

Concrete Thickness		Insert Spacing on 10" CC Both Ways				Insert Spacing on 12" Both Ways				Insert Spacing on 14" C/C or More Both Ways (Full Capacity)			
		A-307 Bolts or A-36 THD. Rods used with insert		A-325 or Better Bolts used with insert		A-307 Bolts or A-36 THD. Rods used with insert		A-325 or Better Bolts used with insert		A-307 Bolts or A-36 THD. Rods used with insert		A-325 or Better Bolts used with insert	
		T	S	T	S	T	S	T	S	T	S	T	S
10" or Thicker	I	9.6	9.6	9.6	9.6	14.16	14.16	14.16	14.16	18.4	18.4	18.4	18.4
	B	19.38	12.56	38.77	18.85	19.38	12.56	38.77	18.85	19.38	12.56	38.77	18.85

TABLE B.8

SSE ALLOWABLE (KIPS)

ALLOWABLE LOADS OF 1 1/2" RICHMOND INSERTS (EC6W) AND BOLTS TO BE USED IN INTERACTION FORMULAS FOR WALLS, SLABS AND COLUMNS

I = Insert Capacity B = Bolt Capacity T = Tension S = Shear

Concrete Thickness		Insert Spacing on 20" CC Both Ways				Insert Spacing on 22" C/C or More Both Ways (Full Capacity)			
		A-307 Bolts or A-36 THD. Rods used with insert		A-325 or Better Bolts used with insert		A-307 Bolts or A-36 THD. Rods used with insert		A-325 or Better Bolts used with insert	
		T	S	T	S	T	S	T	S
12" or Thicker	I	40.0	40.0	40.0	40.0	50.08	43.20	50.08	43.20
	B	44.98	28.27	89.94	42.42	44.98	28.27	89.94	42.42

Attachment C
Minimum Bolt/Insert Spacings and Clearances*

Table	Title	Reference
C.1	Minimum Spacing Between Hilti Expansion Bolts - Hilti's	3.1.1, Appendix 2, Attach. 1
C.2	Minimum Bolt Clearances	3.1.1, Appendix 2, Attach. 1
C.3	Minimum Clearances To Embedded Plates - Hilti's	3.1.1, Appendix 2, Attach. 1
C.4	Minimum Concrete Edge Distance - Richmond Inserts	3.1.1, Appendix 3.
C.5	Minimum Spacings Between - Richmond Inserts	3.1.1, Appendix 3.

2

* Note that all capacities given in the tables of Attachment B to this instruction require evaluation with regard to the minimum spacings and clearances detailed in this attachment. Those anchorages not meeting the minimum spacings and/or clearances require capacity reduction in accordance with Section 5.1 of this instruction.

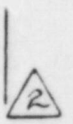
TABLE C.1

MINIMUM SPACING BETWEEN HILTI EXPANSION BOLTS
(INCHES)

Hilti Bolt Size	CENTER TO CENTER SPACING TO: 1/4"	3/8"	1/2"	5/8"	3/4"	1"	1 1/4"
1/4	2 1/2	3 1/8	3 3/4	4 3/8	5	6 1/4	7 1/2
5/16	2 13/16	3 7/16	4 1/16	4 11/16	5 5/16	6 9/16	7 13/16
3/8	3 1/8	3 3/4	4 3/8	5	5 5/8	6 7/8	8 1/8
1/2	3 3/4	4 3/8	5	5 5/8	6 1/4	7 1/2	8 3/4
5/8	4 3/8	5	5 5/8	6 1/4	6 7/8	8 1/8	9 3/8
3/4	5	5 5/8	6 1/4	6 7/8	7 1/2	8 3/4	10
7/8	5 5/8	6 1/4	6 7/8	7 1/2	8 1/8	9 3/8	10 5/8
1	6 1/4	6 7/8	7 1/2	8 1/8	8 3/4	10	11 1/4
1 1/4	7 1/2	8 1/8	8 3/4	9 3/8	10	11 1/4	12 1/2

TABLE C.2

MINIMUM BOLT CLEARANCES
(INCHES)



Hilti Bolt Size	MINIMUM DISTANCE TO Richmond Screw Anchors*		Concrete Edge*	Abandoned Hilti Bolts or Holes and Embedded Anchor Bolts that are Cut Off**
	1 inch	1 1/2 inch		
1/4	7 5/8	12 1/4	1 1/4	1/2
3/8	8 1/4	12 7/8	1 7/8	3/4
1/2	8 7/8	13 1/2	2 1/2	1
5/8	9 1/2	14 1/8	3 1/8	1 1/4
3/4	10 1/8	14 3/4	3 3/4	1 1/2
1	11 3/8	16	5	2
1 1/4	12 5/8	17 1/4	6 1/4	2 1/2

* Measured center-to-center of bolts and bolt center-to-edge of concrete in inches.

** Minimum spacing between holes covered by this column shall be measured center-to-center and based on size of hole being drilled (e.g., pilot hole spacing is based on pilot bit size).

TABLE C.3

MINIMUM CLEARANCES TO EMBEDDED PLATES - HILTI'S
(INCHES)

1. Where embedded steel plates are unoccupied by attachments for a minimum distance of 12 inches on both sides of a proposed Hilit Bolt location, the center of the bolt may be as close as practical to edge of the plate without damage to plate.

2. Where the embedded steel plates are occupied by attachments within minimum distances shown above, the minimum clearance to Hilti Anchors shall be as follows:

Hilti Anchor Size	Nelson Stud to Hilti Anchor	Edge of Plate to Hilti Anchor
1/4	5 1/4	3 3/4
3/8	5 7/8	4 3/8
1/2	6 1/2	5
5/8	7 1/8	5 5/8
3/4	7 3/4	6 1/4
1	9	7 1/2
1 1/4	10 1/4	8 3/4

Distance measured with reference to center of bolts and studs.

Where location of the nearest Nelson Stud can be determined from the "S" stamps on the embedded steel plate, the minimum center-to-center clearance to the Hilti Anchor as shown above shall govern. Where location of the nearest Nelson Stud cannot be so determined, the minimum clearance to "Edge of Plate" as shown above shall govern.

TABLE C.4

MINIMUM CONCRETE EDGE DISTANCES - RICHMOND INSERTS
(INCHES)

Attachment Location	Minimum Edge Distance			
	Tension		Shear	
Bolt Dia.	1"	1 1/2"	1"	1 1/2"
Columns, Walls and Slabs	7	11	9 1/2	14
Beam Sides	7	8	7	8

TABLE C.5

MINIMUM SPACINGS BETWEEN - RICHMOND INSERTS
(INCHES)

Attachment Location	Minimum Spacing	
Bolt Dia.	1"	1 1/2"
Beam Sides	6 c/c by 20 c/c	8 c/c by 20 c/c
Columns, Walls and Slabs		
Minimum Concrete Thickness		
10"	10 c/c	--
	12 c/c	--
	14 c/c	--
12"	--	20 c/c
		22 c/c
16"	--	10 c/c
		12 c/c
22"	--	10 c/c
		12 c/c

Attachment D

Title	Reference
Design Criteria For Embedded Plate Strips	3.13, Appendix 4 (2)

DBD-CS-17 Rev. 0 to be supplied by WEBS (TUGCo) later.

Attachment E

Expansion Anchor Length Identification System

E.1 General

Reference 3.1.4, Section 3.2.2.8.3 details the anchor bolt information provided on the "as-built" drawings. Included is a description of the "Concrete Bolt Table" that is provided with the "as-built".

E.2 Embedment Length

The "Concrete Bolt Table" provides various information. However, key to the determination of embedment length is "mark" and "projection". The "mark" indicates the full length of the bolt. Refer to Table E.1 for letter designation corresponding to a bolt length range. Embedment length equals minimum length per Table E.1 minus the "projection" dimension given in the "Concrete Bolt Table".

Note that in locations where "topping" slabs are encountered, the "embedment length" of a particular expansion bolt must be reduced by the thickness of the "topping" slab.

Table E.1

Length Identification System*

Stamp on Anchor	Length of Anchor (Inches) From	Up to (Not including)
A	1 1/2	2
B	2	2 1/2
C	2 1/2	3
D	3	3 1/2
E	3 1/2	4
F	4	4 1/2
G	4 1/2	5
H	5	5 1/2
I	5 1/2	6
J	6	6 1/2
K	6 1/2	7
L	7	7 1/2
M	7 1/2	8
N	8	8 1/2
O	8 1/2	9
P	9	9 1/2
Q	9 1/2	10
R	10	11
S	11	12
T	12	13
U	13	14
V	14	15
W	15	16
X	16	17
Y	17	18
Z	18	19
AA	19	20
BB	20	21
CC	21	22
DD	22	23
EE	23	24
FF	24	25
GG	25	26
HH	26	27
II	27	28
JJ	28	29
KK	29	30
LL	30	31
MM	31	32
NN	32	33

Table E.1 (cont'd)

Length Identification System*

Stamp on Anchor	Length of Anchor (Inches) From	Up to (Not including)
OO	33	34
PP	34	35
QQ	35	36
RR	36	37
SS	37	38
TT	38	39
UU	39	40
VV	40	41

NOTE: 1. Stamped letters shall be on top (threaded) end of bolt.

2. Bolts of 19-inch length and greater may be stamped with number corresponding to the bolt length in inches in the same manner instead of the stamped letters as listed above.

3. Star stamp signifies Hilti Super Kwik bolts.

Attachment F

ANCHORAGE PLATE/ANGLE STRESS VERIFICATION

Anchorages that cannot be correlated to one of the interaction diagrams of Attachment A (Step 1, Fig. F.1), but may be verified via hand calculations (Step 2, Fig. F.1) require the verification of plate/angle stress levels.

The methodologies presented below provide conservative means to assess the plate/angle stress levels. The methodologies utilize the maximum bolt tension, T and shear, S values determined utilizing the standard equations of section 4.3.

Qualification of Anchorage Angles and Plates

The stresses in the anchorage angles and baseplates must be checked to ensure that they are within the appropriate allowable values. This section presents equations that can be used in the checking procedure. The equations are based on anchorage statics and the stresses are computed using standard mechanics equations. For uncommon anchorage arrangements where the equations may not apply, a stress check procedure using similar principles must be employed.

1.0 Angle having one bolt

The geometry is shown in Figure 1.0, 1.1 and 1.2 stesses are checked globally and locally. See Figure F3.b for variable descriptions.

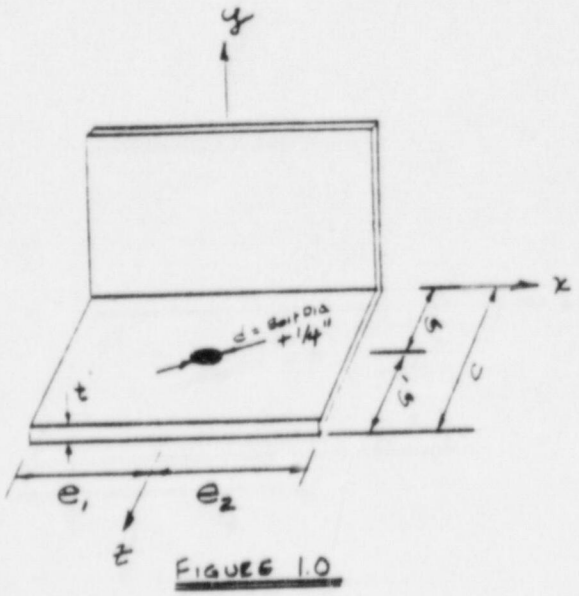


FIGURE 1.0

1.1 Global stress assessment

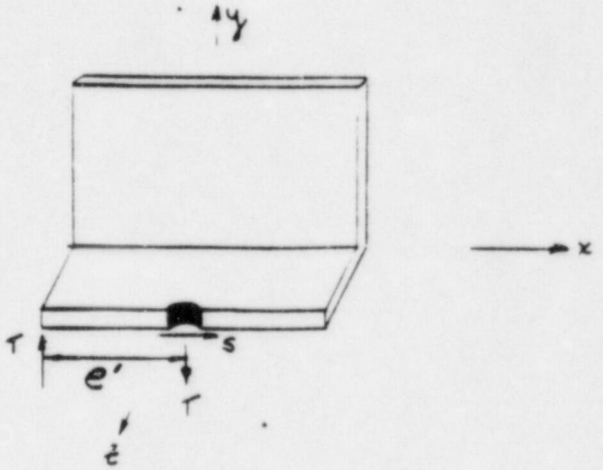


FIGURE 1.1

Section Properties:

$e_{max} = \max [e_1, e_2]$

$A_t = (C - d)t$

$A_s = .67(Ct)$

Z_z = Angle section modulus

Forces:

T = Bolt tension

S = Bolt shear

$M_z = T \times e_{max}$

Stresses:

$f_a = S/A_t$

$f_b = M_z/Z_z$

$f_v = [(T^2 + S^2)^{1/2}]/A_s$

Stress Ratios:

$$\frac{f_a}{.6F_{yd} \times LF} + \left[\frac{f_b}{.6F_{yd} \times LF}\right] \times 1.2 \leq 1.0$$

$$f_v \leq 0.4 \times F_{yd} \times LF \leq 0.55F_{yd}$$

where,

F_{yd} = Material yield stress

LF = Load Factor (LF) as defined in Ref. 3.1.7.

1.2 Local stress assessment

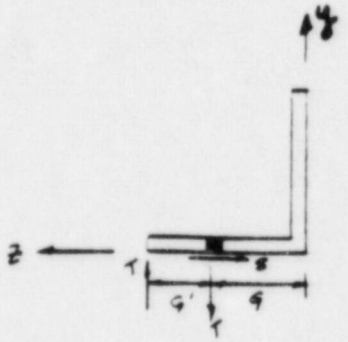


FIGURE 1.2

Section Properties:

$L = e_1 + e_2$

$B = L \leq 8$ in

$A_s = (B \times t).67$

$Z_x = B\ t^2/6$

$A_t = t(B - d)$

Forces:

T = Bolt tension

S = Bolt shear

$M_x = T \times G'$

Stresses:

$f_a = S/A_t$

$f_b = M_x/Z_x$

$f_v = [(T^2 + S^2)^{1/2}]/A_s$

Stress Ratio:

$$\frac{f_a}{.6F_{yd} \times LF} + \frac{f_b}{\min\ (.75F_{yd} \times LF,\ 1.0F_{yd})} \leq 1.0$$

$f_v \leq 0.4F_{yd} \times LF \leq 0.55F_{yd}$

Check Edge Distance

$G' \geq 2S/F_u t$ Check also TABLE 1.16.5 of Reference 3.2.1

where,

F_u = As defined in Table 2, Appendix A of Ref. 3.2.1
= 58.0 ksi for A36 steel

2.0 Angle having two bolts

The geometry is shown in Figures 2.0, 2.1 and 2.2. Stresses are checked globally and locally. See Figure F3.a for variable descriptions.

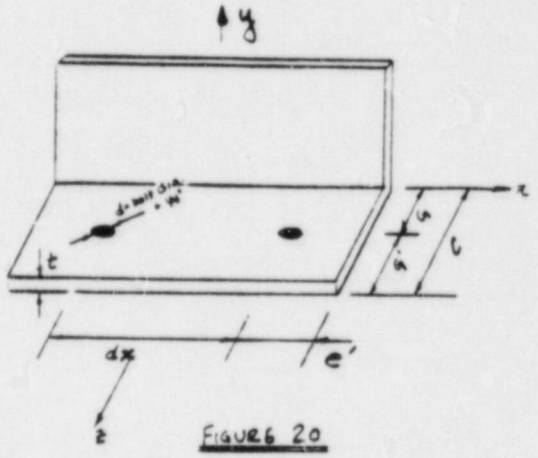


FIGURE 2.0

2.1 Global stress assessment

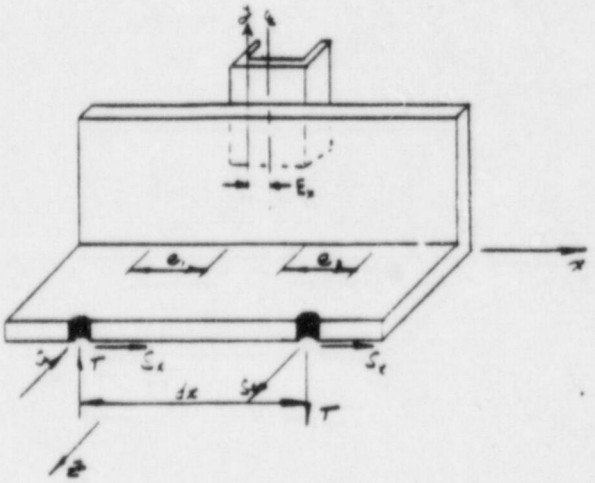


FIGURE 2.1

Section Properties:

$A_t = (C - d)t$

$A_s = .67(Ct)$

Z_y = Angle section modulus

Z_z = Angle section modulus

$e_{max} = \max(e_1, e_2)$

Forces:

T = Bolt tension

S = Bolt shear

$M_y = S \times e_{max}$

$M_z = T \times (d_x/2 + E_x)$

2

Stresses:

$f_a = S/A_t$

$f_{by} = M_y/Z_y$

$f_{bz} = M_z/Z_z$

$f_v = [(T^2 + S^2)^{1/2}]/A_s$

Stress Ratios:

$$\frac{f_a}{.6F_{yd} \times LF} + \left[\frac{f_{by}}{.6F_{yd} \times LF} + \frac{f_{bz}}{.6F_{yd} \times LF}\right] \times 1.2 \leq 1.0$$

$f_v \leq 0.4F_{yd} x(LF) \leq 0.55F_{yd}$

Check Edge Distance

$e' \geq 2S/F_u t$ Check also TABLE 1.16.5 of Reference 3.2.1

2.2 Local stress assessment

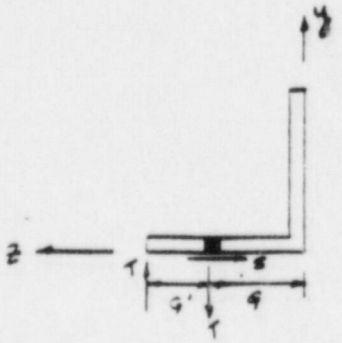


FIGURE 2.2

Section Properties:

$B = L/2 \leq 8in$

$A_t = (B-d)t$

$Z_x = Bt^2/6$

$Z_y = \min[C^2t/6, L^2t/12]$

Forces:

T = Bolt tension

S = Bolt shear

$M_x = T \times G'$

$M_y = S \times e_{max}$

Stresses:

$f_a = S/A_t$

$f_{bx} = M_x/Z_x$

$f_{by} = M_y/Z_y$

Stress Ratio:

$$\left[\frac{f_a}{.6F_{yd} \times LF}\right] + \left[\frac{f_{bx}}{\min(.75F_{yd} \times LF,\ 1.0\ F_{yd})}\right] + \left[\frac{f_{by}}{.6F_{yd} \times LF}\right] \leq 1.0$$

Check Edge Distance:

$G' \geq 2S/F_u t$ Check also TABLE 1.16.5 of Reference 3.2.1

2

2

3.0 Angle having more than two bolts

The geometry is shown in Figure F.5 . Stresses are checked globally and locally. See Figure F.5 for variable descriptions.

Section Properties:

$A_t = (C - d)t$

$A_s = 0.67(Ct)$

Z_y = Angle section modulus about Y axis

Z_z = Angle section modulus about z axis

$e_{max} = \max(d_{xi}/2 + E_{xi}) \quad i = 1, m$

Forces:

T_{max} = max bolt tension

S_{max} = max bolt shear

$M_y = S \times e_{max}$

$M_z = T \times e_{max}$

Stresses:

$f_a = S/A_t$

$f_{by} = M_y/Z_y$

$f_{bz} = M_z/Z_z$

$f_v = [(T^2 + S^2)^{1/2}]/A_s$

Stress Ratios:

$$\frac{f_a}{.6F_{yd} \times LF} + \left[\frac{f_{by}}{.6F_{yd} \times LF} + \frac{f_{bz}}{.6F_{yd} \times LF}\right] \times 1.2 \leq 1.0$$

$f_v \leq 0.40\ F_{yd} \leq 0.55\ F_{yd}$

Check Edge Distance:

Minimum edge distance $\geq 2S/F_u t$

Check also TABLE 1.16.5 of Reference 3.2.1

2

2

Local Stress Check

Section Properties:

$B = \text{minimum } (d_{xi}/2 - E_{xi}) \leq 8in \quad i = 1 \text{ to } m$

$A_t = (B - d)t$

$Z_x = (B)t^2/6$

$D = \text{minimum } (B, C)$

$Z_y = D^2t/6$

Forces:

$M_x = T_{max} \times G'$

$M_y = S_{max} \times e_{max}$

Stresses:

$f_a = S_{max}/A_t$

$f_{bx} = M_x/Z_x$

$f_{by} = M_y/Z_y$

Stress Ratio:

$$\frac{f_a}{.6F_{yd} \times LF} + \frac{f_{bx}}{\min(.75F_{yd} \times LF,\ 1.0\ F_{yd})} + \frac{f_{by}}{.6F_{yd} \times LF} \leq 1.0$$

4.0 Baseplate having two bolts

The geometry is shown in Figures 4.0, 4.1 and 4.2. Stresses are checked globally and locally. See Figure F4.a for variable descriptions.

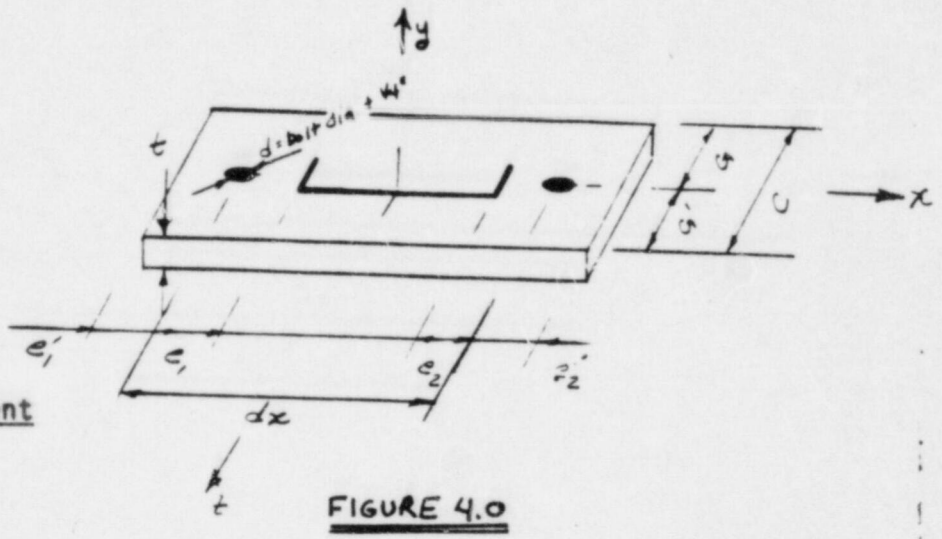


FIGURE 4.0

4.1 Global stress assessment

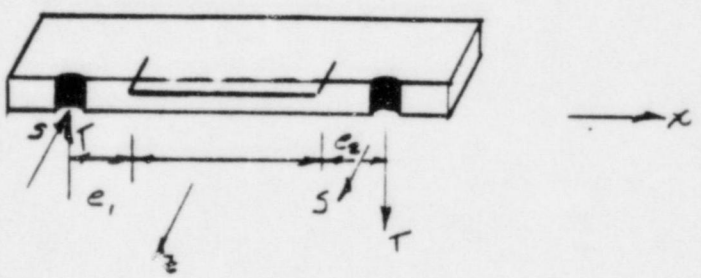


FIGURE 4.1

Section Properties:

$A_t = (C - d)t$

$A_s = .67(Ct)$

$Z_y = tC^2/6$

$Z_z = Ct^2/6$

$e_{max} = \max [e_1, e_2]$

Forces:

T = Bolt tension - T may be reduced by $Fy/2\ (a_3 - 1.0)$ for stress calculation

S = Bolt shear

$M_y = S \times e_{max}$

$M_z = T \times e_{max}$

Stresses:

$f_a = S/A_t$

$f_{by} = M_y/Z_y$

$f_{bz} = M_z/Z_z$

$f_v = [(T^2 + S^2)^{1/2}]/A_s$

Stress Ratios:

$$\frac{f_a}{.6F_{yd} \times LF} + \frac{f_{by}}{.6F_{yd} \times LF} + \frac{f_{bz}}{\min(0.75F_{yd} \times LF,\ 1.0F_{yd})} \leq 1.0$$

2

$f_v \leq .40\ F_{yd} \times LF \leq 0.55\ F_{yd}$

Check Edge Distance:

$e'_{min} \geq 2S/F_ut$ Check also TABLE 1.16.5 of Reference 3.2.1

2

2

4.2 Local stress assessment

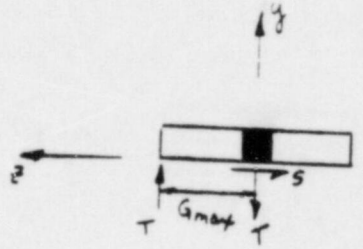


FIGURE 4.2

Section Properties:

$$B = [e'_{min} + dx/2 - Ex] \leq 8in$$

$$D = \min[e'_1 + e_1 , e'_2 + e_2] \leq 8in$$

$$A_t = (D - d)t$$

$$Z_x = Bt^2/6$$

$$Z_y = \min[C^2t/6, L^2t/12]$$

Forces:

T = Bolt tension

S = Bolt shear

$$M_x = T \times G_{max}$$

$$M_y = S \times e_{max}$$

Stresses:

$$f_a = S/A_t$$

$$f_{bx} = M_x/Z_x$$

$$F_{by} = M_y/Z_y$$

Stress Ratio:

$$\frac{f_a}{.6Fyd \times LF} + \frac{f_{bx}}{\min(.75Fyd \times LF,\ 1.0Fyd)} + \frac{f_{by}}{.6Fyd \times LF} \leq 1.0$$

Check Edge Distance:

$G_{min} \geq 2S/F_ut$ Check also TABLE 1.16.5 of Reference 3.2.1

5.0 Baseplate having four bolts

The geometry is shown in Figures 5.0, 5.1 and 5.2, stresses are checked globally in the "xy" and "yz" planes. See Figure F4.b for variable descriptions.

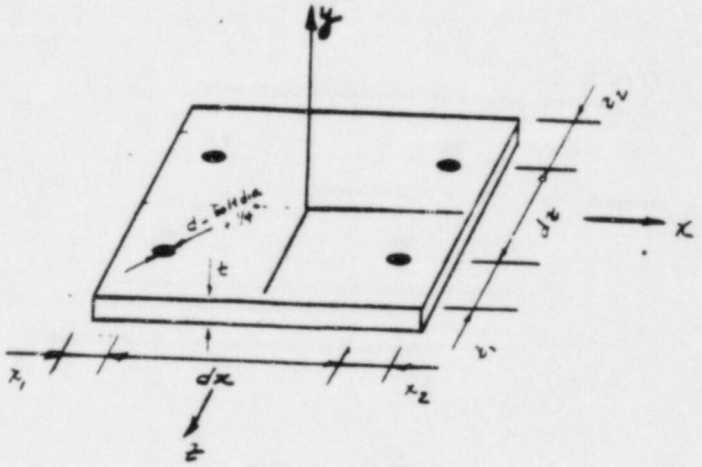


FIGURE 5.0

5.1 Global stress assessment - "XY" plane

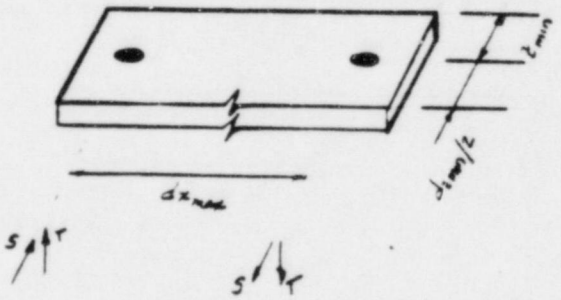


FIGURE 5.1

Section Properties:

$d_{min} = d_x/2 - E_x$

$d_{max} = d_z/2 + E_z$

$Z_x = d_{min}\ t^2/6$

$A_s = .67\ (d_{min} \times t)$

$A_t = d_{min} \times t$

Forces:

T = Bolt tension - T may be reduced by Fy/4 (a_3 - 1.0) for stress calculations.

S = Bolt shear

$M_x = T \times d_{max}$

Stresses:

$f_{bx} = M_x/Z_x$

$f_a = S/A_t$

$f_v = [(T^2 + S^2)^{1/2}]/A_s$

Stress Ratios:

$$\frac{f_a}{.6F_{yd} \times LF} + \frac{f_{bx}}{\min(.75F_{yd} \times LF,\ 1.0F_{yd})} \leq 1.0$$

$f_v \leq 0.4\ F_{yd} \times LF \leq 0.55\ F_{yd}$

5.2 Global stress assessment - YZ plane

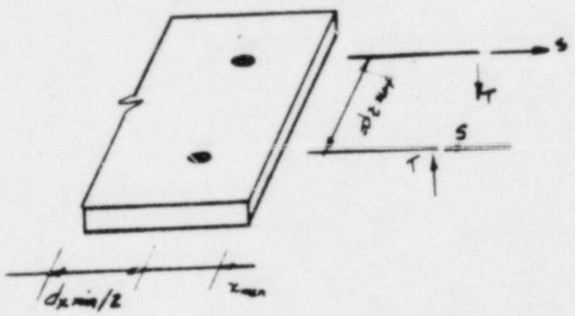


FIGURE 5.2

Section Properties

$d_{min} = d_z/2 - E_z$

$d_{max} = d_x/2 + E_x$

$Z_z = d_{min}\ t^2/6$

$A_s = .67\ (d_{min} \times t)$

$A_t = d_{min} \times t$

Forces

T = Bolt tension - T may be reduced by $F_y/4\ (a_3-1.0)$ for stress calculations.

S = Bolt shear

$M_z = T \times d_{max}$

Stresses

$f_{bz} = M_z/Z_z$

$f_a = S/A_t$

$f_v = [(T^2 + S^2)^{1/2}]/A_s$

Stress Ratio:

$$\frac{f_a}{.6\ F_{yd} x LF} + \frac{f_{bz}}{\min\ (.75F_{yd}\ x\ LF,\ 1.0\ F_{yd})} \leq 1.0$$

$$f_v \leq 0.4\ F_{yd}\ x\ LF \leq 0.55\ F_{yd}$$

Checked Edge Distance:

$e \geq 2S/F_u t$ Check also TABLE 1.16.5 of Reference 3.2.1

$e = \min(x_{min}, z_{min})$

2

TABLE T.1 - PRYING ACTION FACTORS FOR BASE ANGLES HAVING TWO BOLTS

TYPE & SIZE OF BOLTS	BASE ANGLE	d_x	C (INCHES)	Prying action factors a_1	a_2	a_3	
ALL SIZES OF HILTI KWIK & SUPER KWIK 1/4 thru 1-1/2"	L8 x 6 x 3/4	2' - 4" MAX	8.	1.12	2.00	1.09	
	L6 x 6 x 3/4	"	6.	1.09	1.69	1.06	
	L6 x 4 x 3/4	"	4.	1.09	1.69	1.06	2
	L5 x 5 x 3/4	"	5.	1.09	1.69	1.06	
1-1/2" DIA. RICHMOND INSERT	L8 x 6 x 3/4	"	8.	1.27	3.07	1.23	
	L6 x 6 x 3/4	"	6.	1.26	2.56	1.21	
	L6 x 4 x 3/4	"	4.	1.26	2.56	1.21	2
	L5 x 5 x 3/4	"	5.	1.26	2.56	1.21	
1" DIA. RICHMOND INSERT	L8 x 6 x 3/4	"	8.	1.23	2.38	1.19	
	L6 x 6 x 3/4	"	6.	1.22	2.38	1.16	
	L6 x 4 x 3/4	"	4.	1.22	2.38	1.16	2
	L5 x 5 x 3/4	"	5.	1.22	2.38	1.16	

TABLE T.2 - PRYING ACTION FACTORS FOR BASE ANGLES HAVING ONE BOLT

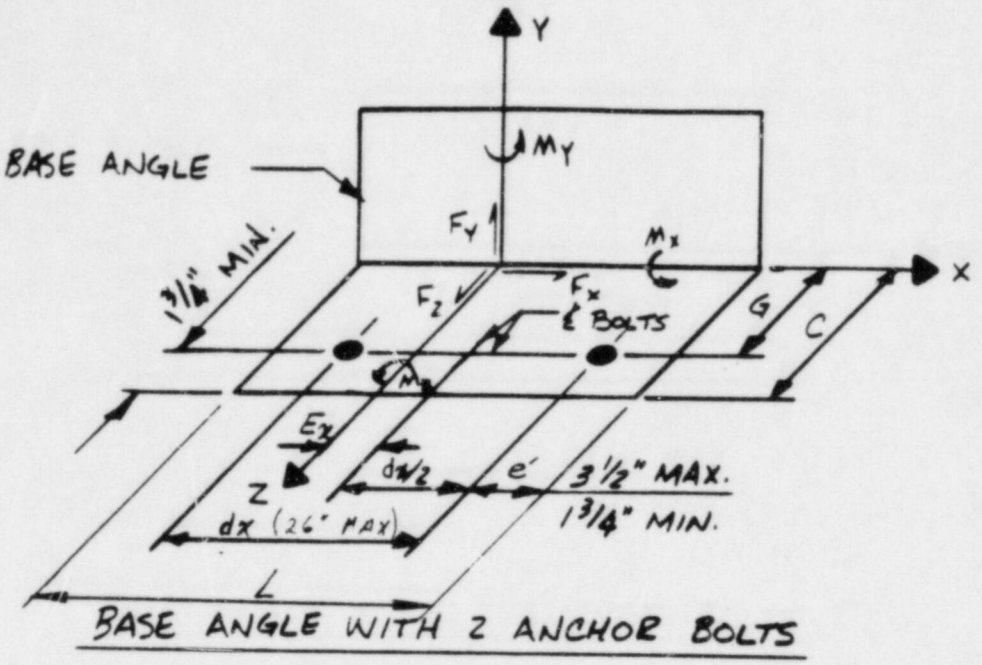
TYPE & SIZE OF BOLTS	BASE ANGLE	C (INCHES)	Prying action factors a_1	a_2	a_3	
ALL SIZES OF HILTI KWIK & SUPER KWIK 1/4 thru 1-1/2"	L8 x 6 x 3/4	8.	1.11	1.15	1.08	
	L6 x 6 x 3/4	6.	1.10	1.11	1.04	
	L6 x 4 x 3/4	4.	1.10	1.11	1.04	2
	L5 x 5 x 3/4	5.	1.10	1.11	1.04	
1-1/2" DIA. RICHMOND INSERT	L8 x 6 x 3/4	8.	1.20	1.67	1.15	
	L6 x 6 x 3/4	6.	1.19	1.56	1.12	
	L6 x 4 x 3/4	4.	1.19	1.56	1.12	2
	L5 x 5 x 3/4	5.	1.19	1.56	1.12	
1" DIA. RICHMOND INSERT	L8 x 6 x 3/4	8.	1.17	1.55	1.12	
	L6 x 6 x 3/4	6.	1.16	1.47	1.11	
	L6 x 4 x 3/4	4.	1.16	1.47	1.11	2
	L5 x 5 x 3/4	5.	1.16	1.47	1.11	

TABLE T.3 - PRYING ACTION FACTORS FOR BASEPLATE HAVING FOUR BOLTS

TYPE & SIZE OF BOLTS	BASE PLATE DIM's (IN) t	dx	dy	PRYING ACTION FACTORS a_1	a_2	a_3
ALL SIZES OF HILTI KWIK & SUPER KWIK	1.0 MIN.	20-1/2 MAX.	20-1/2 MAX.	1.22	1.22	1.30
1-1/2" RICHMOND INSERT	1.0 MIN.	20-1/2 MAX.	20-1/2 MAX.	2.16	2.16	2.09
1" RICHMOND INSERT	1.0 MIN.	20-1/2 MAX.	20-1/2 MAX.	1.95	1.95	1.90

TABLE T.4 - PRYING ACTION FACTORS FOR BASEPLATE HAVING TWO BOLTS

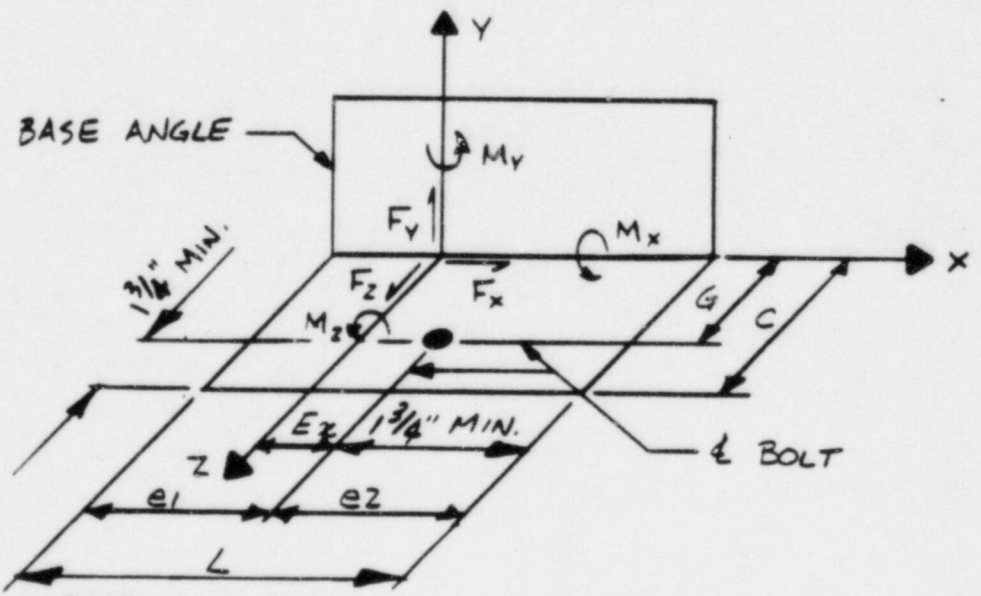
TYPE & SIZE OF BOLTS	BASE PLATE DIM's (IN) t	dx	C	PRYING ACTION FACTORS a_1	a_2	a_3
ALL SIZES OF HILTI KWIK & SUPER KWIK	0.75 MIN.	22 MAX.	8 MIN.	1.24	2.31	2.21
1-1/2" RICHMOND INSERT	0.75 MIN.	22 MAX.	8 MIN.	1.67	3.84	3.29
1" RICHMOND INSERT	0.75 MIN.	22 MAX.	8 MIN.	1.56	3.49	3.06



BASE ANGLE WITH 2 ANCHOR BOLTS

FIG. F.3a

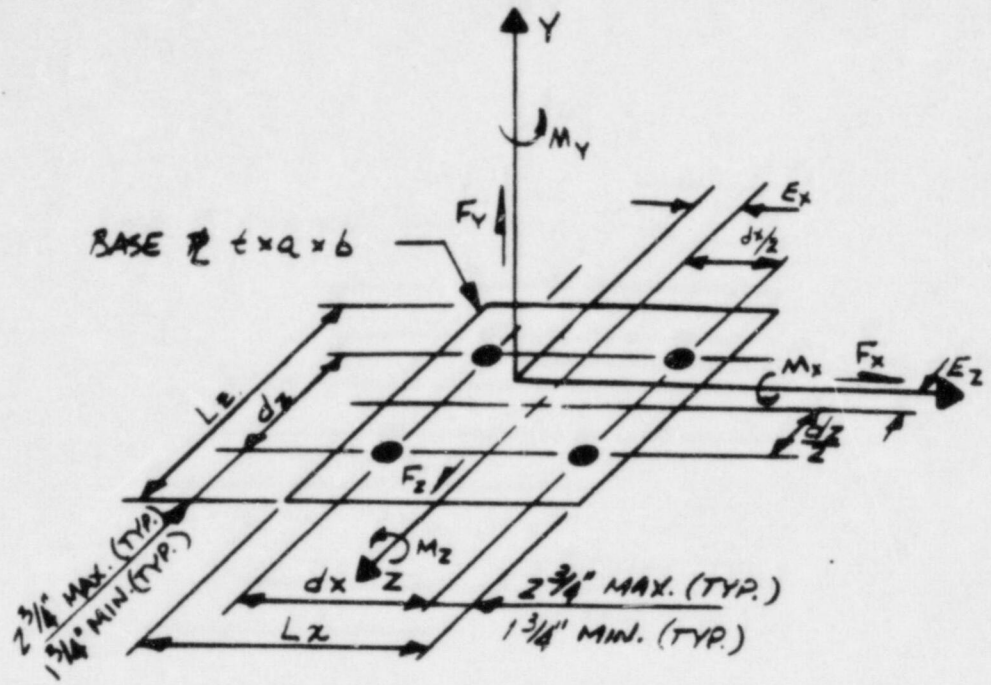
(USE ONLY WITH TABLE T.1)



BASE ANGLE WITH 1 ANCHOR BOLT

FIG F.3b

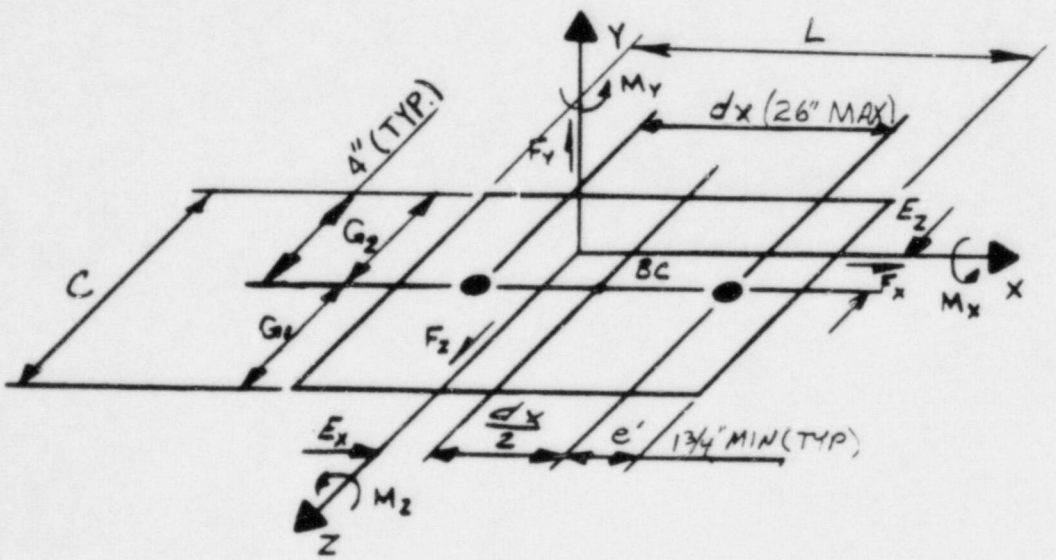
(USE ONLY WITH TABLE T.2)



BASE PLATE WITH 4 ANCHOR BOLTS

FIG. F.4a

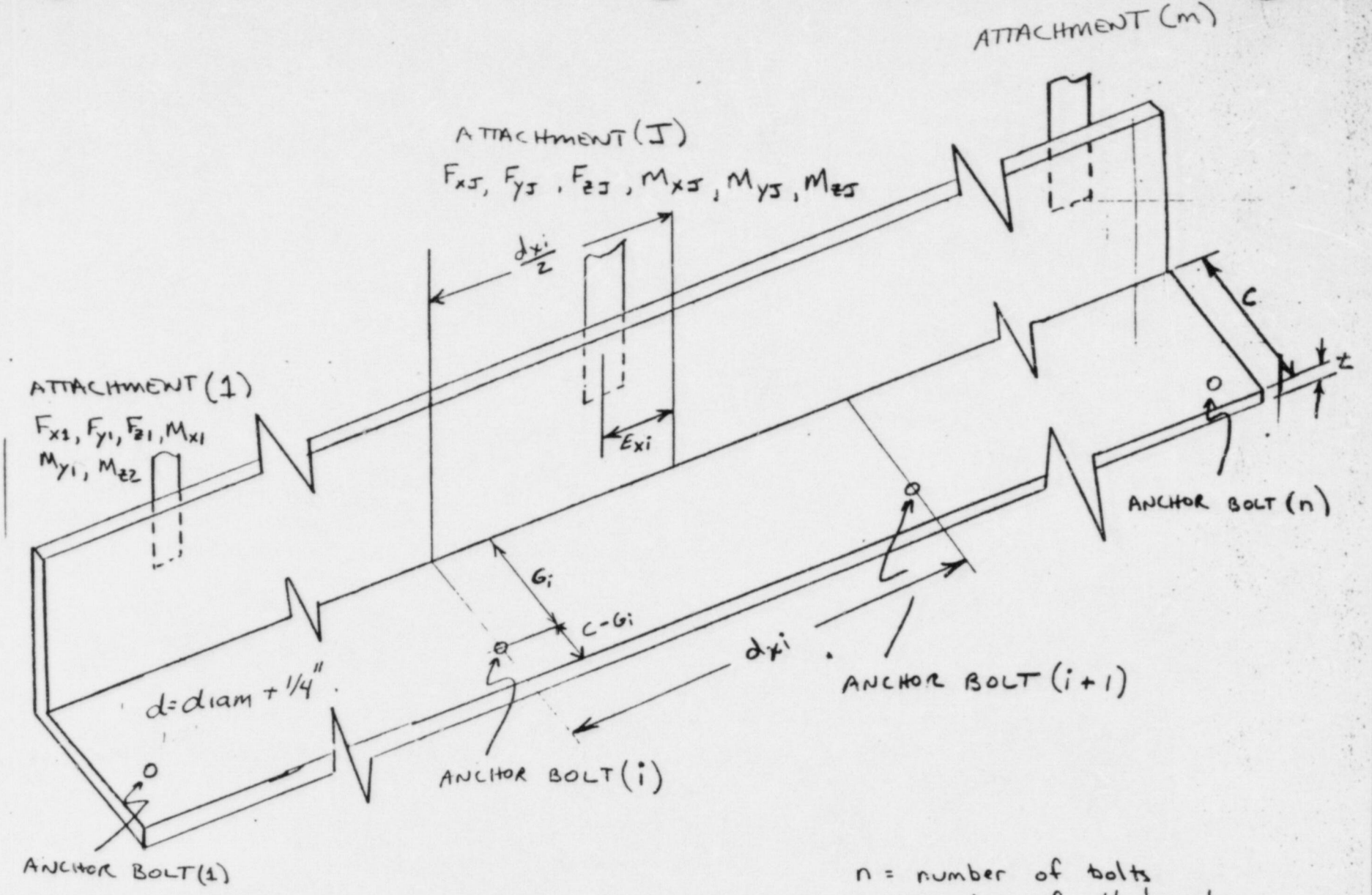
(USE ONLY WITH TABLE T.3)



BASE PLATE WITH 2 ANCHOR BOLTS

FIG. F.4b

(USE ONLY WITH TABLE T.4)



n = number of bolts
m = number of attachments

FIG F.5

CALCULATION/PROBLEM COVER SHEET

Calculation/Problem No: M-03

Title: CABLE TRAY PROPERTIES

Client: TUGCo Project: CPSES - TRAYS

Job No: 0210-010

Design Input/References:

See Within

Assumptions:

See Within

Method:

See Within

Remarks:

REV. NO.	REVISION	APPROVED	DATE
0	Initial Issue	[signature]	12/5/85
1	INCORPORATE GIBBS & HILL COMMENTS	[signature]	3/26/86
	UNCONTROLLED COPY		
	NOTE: THIS IS NOT A COMPLETE COPY.		

ATTACHMENT 1

CALCULATION OF MOMENT OF INERTIA

UNCONTROLLED COPY

					TUGCO	
					Mom. of Inertia - Cable Trays	
					JOB NO 0210-040.	PAGE —
0	A. Saha	12.2.85	[signature]	12.5.85	CALC NO M-03	OF —
REV	BY	DATE	CHECKED	DATE	IMPELL CORPORATION	

1.1 Section Name: GI-30SL-12 Load: Vertical

w1	delta1	w2	delta2	w3	delta3	I1	I2	I3
100	0.156	100	0.142	100	0.130	2.003	2.200	2.403
200	0.306	200	0.297	200	0.294	2.042	2.104	2.125
300	0.453	300	0.426	300	0.419	2.069	2.200	2.237
400	0.561	400	0.581	400	0.570	2.227	2.151	2.192
500	0.712	500	0.700	500	0.687	2.194	2.231	2.274
600	0.825	600	0.809	600	0.793	2.272	2.317	2.364
700	0.943	700	0.928	700	0.908	2.319	2.357	2.408
800	1.089	800	1.053	800	1.026	2.295	2.373	2.436
900	1.237	900	1.192	900	1.159	2.273	2.359	2.426
1000	1.442	1000	1.398	1000	1.360	2.166	2.235	2.297
1100	1.694	1100	1.642	1100	1.595	2.029	2.093	2.155
1200	1.991	1200	1.905	1200	1.852	1.883	1.968	2.024
1294	collaps	1300	2.340	1300	2.283	ERR	1.736	1.779
		1309	collaps	1333	collapse			

Not used in Calc. of I, so o.k.

Average Inertia thru w = 900 #/ft 2.254 (calc)/fs
Standard Deviation 0.1060
Variance 0.0112

1.2 Section Name: GI-30SL-12 Load: Horizontal

w1	delta1	w2	delta2	w3	delta3	I1	I2	I3
75	0.136	75	0.131	75	0.126	1.723	1.789	1.860
150	0.231	150	0.227	150	0.220	2.029	2.064	2.130
225	0.327	225	0.320	225	0.311	2.150	2.197	2.260
300	0.436	300	0.428	300	0.416	2.150	2.190	2.253
375	0.530	375	0.523	375	0.508	2.210	2.240	2.306
450	0.612	450	0.602	450	0.585	2.297	2.335	2.403
525	0.708	525	0.694	525	0.673	2.317	2.363	2.437
600	0.779	600	0.768	600	0.741	2.406	2.441	2.530
675	0.843	675	0.831	675	0.799	2.501	2.538	2.639
750	0.920	750	0.905	750	0.870	2.547	2.589	2.693
825	0.986	825	0.969	825	0.928	2.614	2.660	2.777
900	1.062	900	1.046	900	0.996	2.648	2.688	2.823
975	1.145	975	1.117	975	1.059	2.660	2.727	2.876
1050	1.274	1050	1.239	1050	1.175	2.575	2.648	2.792
1125	1.361	1125	1.317	1125	1.248	2.582	2.669	2.816
1200	1.514	1200	1.458	1200	1.380	2.476	2.571	2.717
1275	1.716	1275	1.647	1275	1.559	2.321	2.418	2.555
1350	1.956	1350	1.873	1350	1.771	2.156	2.252	2.381
1412	collaps	1425	collaps	1425	2.015	ERR	ERR	2.209
				1477	collapse			ERR

Average Inertia thru w = 825 #/ft 2.322
Standard Deviation 0.2522
Variance 0.0636

2.1 Section Name: JM-36SL-12 Load: Vertical

w1	delta1	w2	delta2	w3	delta3	I1	I2	I3
100	0.197	100	0.194	100	0.197	1.586	1.610	1.586
200	0.371	200	0.373	200	0.378	1.684	1.675	1.653
300	0.525	300	0.529	300	0.536	1.785	1.772	1.749
400	0.652	400	0.658	400	0.668	1.917	1.899	1.871

500	0.770	500	0.783	500	0.797	2.029	1.995	1.960
600	0.878	600	0.890	600	0.908	2.135	2.106	2.064
700	0.971	700	0.984	700	1.005	2.252	2.222	2.176
800	1.100	800	1.114	800	1.139	2.272	2.243	2.194
900	1.267	900	1.285	900	1.319	2.219	2.188	2.132
1000	1.478	1000	1.501	1000	1.542	2.114	2.081	2.026
1100	1.682	1100	1.708	1100	1.757	2.043	2.012	1.956
1200	1.964	1200	1.994	1200	2.063	1.909	1.880	1.817
1291	collaps	1289	collaps	1275	collaps	ERR	ERR	ERR

Average Inertia thru w = 900 #/ft 1.962
Standard Deviation 0.2300
Variance 0.0529

2.2 Section Name: JM-36SL-12 Load: Horizontal

w1	delta1	w2	delta2	w3	delta3	I1	I2	I3
100	0.093	100	0.091	100	0.097	3.359	3.433	3.221
200	0.187	200	0.184	200	0.201	3.341	3.396	3.109
300	0.319	300	0.313	300	0.340	2.938	2.994	2.757
400	0.432	400	0.426	400	0.462	2.893	2.933	2.705
500	0.526	500	0.518	500	0.583	2.970	3.016	2.679
600	0.662	600	0.653	600	0.689	2.831	2.871	2.721
700	0.781	700	0.765	700	0.804	2.800	2.859	2.720
800	0.859	800	0.842	800	0.900	2.909	2.968	2.777
900	0.989	900	0.960	900	1.026	2.843	2.929	2.740
1000	1.098	1000	1.052	1000	1.151	2.845	2.970	2.714
1100	1.227	1100	1.185	1100	1.295	2.801	2.900	2.654
1200	1.396	1200	1.329	1200	1.491	2.685	2.821	2.514
1300	1.617	1300	1.53	1300	1.716	2.512	2.654	2.367
1362	collaps	1400	1.794	1327	collaps	ERR	2.438	ERR
		1418	collapse				ERR	

Average Inertia thru w = 900 #/ft 2.952
Standard Deviation 0.2234
Variance 0.0499

3.1 Section Name: JM-30SL-12 Load: Horizontal

w1	delta1	w2	delta2	w3	delta3	I1	I2	I3
100	0.072	100	0.068	100	0.059	4.339	4.594	5.295
200	0.169	200	0.148	200	0.140	3.697	4.222	4.463
300	0.276	300	0.255	300	0.247	3.396	3.675	3.794
400	0.393	400	0.367	400	0.349	3.180	3.405	3.581
500	0.497	500	0.481	500	0.456	3.143	3.247	3.426
600	0.631	600	0.603	600	0.590	2.971	3.109	3.177
700	0.748	700	0.719	700	0.697	2.924	3.042	3.138
800	0.832	800	0.802	800	*0.731 B1	3.004	3.116	3.419
900	0.944	900	0.900	900	0.854	2.978	3.124	3.292
1000	1.046	1000	0.998	1000	0.972	2.987	3.130	3.214
1100	1.167	1100	1.117	1100	1.093	2.945	3.077	3.144
1200	1.312	1200	1.264	1200	1.206	2.857	2.966	3.109
1300	1.509	1300	1.44	1300	1.478	2.691	2.820	2.748
1389	collaps	1400	1.671	1400	1.604	ERR	2.617	2.727
		1414	collaps	1421	collapse		ERR	ERR

* Since average of deflections used in calc. so no change in Inertia Calc.

By: RPO 10/31/85 Chk: A. Saha 11.5.85

Average Inertia thru w = 700 #/ft 3.280
Standard Deviation 0.2492
Variance 0.1684

4.1 Section Name: JM-24SL-12 Load: Horizontal

w1	delta1	w2	delta2	w3	delta3	I1	I2	I3
100	0.045	100	0.048	100	0.055	6.942	6.508	5.680
200	0.130	200	0.136	200	0.149	4.806	4.594	4.193
300	0.234	300	0.243	300	0.250	4.005	3.857	3.749
400	0.331	400	0.350	400	0.359	3.775	3.570	3.481
500	0.396	500	0.451	500	0.472	3.945	3.463	3.309
600	0.548	600	0.576	600	0.596	3.421	3.254	3.145
700	0.639	700	0.668	700	0.699	3.422	3.274	3.129
800	0.732	800	0.760	800	0.793	3.414	3.288	3.152
900	0.820	900	0.847	900	0.890	3.429	3.320	3.159
1000	0.909	1000	0.941	1000	0.988	3.437	3.320	3.162
1100	1.004	1100	1.049	1100	1.105	3.423	3.276	3.110
1200	1.108	1200	1.165	1200	1.219	3.383	3.218	3.075
1300	1.26	1300	1.331	1300	1.402	3.223	3.051	2.897
1400	1.435	1400	1.537	1400	1.652	3.048	2.846	2.648
1500	1.681	1497	collaps	1438	collaps	2.788	ERR	ERR
1526	collapse					ERR		

Average Inertia w = 200 to 1100 #/ft 3.529
Standard Deviation 0.4161
Variance 0.1731

5.1 Section Name: JM-18SL-12 Load: Vertical

w1	delta1	w2	delta2	w3	delta3	I1	I2	I3
100	0.271	100	0.247	100	0.394	1.153	1.265	0.793
200	0.502	200	0.493	200	0.510	1.245	1.267	1.225
300	0.697	300	0.670	300	0.712	1.345	1.399	1.316
400	0.873	400	0.834	400	0.898	1.431	1.498	1.392
500	1.091	500	1.046	500	1.131	1.432	1.493	1.381
600	1.348	600	1.301	600	1.385	1.391	1.441	1.353
700	1.659	700	1.607	700	1.722	1.318	1.361	1.270
779	collpas	791	collpas	746	collpas	ERR	ERR	ERR

Average Inertia thru w = 600 #/ft 1.323
Standard Deviation 0.1580
Variance 0.0250

5.2 Section Name: JM-18SL-12 Load: Horizontal

w1	delta1	w2	delta2	w3	delta3	I1	I2	I3
100	0.152	100	0.168	100	0.096	2.055	1.860	3.254
200	0.306	200	0.322	200	0.361	2.042	1.940	1.731
300	0.489	300	0.504	300	0.538	1.917	1.860	1.742
400	0.636	400	0.686	400	0.711	1.965	1.822	1.758
500	0.841	500	0.881	500	0.916	1.857	1.773	1.705
600	1.056	600	1.093	600	1.120	1.775	1.715	1.674
700	1.284	700	1.324	700	1.387	1.703	1.652	1.577
800	1.590	800	1.619	800	1.692	1.572	1.544	1.477
869	collaps	839	collaps	821	collaps	ERR	ERR	ERR

Average Inertia thru w = 500 #/ft 1.843
Standard Deviation 0.1017
Variance 0.0103

6.1 Section Name: GG-36SL-12-06 Load: Vertical

w1	delta1	w2	delta2	w3	delta3	I1	I2	I3
50	0.046	50	0.043	50	0.040	3.396	3.633	3.905
100	0.103	100	0.098	100	0.094	3.033	3.188	3.323
150	0.158	150	0.146	150	0.139	2.966	3.210	3.371
200	0.227	200	0.219	200	0.202	2.752	2.853	3.093
250	0.278	250	0.282	250	0.276	2.809	2.770	2.830
300	0.359	300	0.345	300	0.335	2.611	2.717	2.798
350	0.424	350	0.415	350	0.403	2.579	2.635	2.713
400	0.511	400	0.493	400	0.478	2.445	2.535	2.614
450	0.583	450	0.572	450	0.567	2.411	2.458	2.479
500	0.665	500	0.657	500	0.634	2.349	2.378	2.464
550	0.759	550	0.734	550	0.716	2.264	2.341	2.400
600	0.838	600	0.811	600	0.779	2.237	2.311	2.406
650	0.937	650	0.913	650	0.864	2.167	2.224	2.350
700	1.059	700	0.996	700	0.961	2.065	2.196	2.276
750	1.157	750	1.104	750	1.068	2.025	2.122	2.194
800	1.273	800	1.227	800	1.203	1.963	2.037	2.078
850	1.428	850	1.366	850	1.321	1.860	1.944	2.010
900	1.633	900	1.529	900	1.472	1.722	1.839	1.910
950	1.879	950	1.768	950	1.660	1.579	1.679	1.788
981	collaps	991	collaps	997.5	collaps	ERR	ERR	ERR

Not used in Calc. of I → ∴ o.k.

Average Inertia w = 100 to 550 #/ft 2.713
Standard Deviation 0.3026
Variance 0.1861

6.2 Section Name: GG-36SL-12-06 Load: Horizontal

w1	delta1	w2	delta2	w3	delta3	I1	I2	I3
135	0.119	135	0.122	135	0.106	3.544	3.457	3.979
180	0.169	180	0.174	180	0.149	3.327	3.232	3.774
225	0.217	225	0.229	225	0.201	3.239	3.069	3.497
270	0.276	270	0.291	270	0.248	3.056	2.899	3.401
315	0.302	315	0.309	315	0.282	3.259	3.185	3.490
360	0.323	360	0.335	360	0.304	3.482	3.357	3.700
405	0.354	405	0.369	405	0.323	3.574	3.429	3.917
450	0.387	450	0.402	450	0.357	3.633	3.497	3.938
495	0.404	495	0.418	495	0.380	3.828	3.700	4.070
540	0.430	540	0.450	540	0.408	3.923	3.749	4.135
585	0.474	585	0.506	585	0.441	3.856	3.612	4.144
630	0.513	630	0.529	630	0.493	3.837	3.721	3.992
675	0.557	675	0.587	675	0.511	3.786	3.592	4.127
720	0.601	720	0.628	720	0.562	3.743	3.582	4.002
765	0.665	765	0.706	765	0.605	3.594	3.385	3.950
810	0.758	810	0.798	810	0.701	3.338	3.171	3.610
855	0.847	855	0.897	855	0.794	3.154	2.978	3.364
900	0.956	900	1.014	900	0.899	2.941	2.773	3.128
945	1.106	945	1.181	945	1.023	2.669	2.500	2.886

990	1.318	990	1.429	990	1.207	2.347	2.164	2.562
1035	1.582	1035	1.680	1035	1.442	2.044	1.925	2.242
1080	1.911	1077	collaps	1080	1.704	1.766	ERR	1.980
1083	collapse			1116	collaps	ERR		ERR

Average Inertia thru w = 720 #/ft 3.673
Standard Deviation 0.3048
Variance 0.0929

7.1 Section Name: GG-24SL-12-06 Load: Vertical

w1	delta1	w2	delta2	I1	I2
40	0.091	40	0.097	1.373	1.288
80	0.167	80	0.174	1.497	1.436
120	0.226	120	0.234	1.659	1.602
160	0.313	160	0.331	1.597	1.510
200	0.382	200	0.404	1.636	1.547
240	0.457	240	0.480	1.641	1.562
280	0.529	280	0.562	1.654	1.556
320	0.590	320	0.619	1.694	1.615
360	0.669	360	0.680	1.681	1.654
400	0.772	400	0.796	1.619	1.570
440	0.867	440	0.882	1.585	1.558
480	0.965	480	0.993	1.554	1.510
520	1.099	520	1.121	1.478	1.449
560	1.223	560	1.268	1.430	1.380
600	1.371	600	1.429	1.367	1.312
640	1.554	640	1.662	1.287	1.203
675	collaps	666	collaps	ERR	ERR

Average Inertia thru w = 440 #/ft 1.570
Standard Deviation 0.0980
Variance 0.0096

7.2 Section Name: GG-24SL-12-06 Load: Horizontal

w1	delta1	w2	delta2	I1	I2
37.5	0.081	37.5	0.080	1.446	1.464
75.0	0.133	75.0	0.128	1.762	1.831
112.5	0.192	112.5	0.176	1.831	1.997
150.0	0.240	150.0	0.219	1.953	2.140
187.5	0.298	187.5	0.290	1.966	2.020
225.0	0.334	225.0	0.313	2.105	2.246
262.5	0.370	262.5	0.349	2.216	2.350
300.0	0.402	300.0	0.374	2.331	2.506
337.5	0.429	337.5	0.417	2.458	2.528
375.0	0.476	375.0	0.451	2.461	2.598
412.5	0.535	412.5	0.502	2.409	2.567
450.0	0.597	450.0	0.555	2.355	2.533
487.5	0.673	487.5	0.620	2.263	2.456
525.0	0.800	525.0	0.736	2.050	2.228
562.5	0.946	562.5	0.868	1.858	2.025
600.0	1.156	600.0	1.067	1.621	1.757
637.5	1.443	637.5	1.327	1.380	1.501
651.0	Collaps	675.0	1.766	ERR	1.194
		679	Collapse		ERR

Average Inertia thru w = 450 #/ft 2.278
Standard Deviation 0.2308
Variance 0.0533

8.1 Section Name: GG-30SL-12-06 Load: Horizontal

w1	delta1	w2	delta2	w3	delta3	I1	I2	I3
135	0.121	135	0.115	135	0.099	3.486	3.667	4.260
180	0.157	180	0.151	180	0.141	3.582	3.724	3.988
225	0.179	225	0.183	225	0.176	3.927	3.841	3.994
270	0.234	270	0.229	270	0.197	3.605	3.683	4.282
315	0.289	315	0.270	315	0.223	3.405	3.645	4.413
360	0.321	360	0.303	360	0.249	3.504	3.712	4.517
405	0.362	405	0.328	405	0.302	3.495	3.857	4.190
450	0.381	450	0.342	450	0.328	3.690	4.111	4.286
495	0.402	495	0.371	495	0.347	3.847	4.168	4.457
540	0.428	540	0.394	540	0.369	3.942	4.282	4.572
585	0.453	585	0.417	585	0.400	4.034	4.383	4.569
630	0.477	630	0.444	630	0.427	4.126	4.433	4.609
675	0.532	675	0.483	675	0.458	3.964	4.366	4.604
720	0.566	720	0.526	720	0.493	3.974	4.276	4.563
765	0.618	765	0.579	765	0.544	3.867	4.128	4.393
810	0.694	810	0.633	810	0.596	3.646	3.998	4.246
855	0.777	855	0.719	855	0.657	3.438	3.715	4.066
900	0.862	900	0.818	900	0.763	3.262	3.437	3.685
945	0.987	945	0.931	945	0.871	2.991	3.171	3.389
990	1.158	990	1.081	990	0.996	2.671	2.861	3.105
1035	1.355	1035	1.274	1035	1.137	2.386	2.538	2.844
1080	1.637	1080	1.476	1080	1.309	2.061	2.286	2.578
1101	collaps	1111	collaps	1124	collaps	ERR	ERR	ERR

Average Inertia thru w = 765 #/ft 4.054
Standard Deviation 0.3489
Variance 0.1217

9.1 Section Name: GG-18SL-12-06 Load: Horizontal

w1	delta1	w2	delta2	w3	delta3	I1	I2	I3
37.5	0.073	37.5	0.071	37.5	0.069	1.605	1.650	1.698
75.0	0.116	75.0	0.113	75.0	0.112	2.020	2.073	2.092
112.5	0.173	112.5	0.169	112.5	0.158	2.032	2.080	2.224
150.0	0.221	150.0	0.214	150.0	0.203	2.120	2.190	2.308
187.5	0.264	187.5	0.251	187.5	0.237	2.219	2.334	2.472
225.0	0.317	225.0	0.299	225.0	0.284	2.217	2.351	2.475
262.5	0.351	262.5	0.323	262.5	0.326	2.336	2.539	2.516
300.0	0.390	300.0	0.375	300.0	0.343	2.403	2.499	2.732
337.5	0.427	337.5	0.404	337.5	0.386	2.469	2.610	2.732
375.0	0.481	375.0	0.450	375.0	0.419	2.436	2.603	2.796
412.5	0.526	412.5	0.508	412.5	0.467	2.450	2.537	2.759
450.0	0.595	450.0	0.559	450.0	0.520	2.363	2.515	2.704
487.5	0.661	487.5	0.627	487.5	0.585	2.304	2.429	2.603
525.0	0.794	525.0	0.738	525.0	0.677	2.066	2.222	2.423
562.5	0.958	562.5	0.869	562.5	0.810	1.834	2.022	2.169
600.0	1.152	600.0	1.090	600.0	0.998	1.627	1.720	1.878
637.5	1.406	637.5	1.321	637.5	1.231	1.416	1.508	1.618
660.0	collaps	670.6	collaps	675.0	1.502	ERR	ERR	1.404

677 collapse ERR

Average Inertia thru w = 337.5 #/ft 2.374
Standard Deviation 0.1934
Variance 0.0374

10.1 Section Name: GG-12SL-12-06 Load: Vertical

w1	delta1	w2	delta2	w3	delta3	I1	I2	I3
15	0.057	15	0.060	15	0.056	0.822	0.781	0.837
30	0.116	30	0.122	30	0.117	0.808	0.768	0.801
45	0.168	45	0.177	45	0.170	0.837	0.794	0.827
60	0.225	60	0.248	60	0.220	0.833	0.756	0.852
75	0.274	75	0.296	75	0.266	0.855	0.792	0.881
90	0.339	90	0.367	90	0.331	0.829	0.766	0.849
105	0.401	105	0.428	105	0.389	0.818	0.766	0.843
120	0.456	120	0.491	120	0.448	0.822	0.779	0.837
135	0.512	135	0.543	135	0.502	0.824	0.777	0.840
150	0.567	150	0.592	150	0.554	0.826	0.792	0.846
165	0.631	165	0.673	165	0.620	0.817	0.766	0.831
180	0.700	180	0.732	180	0.681	0.803	0.768	0.826
195	0.778	195	0.819	195	0.763	0.783	0.744	0.798
210	0.876	210	0.936	210	0.852	0.749	0.701	0.770
225	0.981	225	1.077	225	0.934	0.717	0.653	0.753
240	1.128	240	1.208	240	1.089	0.665	0.621	0.688
255	1.258	255	1.351	255	1.221	0.633	0.590	0.652
265	collaps	262.5	collaps	269	collaps	ERR	ERR	ERR

Not used in Calc. of I - o.k. → 3

Average Inertia thru w = 180 #/ft 0.813
Standard Deviation 0.0309
Variance 0.0010

10.2 Section Name: GG-12SL-12-06 Load: Horizontal

w1	delta1	w2	delta2	w3	delta3	I1	I2	I3
56.25	0.100	56.25	0.101	56.25	0.103	1.757	1.740	1.706
75.00	0.131	75.00	0.132	75.00	0.135	1.789	1.775	1.736
93.75	0.174	93.75	0.177	93.75	0.179	1.683	1.655	1.636
112.50	0.214	112.50	0.216	112.50	0.221	1.642	1.627	1.590
131.25	0.243	131.25	0.248	131.25	0.255	1.687	1.653	1.608
150.00	0.297	150.00	0.303	150.00	0.311	1.578	1.547	1.507
168.75	0.341	168.75	0.347	168.75	0.360	1.546	1.519	1.464
187.50	0.382	187.50	0.389	187.50	0.402	1.533	1.506	1.457
206.25	0.447	206.25	0.456	206.25	0.471	1.441	1.413	1.368
225.00	0.509	225.00	0.521	225.00	0.539	1.381	1.349	1.304
243.75	0.593	243.75	0.601	243.75	0.622	1.284	1.267	1.224
262.50	0.674	262.50	0.689	262.50	0.715	1.217	1.190	1.147
281.25	0.778	281.25	0.792	281.25	0.829	1.129	1.109	1.060
300.00	0.866	300.00	0.899	300.00	0.932	1.082	1.043	1.006
318.75	0.971	318.75	1.009	318.75	1.061	1.026	0.987	0.939
337.50	1.096	337.50	1.117	337.50	1.183	0.962	0.944	0.891
356.25	1.221	356.25	1.268	356.25	1.334	0.912	0.878	0.834
375.00	1.372	375.00	1.416	375.00	1.485	0.854	0.827	0.789
393.75	1.559	393.75	1.606	393.75	1.713	0.789	0.766	0.718
412.50	1.805	412.50	1.867	412.50	1.984	0.714	0.690	0.650
420.00	collaps	426.00	collaps	429.40	collaps	ERR	ERR	ERR

3

Not used in Calc. of I - ∴ o.k.

By: RKD 10/31/85 Chk: A. Saha 11.5.85.

Average Inertia thru w = 243.75 #/ft 1.499
Standard Deviation 0.1342
Variance 0.0180

11.1 Section Name: GG-06SL-12-06 Load: Horizontal

Average of Defl. used - no effect in calc. of I

w1	delta1	w2	delta2	w3	delta3	I1	I2	I3
37.5	0.096	37.5	0.096	37.5	0.094	1.220	1.220	1.246
56.25	0.147	56.25	0.149	56.25	0.143	1.195	1.179	1.229
75.00	0.188	75.00	0.191	75.00	0.180	1.246	1.227	1.302
93.75	0.225	93.75	0.230	93.75	0.219	1.302	1.273	1.337
112.50	0.269	112.50	0.278	112.50	0.263	1.307	1.264	1.336
131.25	0.327	131.25	0.332	131.25	0.318	1.254	1.235	1.289
150.00	0.400	150.00	0.399	150.00	0.381	1.172	1.174	1.230
168.75	0.470	168.75	0.483	168.75	0.469	1.122	1.091	1.124
187.50	0.559	187.50	0.580	187.50	0.542	1.048	1.010	1.081
206.25	0.678	206.25	0.715	206.25	0.653	0.950	0.901	0.987
225.00	0.784	225.00	0.809	225.00	0.748	0.897	0.869	0.940
243.75	0.919	243.75	0.967	243.75	0.880	0.829	0.787	0.865
262.50	1.070	262.50	1.133	262.50	1.031	0.766	0.724	0.795
281.25	1.218	281.25	1.286	281.25	1.179	0.721	0.683	0.745
300.00	1.371	300.00	1.438	300.00	1.296	0.684	0.652	0.723
318.00	collaps	317.50	collaps	318.75	1.403	ERR	ERR	0.710
				324.4	collapse			ERR

Average Inertia thru w = 131.25 #/ft 1.259
Standard Deviation 0.0444
Variance 0.0020

12.1 Section Name: GF-36SL-12 Load: Vertical

w1	delta1	w2	delta2	w3	delta3	I1	I2	I3
50	0.083	50	0.091	50	0.082	1.882	1.717	1.905
100	0.162	100	0.169	100	0.160	1.928	1.849	1.953
150	0.220	150	0.229	150	0.218	2.130	2.046	2.150
200	0.287	200	0.298	200	0.283	2.177	2.097	2.208
250	0.348	250	0.351	250	0.342	2.244	2.225	2.284
300	0.422	300	0.435	300	0.413	2.221	2.155	2.269
350	0.514	350	0.527	350	0.501	2.127	2.075	2.182
400	0.603	400	0.620	400	0.589	2.072	2.016	2.122
450	0.702	450	0.731	450	0.684	2.003	1.923	2.055
500	0.797	500	0.818	500	0.778	1.960	1.910	2.008
550	0.901	550	0.923	550	0.876	1.907	1.862	1.961
600	1.009	600	1.034	600	0.978	1.858	1.813	1.917
650	1.128	650	1.157	650	1.098	1.800	1.755	1.849
700	1.261	700	1.284	700	1.223	1.734	1.703	1.788
750	1.383	750	1.422	750	1.341	1.694	1.648	1.747
800	1.548	800	1.606	800	1.493	1.615	1.556	1.674
850	1.746	850	1.846	850	1.686	1.521	1.438	1.575
900	1.982	898	collaps	900	1.914	1.419	ERR	1.469
931	collapse			946	collaps	ERR		ERR

Average Inertia thru w = 600 #/ft 2.066
Standard Deviation 0.1313
Variance 0.0172

13.1 Section Name: GF-30SL-12 Load: Horizontal

w1	delta1	w2	delta2	w3	delta3	I1	I2	I3
225	0.186	225	0.191	225	0.204	3.779	3.680	3.446
270	0.226	270	0.232	270	0.246	3.732	3.636	3.429
315	0.255	315	0.265	315	0.278	3.859	3.714	3.540
360	0.289	360	0.301	360	0.316	3.892	3.736	3.559
405	0.320	405	0.336	405	0.349	3.954	3.766	3.625
450	0.360	450	0.379	450	0.395	3.905	3.709	3.559
495	0.395	495	0.416	495	0.435	3.915	3.717	3.555
540	0.427	540	0.451	540	0.471	3.951	3.741	3.582
585	0.463	585	0.488	585	0.512	3.947	3.745	3.569
630	0.511	630	0.542	630	0.569	3.852	3.631	3.459
675	0.562	675	0.596	675	0.632	3.752	3.538	3.337
720	0.593	720	0.631	720	0.676	3.793	3.565	3.327
765	0.673	765	0.714	765	0.768	3.551	3.347	3.112
810	0.721	810	0.766	810	0.824	3.510	3.304	3.071
855	0.781	855	0.829	855	0.896	3.420	3.222	2.981
900	0.852	900	0.908	900	0.983	3.300	3.097	2.860
945	0.936	945	0.998	945	1.087	3.154	2.958	2.716
990	1.044	990	1.112	990	1.214	2.962	2.781	2.548
1035	1.237	1035	1.309	1026	collaps	2.614	2.470	ERR
1070	collaps	1042	collapse			ERR	ERR	

Average of Defl. used - no effect on I.

Average Inertia thru w = 630 #/ft 3.706
Standard Deviation 0.1534
Variance 0.0235

14.1 Section Name: GF-24SL-12 Load: Horizontal

w1	delta1	w2	delta2	w3	delta3	I1	I2	I3
225	0.123	225	0.122	225	0.127	5.715	5.762	5.535
270	0.164	270	0.160	270	0.169	5.143	5.272	4.991
315	0.203	315	0.200	315	0.211	4.848	4.920	4.664
360	0.227	360	0.224	360	0.237	4.954	5.021	4.745
405	0.272	405	0.266	405	0.286	4.652	4.757	4.424
450	0.312	450	0.304	450	0.329	4.506	4.624	4.273
495	0.346	495	0.336	495	0.364	4.469	4.602	4.248
540	0.370	540	0.358	540	0.393	4.559	4.712	4.293
585	0.409	585	0.395	585	0.436	4.468	4.627	4.192
630	0.431	630	0.414	630	0.460	4.567	4.754	4.279
675	0.495	675	0.475	675	0.529	4.260	4.439	3.986
720	0.530	720	0.506	720	0.564	4.244	4.445	3.988
765	0.576	765	0.548	765	0.613	4.149	4.361	3.899
810	0.621	810	0.592	810	0.661	4.075	4.274	3.828
855	0.702	855	0.67	855	0.745	3.805	3.987	3.585
900	0.764	900	0.729	900	0.809	3.680	3.857	3.475
945	0.853	945	0.816	945	0.901	3.461	3.618	3.277
990	0.959	990	0.917	990	1.018	3.225	3.373	3.038
1035	1.069	1035	1.022	1035	1.157	3.025	3.164	2.795
1080	1.212	1080	1.16	1059	collaps	2.784	2.909	ERR
1125	1.349	1125	1.292			2.605	2.720	
1126	collaps	1139	collapse			ERR	ERR	

Average Inertia thru w = 675 #/ft 4.550
Standard Deviation 0.2494

Variance 0.0622

15.1 Section Name: GF-18SL-12 Load: Vertical

w1	delta1	w2	delta2	w3	delta3	I1	I2	I3
30	0.072	30	0.074	30	0.072	1.302	1.267	1.302
60	0.134	60	0.137	60	0.132	1.399	1.368	1.420
90	0.213	90	0.216	90	0.209	1.320	1.302	1.345
120	0.270	120	0.274	120	0.265	1.388	1.368	1.415
150	0.342	150	0.346	150	0.337	1.370	1.354	1.391
180	0.417	180	0.420	180	0.411	1.349	1.339	1.368
210	0.489	210	0.493	210	0.481	1.342	1.331	1.364
240	0.572	240	0.577	240	0.563	1.311	1.299	1.332
270	0.648	270	0.653	270	0.636	1.302	1.292	1.326
300	0.729	300	0.738	300	0.716	1.286	1.270	1.309
330	0.826	330	0.832	330	0.813	1.248	1.239	1.268
360	0.902	360	0.911	360	0.890	1.247	1.235	1.264
390	1.006	390	1.017	390	0.992	1.211	1.198	1.228
420	1.091	420	1.106	420	1.074	1.203	1.186	1.222
450	1.197	450	1.208	450	1.180	1.174	1.164	1.191
480	1.293	480	1.308	480	1.274	1.160	1.146	1.177
510	1.395	510	1.410	510	1.373	1.142	1.130	1.160
540	1.501	540	1.516	540	1.477	1.124	1.113	1.142
570	1.630	570	1.647	570	1.604	1.092	1.081	1.110
600	1.763	600	1.781	600	1.735	ERR	1.052	1.080
607	collaps	603	collaps	607	collaps	ERR	ERR	ERR

Average Inertia thru w = 420 #/ft 1.307
Standard Deviation 0.0614
Variance 0.0038

15.2 Section Name: GF-18SL-12 Load: Horizontal

w1	delta1	w2	delta2	w3	delta3	I1	I2	I3
37.5	0.095	37.5	0.101	37.5	0.098	1.233	1.160	1.195
75.0	0.179	75.0	0.187	75.0	0.184	1.309	1.253	1.273
112.5	0.268	112.5	0.275	112.5	0.272	1.311	1.278	1.292
150.0	0.361	150.0	0.369	150.0	0.367	1.298	1.270	1.277
187.5	0.468	187.5	0.478	187.5	0.476	1.252	1.225	1.231
225.0	0.567	225.0	0.583	225.0	0.578	1.240	1.206	1.216
262.5	0.692	262.5	0.709	262.5	0.704	1.185	1.157	1.165
300.0	0.804	300.0	0.815	300.0	0.808	1.166	1.150	1.160
337.5	0.931	337.5	0.943 S	337.5	0.936	1.133	1.118	1.126
375.0	1.035	375.0	1.049	375.0	1.042	1.132	1.117	1.124
412.5	1.161	412.5	1.181	412.5	1.169	1.110	1.091	1.102
450.0	1.311	450.0	1.334	450.0	1.321	1.072	1.054	1.064
487.5	1.438	487.5	1.463	487.5	1.447	1.059	1.041	1.053
525.0	1.596	525.0	1.637	525.0	1.599	1.028	1.002	1.026
562.5	1.776	547.0	collaps	562.5	1.784	0.989	ERR	0.985
570.0	collapse			565.0	collaps	ERR		ERR

Average of defl. used in calc. of I - ∴ O.K.

Average Inertia thru w = 450 #/ft 1.187
Standard Deviation 0.0743
Variance 0.0055

By: RFO 10/31/85 Chk: A. Saha 11.5.85

16.1 Section Name: GF-12SL-12 Load: Horizontal

w1	delta1	w2	delta2	w3	delta3	I1	I2	I3
37.5	0.094	37.5	0.099	37.5	0.103	1.246	1.183	1.137
75.0	0.180	75.0	0.183	75.0	0.186	1.302	1.280	1.260
112.5	0.269	112.5	0.274	112.5	0.279	1.307	1.283	1.260
150.0	0.362	150.0	0.368	150.0	0.374	1.295	1.273	1.253
187.5	0.464	187.5	0.472	187.5	0.481	1.262	1.241	1.218
225.0	0.558	225.0	0.569	225.0	0.577	1.260	1.235	1.218
262.5	0.676	262.5	0.690	262.5	0.699	1.213	1.189	1.173
300.0	0.785	300.0	0.798	300.0	0.811	1.194	1.174	1.156
337.5	0.903	337.5	0.924	337.5	0.935	1.168	1.141	1.128
375.0	1.011	375.0	1.027	375.0	1.063	1.159	1.141	1.102
412.5	1.145	412.5	1.156	412.5	1.188	1.125	1.115	1.085
450.0	1.280	450.0	1.305	450.0	1.331	1.098	1.077	1.056
487.5	1.403	487.5	1.433	487.5	1.469	1.086	1.063	1.037
525.0	1.538	525.0	1.592	525.0	1.632	1.066	1.030	1.005
562.5	1.705	562.5	1.771	558.0	collaps	1.031	0.992	ERR
598.0	collaps	570.0	collapse			ERR	ERR	

Average Inertia thru w = 412.5 #/ft 1.205
Standard Deviation 0.0628
Variance 0.0039

17.1 Section Name: GF-06SL-12 Load: Vertical

w1	delta1	w2	delta2	w3	delta3	I1	I2	I3
15	0.048	15	0.052	15	0.049	0.976	0.901	0.956
30	0.107	30	0.121	30	0.105	0.876	0.775	0.893
45	0.164	45	0.184	45	0.160	0.857	0.764	0.879
80	0.291	80	0.316	80	0.287	0.859	0.791	0.871
122.5	0.460	122.5	0.486	122.5	0.454	0.832	0.787	0.843
170	0.703	170	collaps	170	0.671	0.755	ERR	0.791
187	collapse			192	collaps	ERR		ERR

Average Inertia thru w = 122.5 #/ft 0.857
Standard Deviation 0.0599
Variance 0.0036

17.2 Section Name: GF-06SL-12 Load: Horizontal

w1	delta1	w2	delta2	w3	delta3	I1	I2	I3
4.5	0.018	4.5	0.017	4.5	0.017	0.781	0.827	0.827
24.5	0.119	24.5	0.117	24.5	0.115	0.643	0.654	0.666
44.5	0.209	44.5	0.208	44.5	0.204	0.665	0.668	0.681
64.5	0.293	64.5	0.294	64.5	0.289	0.688	0.685	0.697
84.5	0.381	84.5	0.378	84.5	0.376	0.693	0.698	0.702
103.0	0.482	103.0	0.480	103.0	0.473	0.668	0.670	0.680
122.0	0.590	122.0	0.589	122.0	0.582	0.646	0.647	0.655
131.0	0.645	131.0	0.642	131.0	0.634	0.635	0.637	0.646
141.0	0.714	141.0	0.717	141.0	0.707	0.617	0.614	0.623
150.0	0.798	150.0	0.788	150.0	0.777	0.587	0.595	0.603
159.5	0.881	159.5	0.873	159.5	0.859	0.566	0.571	0.580
169.0	0.938	169.0	0.935	169.0	0.922	0.563	0.565	0.573
178.0	1.056	178.0	1.021	178.0	0.988	0.527	0.545	0.563
188.0	1.191	188.0	1.178	188.0	1.115	0.493	0.499	0.527
197.0	1.293	197.0	1.249	197.0	1.138	0.476	0.493	0.541

206.5	1.374	206.5	1.345	206.5	1.221	0.470	0.480	0.528
216.0	1.489	216.0	1.476	216.0	1.298	0.453	0.457	0.520
225.0	1.583	225.0	1.510	225.0	1.436	0.444	0.466	0.489
234.5	1.692	234.5	1.632	234.5	1.461	0.433	0.449	0.501
244.0	1.799	244.0	1.730	244.0	1.58	0.424	0.441	0.482
246.0	collaps	251.0	collaps	246.0	1.698	ERR	ERR	0.453
				253.0	collapse			ERR

Average Inertia thru w = 169 #/ft	0.653
Standard Deviation	0.0638
Variance	0.0041

By: RKO 10/31/85 Chk: A. Saha 11.5.85

ATTACHMENT 3:

LETTER FROM J. F. CURRIE (OF T.J. COPE, INC.)
WITH TEST DATA AND RESULTS.

UNCONTROLLED COPY

					TUGCO	
					Mom of Inertia - Cable Trays	
					JOB NO 0210-040	PAGE —
0	A. Saha	12.2.85	[illegible]	12.5.85	CALC NO M-03	OF —
REV	BY	DATE	CHECKED	DATE	IMPELL CORPORATION	

T. J. Cope, Inc.

COLLEGEVILLE, PA 19426
TEL. (215)489-4200
TELEX 84-6486

February 3, 1982

GIBBS & HILL, INCORPORATED
393, 7th Avenue
New York, New York 10001

Attention: Mr. R. Manuelyan

Subject: Texas Utilities Generating Company, G. & H. Project - 2323
Cable Tray Qualification CP-0579

Dear Mr. Manuelyan:

We enclose your recently-requested package of information to add to the Test Reports and Data submitted on March 3, 1978.

Of the seventeen Ladder and Trof systems listed in the Data and Calculations submittal, we enclose the (nine) systems you requested that we re-submit:

✓GG-18SL-12-06	✓GF-30SL-12	✓JM-30SL-12
✓GG-36SL-12-06	✓GI-30SL-12-06	✓JM-24SL-12
✓GF-06SL-12	✓JM-36SL-12	✓JM-18SL-12

On these re-submittal sheets we have added the Ix from the known tray configuration, and the apparent Iy from the respective test results, of each tray system. These are marked "Δ".

We also re-submit as requested the (eight) remaining Test Data sheets with the added Ix from the design, and apparent Iy from testing, of each respective system:

✓GG-06SL-12-06	✓GF-12SL-12
✓GG-12SL-12-06	✓GF-18SL-12
✓GG-24SL-12-06	✓GF-24SL-12
✓GG-30SL-12-06	✓GF-36SL-12

With the calculated Euler's Iy and the apparent Iy from testing, and the Ix known from the sum of the side rails' Ix, we show that the smallest possible moment of inertia or the most conservative figure was used in our calculations.

February 3, 1982

Subject: Letter to Gibbs & Hill, Incorporated - New York
Attention: Mr. R. Manuelyan
Subject: Texas Utilities Generating Company
G. & H. project - 2323
Cable Tray Qualification CP-0579

As for the choice of Euler's Formula in our figures, we chose not only the Light Guage Column Formula for longitudinal loading, including the more conservative constant for hinged-ends rather than fixed-ends, but we considered the "worst-case" of side flanges, toe-to-toe. This is a purely theoretical adverse condition, since we did not include a value for the tray rungs or bottoms, both of which would increase the real Iy value.

Also included in the listed enclosures are the corrected copies of:

GG-06SL-12-06
GG-12SL-12-06

The corrections show the transposed working loads in their correct positions in the formula, and the calculations performed to show no appreciable difference in the results. These are marked "*"

We hope this data is helpful to you in your review of this job and if you have any questions, do not hesitate to call us.

Very truly yours,

John F. Currie
Engineering Manager

JC/bh

CALCULATIONS FOR CYPRUS CABLE TRAY CAT NO. GE-30SL-12
TO COMANCHE PEAK S.E.S. SPECIFICATION # 2323-ES-19

SPECIFICATION LOADING IS 88 LBS/FT.

6 1/4 x 1 1/8 x 12 GA: $I_x = 4.39$ Δ

FROM ATTACHED TEST DATA:

$F_m = 1294 + 1309 + 1333 \div 3 = 1312$

$F_x = 1412 + 1425 + 1477 \div 3 = 1438$

USING EULERS COLUMN FORMULA: $P = \frac{\pi^2 I E}{l^2}$

WHERE: P = ALLOWABLE LOAD LBS

$E = 29.5 \times 10^6$

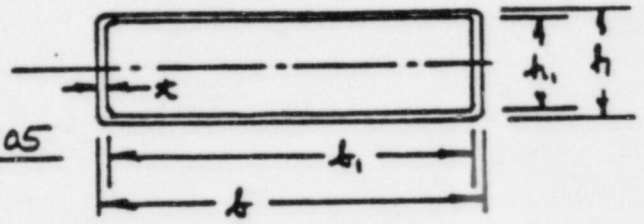
$l = 96''$

$I = 1/2 [b h^3 - b_1 h_1^3]$

$b = 6.25$ $h = 2.5$ $t = .105$

$I = 2.09$

2.354, 2.410, 2.508 } 2.424 Δ



$F_l = \frac{P}{8} = \frac{\pi^2 \times 2.09 \times 29.5 \times 10^6}{8 \times 96^2} = 8247$

THE FOLLOWING CRITERIA MUST BE MET:

$$1.6\left[\frac{f'_m}{F_m} + \sqrt{\left(\frac{f_m}{F_m}\right)^2 + \left(\frac{f_x}{F_x}\right)^2 + \left(\frac{f_l}{F_l}\right)^2}\right] \leq 1.0$$

CYPRUS

WHERE f'_m = SPECIFICATION LOADING
f_m = 2.5 x SPECIFICATION LOADING
f_x = 4.0 x SPECIFICATION LOADING
f_l = 4.0 x SPECIFICATION LOADING

$$\frac{88}{1312} + \sqrt{\left(\frac{2.5 \times 88}{1312}\right)^2 + \left(\frac{4.0 \times 88}{1438}\right)^2 + \left(\frac{4.0 \times 88}{8247}\right)^2} \leq .625$$

$$.067 + \sqrt{.028 + .060 + .0018} \leq .625$$

$$.067 + .300 \leq .625$$

$$.367 \leq .625 \therefore \text{OK}$$

TRAY DESCRIPTION: GI-303L-12

LADDER 6 1/4 x 1 1/4 12GA · 50KSI 30" WIDE

166A RUNG CONN AT MIDSPAN

01-CP-12

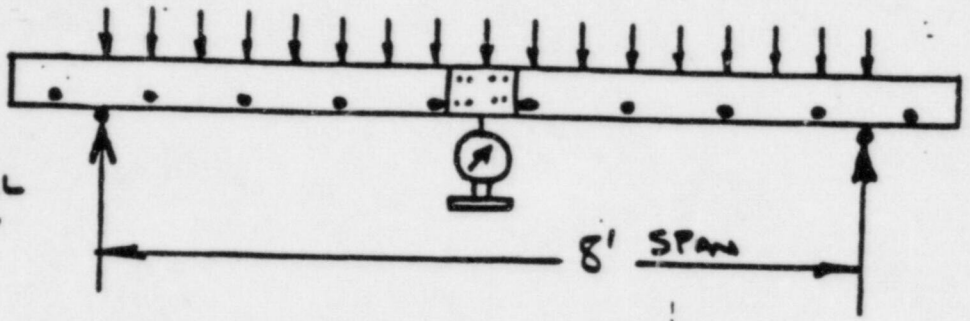
TEST NO.

2-7-7[illegible]

DATE

DB

BY



VERTICAL TEST

NO. OF BUNDLES	TOTAL WEIGHT LBS.	LBS/FT.	#1 INDICATOR READING	#1 INDICATOR Δ"	#2 INDICATOR READING	#2 INDICATOR Δ"	MAX AVERAGE Δ"	COMMENTS
0	0	0	4.215	0	4.306	0	0	
80	800	100	4.065	.150	4.144	.162	.156	
160	1600	200	3.919	.296	3.990	.316	.306	
240	2400	300	3.778	.437	3.838	.468	.453	
320	3200	400	3.679	.536	3.721	.585	.561	
400	4000	500	3.536	.679	3.561	.745	.712	
480	4800	600	3.431	.784	3.441	.865	.825	
560	5600	700	3.325	.890	3.310	.996	.943	
640	6400	800	3.192	1.023	3.152	1.154	1.089	
720	7200	900	3.057	1.158	2.990	1.316	1.237	
800	8000	1000	2.866	1.349	2.771	1.535	1.442	
880	8800	1100	2.629	1.586	2.612	1.694	1.694	
960	9600	1200	2.351	1.864	2.189	2.117	1.991	
1035	10350	1294	—	—	—	—	—	COLLAPSE

CYPRUS

5

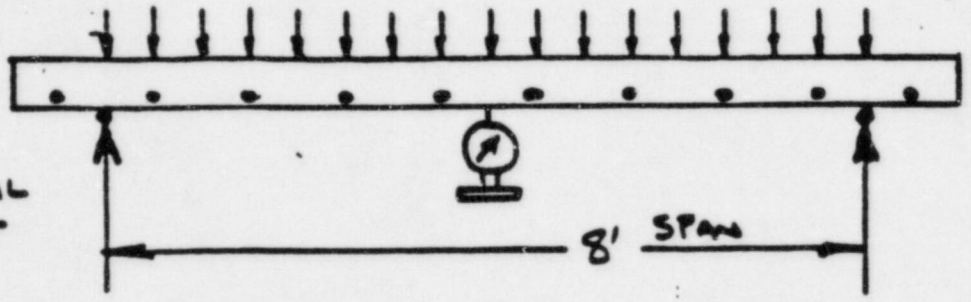
TRAY DESCRIPTION: 61-30SL-12 01-CP-33

30"w LADDER 6 1/4 x 1 1/4 12GA - 50KSI TEST NO.

16GA rung Conn @ MID-SPAN 2-8-78

DATE

BY



VERTICAL TEST

No. of Bundles	Total Weight Lbs.	Lbs/Ft.	#1 Indicator Reading	#1 Indicator Δ"	#2 Indicator Reading	#2 Indicator Δ"	Max Average Δ"	Comments
0	0	0	4.570	0	4.683	0	0	
80	800	100	4.431	.139	4.538	.145	.142	
160	1600	200	4.277	.293	4.382	.301	.297	
240	2400	300	4.151	.419	4.250	.433	.426	
320	3200	400	3.999	.571	4.093	.590	.581	
400	4000	500	3.883	.687	3.971	.712	.700	
480	4800	600	3.775	.795	3.860	.823	.809	
560	5600	700	3.658	.912	3.740	.943	.928	
640	6400	800	3.537	1.033	3.610	1.073	1.053	
720	7200	900	3.404	1.166	3.466	1.217	1.192	
800	8000	1000	3.205	1.365	3.253	1.430	1.398	
880	8800	1100	2.969	1.601	3.002	1.681	1.641	
960	9600	1200	2.718	1.852	2.727	1.956	1.904	
1040	10400	1300	2.291	2.279	2.282	2.401	2.340	
1047	10470	1309	—	—	—	—	—	COLLAPSE

CERTIFIED CYPRUS

TRAY DESCRIPTION: GI-BOSL-12

30"W 6"-x 1/4 LADDER RGA-50KSI

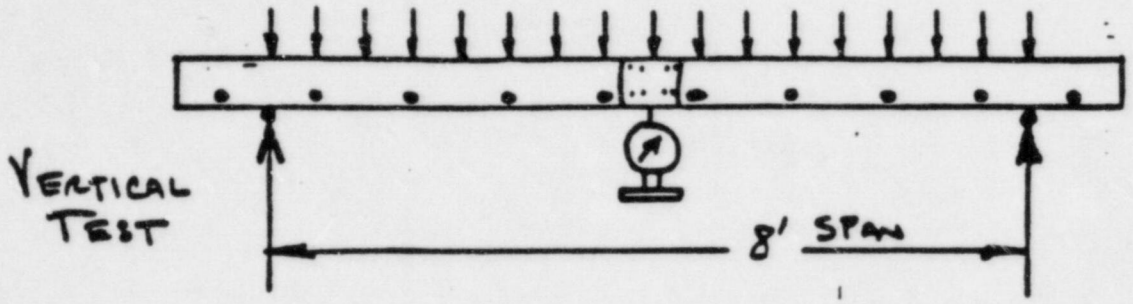
COMM at MIDSPAN

01-C[illegible]4

TEST NO. 2-9-7[illegible]

DATE [illegible]

BY



NO. OF BUNDLES	TOTAL WEIGHT LBS.	LBS/FT.	#1 INDICATOR READING	#1 INDICATOR Δ"	#2 INDICATOR READING	#2 INDICATOR Δ"	MAX AVERAGE Δ"	COMMENTS
0	0	0	4.736	0	4.434	0	0	
80	800	100	4.604	.132	4.306	.128	.130	
160	1600	200	4.439	.297	4.143	.291	.294	
240	2400	300	4.315	.421	4.018	.416	.419	
320	3200	400	4.160	.576	3.870	.564	.570	
400	4000	500	4.041	.695	3.755	.679	.687	
480	4800	600	3.933	.803	3.651	.783	.793	
560	5600	700	3.815	.921	3.539	.895	.908	
640	6400	800	3.693	1.043	3.426	1.008	1.026	
720	7200	900	3.552	1.184	3.300	1.134	1.159	
800	8000	1000	3343	1393	3.107	1.327	1.360	
880	8800	1100	3.096	1.640	2.885	1.549	1.595	
960	9600	1200	2.925	1.911	2.641	1.793	1.852	
1040	10400	1300	2.375	2361	2.229	2.205	2.283	
1066	10660	1333	—	—	—	—	—	COLLAPSE

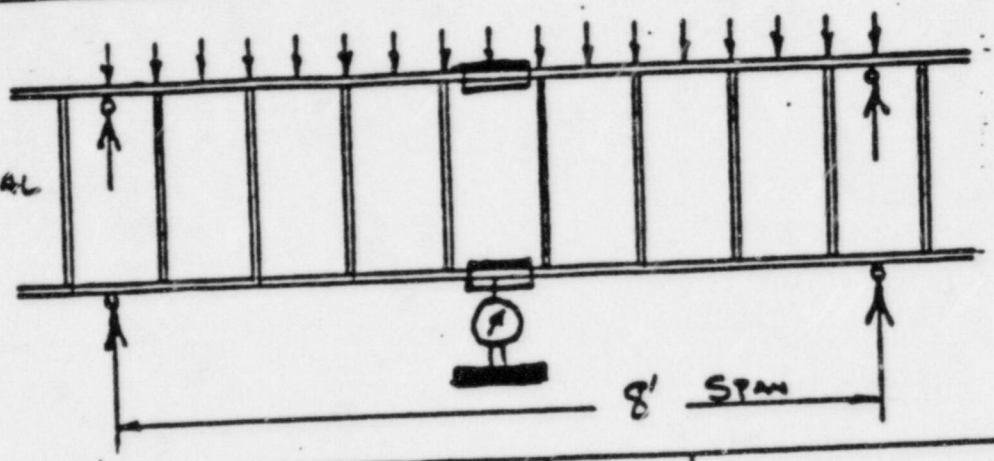
CYPRUS

CC

TRAY DESCRIPTION: GI-30SL-12
30"W LADDER 6 1/4 X 1 1/4 - 12GA 50KST
16GA rung CONN at MID-SPAN

01-CP-21
TEST NO.
2-3-78
DATE
BB
BY

HORIZONTAL TEST



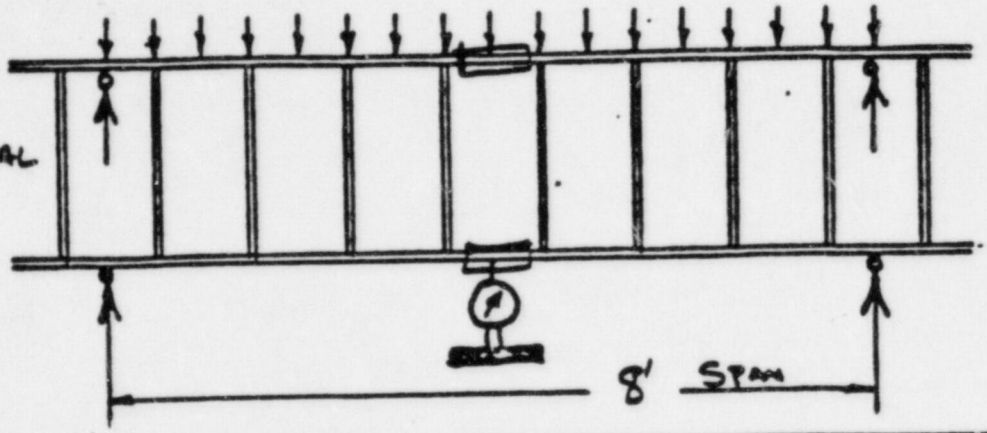
No. of Bundles	Total Weight Lbs.	Total Weight Lbs/2	Lbs/Ft	Indicator Reading	Δ"	Comments
0	0	0	0	4.629	0	
120	1200	600	75	4.493	.136	
240	2400	1200	150	4.398	.231	
360	3600	1800	225	4.302	.327	
480	4800	2400	300	4.193	.436	
600	6000	3000	375	4.099	.530	
720	7200	3600	450	4.017	.612	
840	8400	4200	525	3.921	.708	
960	9600	4800	600	3.850	.779	
1080	10800	5400	675	3.786	.843	
1200	12000	6000	750	3.709	.920	
1320	13200	6600	825	3.643	.986	
1440	14400	7200	900	3.567	1.062	
1560	15600	7800	975	3.484	1.145	
1680	16800	8400	1050	3.355	1.274	
1800	18000	9000	1125	3.268	1.361	
1920	19200	9600	1200	3.115	1.514	
2040	20400	10200	1275	2.913	1.716	
2160	21600	10800	1350	2.673	1.956	
2259	22590	11295	1412	—	—	CO'LLAPSE

CERTIFIED
CYPRUS

TRAY DESCRIPTION: GI-308L-12
30"W LADDER 6¼ X 1¼ - 12GA 50KST
16GA rung Conn. @ midspan

01-CP-30
TEST NO.
2-6-78
DATE
BB
BY

HORIZONTAL TEST



No. OF BUNDLES	TOTAL WEIGHT LBS.	TOTAL WEIGHT LBS/2	LBS/FT	INDIMATH READING	Δ"	COMMENTS
0	0	0	0	4.005	0	
120	1200	600	75	3.874	.131	
240	2400	1200	150	3.778	.227	
360	3600	1800	225	3.685	.320	
480	4800	2400	300	3.577	.428	
600	6000	3000	375	3.482	.523	
720	7200	3600	450	3.403	.602	
840	8400	4200	525	3.311	.694	
960	9600	4800	600	3.237	.768	
1080	10800	5400	675	3.174	.831	
1200	12000	6000	750	3.100	.905	
1320	13200	6600	825	3.036	.969	
1440	14400	7200	900	2.959	1.046	
1560	15600	7800	975	2.888	1.117	
1680	16800	8400	1050	2.766	1.239	
1800	18000	9000	1125	2.688	1.317	
1920	19200	9600	1200	2.547	1.458	
2040	20400	10200	1275	2.358	1.647	
2160	21600	10800	1350	2.132	1.873	
2280	22800	11400	1425	—	—	COLLAPSE

CERTIFIED CYPRUS

TRAY DESCRIPTION: GI-30SL42

30" W LADDER 6 1/4 X 1 1/4 - 12GA SUKST

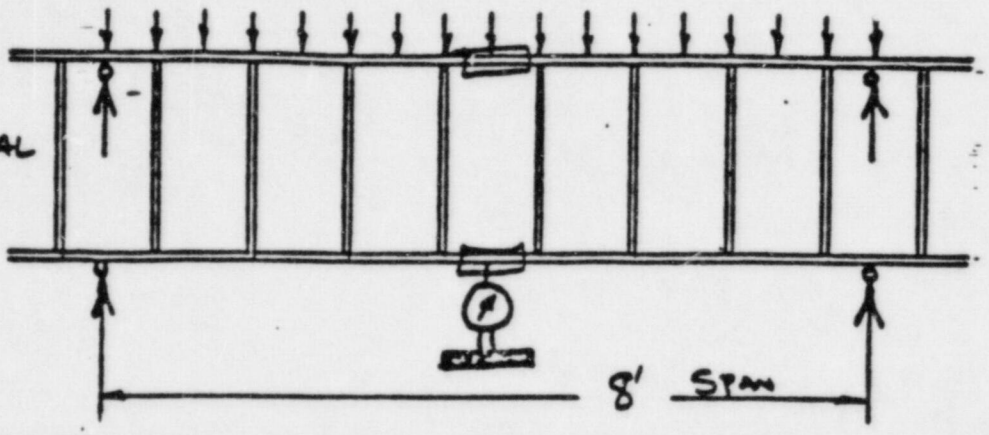
Conn. @ midspan

01-CD-21
TEST NO.

2-6-78
DATE

3B
BY

HORIZONTAL TEST

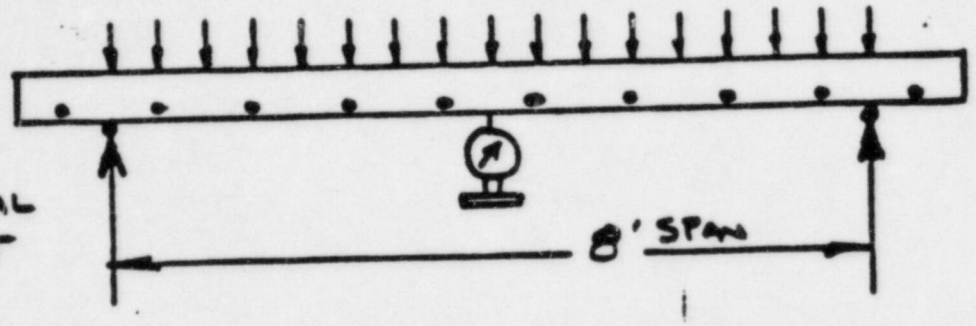


No. OF BUNDLES	TOTAL WEIGHT LBS.	TOTAL WEIGHT LBS/2	LBS/FT	INDICATOR READING	Δ"	COMMENTS
0	0	0	0	4.732	0	
120	1200	600	75	4.606	.126	
240	2400	1200	150	4.512	.220	
360	3600	1800	225	4.421	.311	
480	4800	2400	300	4.316	.416	
600	6000	3000	375	4.224	.508	
720	7200	3600	450	4.147	.585	
840	8400	4200	525	4.059	.673	
960	9600	4800	600	3.991	.741	
1080	10800	5400	675	3.933	.799	
1200	12000	6000	750	3.862	.870	
1320	13200	6600	825	3.904	.928	
1440	14400	7200	900	3.736	.996	
1560	15600	7800	975	3.673	1.059	
1680	16800	8400	1050	3.557	1.175	
1800	18000	9000	1125	3.484	1.248	
1920	19200	9600	1200	3.352	1.380	
2040	20400	10200	1275	3.173	1.559	
2160	21600	10800	1350	2.961	1.771	
2280	22800	11400	1425	2.717	2.015	
2363	23630	11815	1477	—	—	COLLAPSE

CYPRUS

TRAY DESCRIPTION JM-36S-L-'2

01-C-01
TEST NO.
1-13-78
DATE
SR
BY



VERTICAL TEST

NO. OF BUNDLES	TOTAL WEIGHT LBS.	LBS/FT.	#1 INDICATOR READING	#1 INDICATOR Δ"	#2 INDICATOR READING	#2 INDICATOR Δ"	MAX AVERAGE Δ"	COMMENTS
0	0	0	3.926	0	3.754	0	0	
80	800	100	3.729	.197	3.565	.189	.193	
160	1600	200	3.549	.377	3.389	.365	.371	
240	2400	300	3.393	.533	3.238	.516	.525	
320	3200	400	3.262	.664	3.114	.640	.652	
400	4000	500	3.140	.786	3.008	.754	.770	
480	4800	600	3.026	.900	2.899	.855	.878	
560	5600	700	2.927	.999	2.812	.942	.971	
640	6400	800	2.788	1.138	2.692	1.062	1.100	
720	7200	900	2.607	1.319	2.539	1.215	1.267	
800	8000	1000	2.379	1.547	2.345	1.409	1.478	
880	8800	1100	2.156	1.770	2.160	1.594	1.682	
960	9600	1200	1.850	2.076	1.902	1.852	1.964	
1033	10330	1291	—	—	—	—	—	Collapse

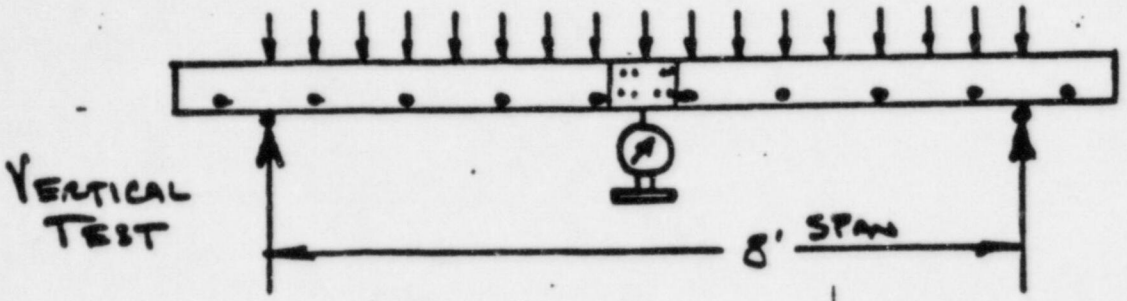
CYPRUS

TRAY DESCRIPTION. JM-365L-12

36" w TROF 12GA 60KSI

6 1/4 x 1 1/4

01-CP-06
TEST NO.
1-18-78
DATE
3B
BY



No. of Bundles	Total Weight Lbs.	Lbs/Ft.	#1 Indicator Reading	#1 Indicator Δ"	#2 Indicator Reading	#2 Indicator Δ"	Max Average Δ"	Comments
0	0	0	4.482	0	3.615	0	0	
80	800	100	4.279	.203	3.425	.190	.197	
160	1600	200	4.093	.389	3.248	.367	.378	
240	2400	300	3.930	.552	3.096	.519	.536	
320	3200	400	3.787	.695	2.974	.641	.668	
400	4000	500	3.646	.836	2.957	.758	.797	
480	4800	600	3.518	.964	2.763	.852	.908	
560	5600	700	3.400	1.082	2.687	.928	1.005	
640	6400	800	3.240	1.242	2.579	1.036	1.139	
720	7200	900	3.163	1.319	2.432	1.183	1.319	
800	8000	1000	2.762	1.720	2.252	1.363	1.542	
880	8800	1100	2.500	1.982	2.083	1.532	1.757	
960	9600	1200	2.145	2.337	1.834	1.789	2.063	
1020	10200	1275	—	—	—	—	—	COLLAPSE

CERTIFIED CYPRUS

TRAY DESCRIPTION JM-36SL-12 01-CP-05

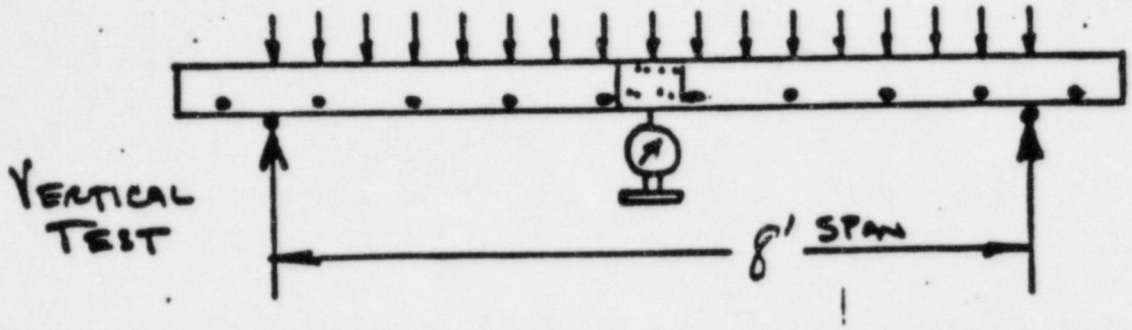
36" W TROF Conn @ midpoint TEST NO.

12 GA - 50 KSI 6 1/4 x 1 1/4 SIDERAILS 1-18-78

DATE

BB

BY



No. of Bundles	Total Weight Lbs.	Lbs/Ft.	#1 Indicator Reading	#1 Indicator Δ"	#2 Indicator Reading	#2 Indicator Δ"	Max Average Δ"	Comments
0	0	0	4.672	0	4.806	0	0	
80	800	100	4.481	.191	4.610	.196	.194	
160	1600	200	4.304	.368	4.428	.378	.373	
240	2400	300	4.152	.520	4.269	.537	.529	
320	3200	400	4.026	.646	4.136	.670	.658	
400	4000	500	3.907	.765	4.006	.800	.783	
480	4800	600	3.806	.866	3.892	.914	.890	
560	5600	700	3.724	.948	3.786	1.020	.984	
640	6400	800	3.607	1.065	3.643	1.163	1.114	
720	7200	900	3.451	1.221	3.458	1.348	1.285	
800	8000	1000	3.253	1.419	3.223	1.583	1.501	
880	8800	1100	3.070	1.602	2.992	1.814	1.708	
960	9600	1200	2.812	1.860	2.678	2.128	1.994	
1031	10310	1289	—	—	—	—	—	Collapse

CERTIFIED CYPRUS

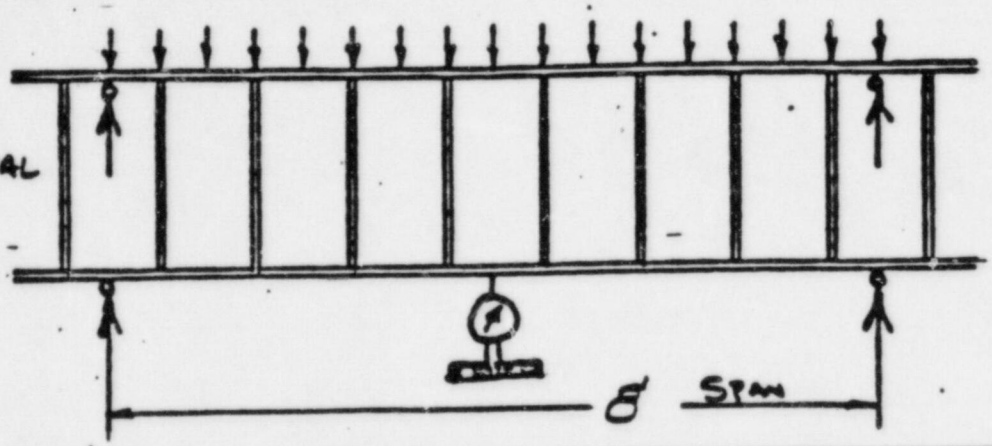
TRAY DESCRIPTION. JM-36SL-12

36" W TROF 6 1/4 x 1 1/4 126A-50KSI

CONN AT MIDSPAN

01-CP-16
TEST NO.
1-30-71
DATE
BB
BY

HORIZONTAL TEST



No. of Bundles	Total Weight Lbs.	Total Weight Lbs/2	Lbs/Ft	Indicator Reading	Δ"	Comments
0	0	0	0	4.815	0	
160	1600	800	100	4.722	.093	
320	3200	1600	200	4.628	.187	
480	4800	2400	300	4.496	.319	
640	6400	3200	400	4.383	.432	
800	8000	4000	500	4.289	.526	
960	9600	4800	600	4.153	.662	
1120	11200	5600	700	4.034	.781	
1280	12800	6400	800	3.956	.859	
1440	14400	7200	900	3.826	.989	
1600	16000	8000	1000	3.717	1.098	
1760	17600	8800	1100	3.588	1.227	
1920	19200	9600	1200	3.419	1.396	
2080	20800	10400	1300	3.198	1.617	
2179	21790	10895	1362	—	—	COLLAPSE

CERTIFIED CYPRUS

CALCULATIONS FOR CYPRUS CABLE TRAY CAT NO. TM-36SL-12
TO COMANCHE PEAK S.E.S. SPECIFICATION # 2323-ES-19
SPECIFICATION LOADING IS 105 LBS/FT.
FROM ATTACHED, TEST DATA:

6 1/4 x 1 1/4 x 12 GA
$I_x = 4.39$ Δ

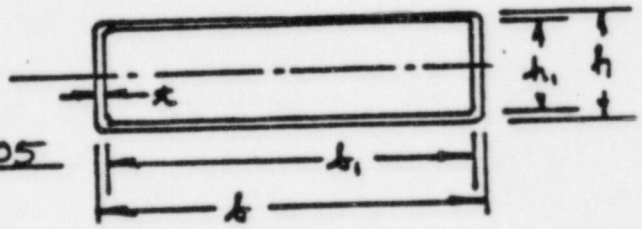
$F_m = \frac{1275 + 1289 + 1291}{3} \div 3 = 1285$

$F_x = \frac{1327 + 1418 + 1362}{3} \div 3 = 1369$

USING EULERS COLUMN FORMULA: $P = \frac{\pi^2 I E}{l^2}$

WHERE: P = ALLOWABLE LOAD LBS
$E = 29.5 \times 10^6$
$l = 96''$
$I = 1/2 [b h^3 - b_1 h_1^3]$
$b = 6.25$ $h = 2.5$ $t = .105$
$I = 2.09$

2.902
2.939
2.744
2.862 Δ



$F_l = \frac{P}{8} = \frac{\pi^2 \times 2.09 \times 29.5 \times 10^6}{8 \times 96^2} = 8247$

THE FOLLOWING CRITERIA MUST BE MET:

$$1.6\left[\frac{f'_m}{F_m} + \sqrt{\left(\frac{f_m}{F_m}\right)^2 + \left(\frac{f_x}{F_x}\right)^2 + \left(\frac{f_l}{F_l}\right)^2}\right] \leq 1.0$$

WHERE f'_m = SPECIFICATION LOADING
f_m = 2.5 x SPECIFICATION LOADING
f_x = 4.0 x SPECIFICATION LOADING
f_l = 4.0 x SPECIFICATION LOADING

CYPRUS

$$\frac{105}{1285} + \sqrt{\left(\frac{2.5 \times 105}{1285}\right)^2 + \left(\frac{4.0 \times 105}{1369}\right)^2 + \left(\frac{4.0 \times 105}{8247}\right)^2} \leq .625$$

$$.082 + \sqrt{.042 + .094 + .0026} \leq .625$$

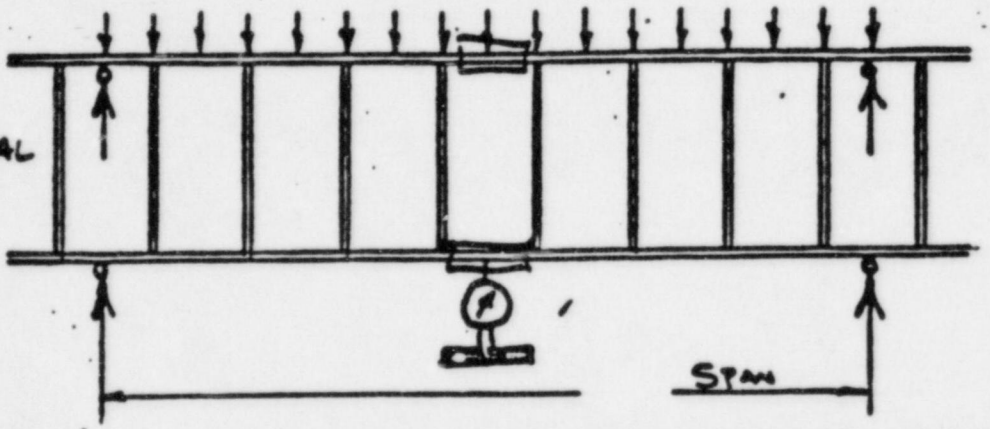
$$.082 + .372 \leq .625$$

$$.454 \leq .625 \therefore \text{OK}$$

TRAY DESCRIPTION: 36SL-12
36" WIDE TROF 6"1/4x1"1/4 12GA-50
conn. at midspan

01-CP
TEST #
1-30-7
DATE
BE
BY

HORIZONTAL TEST



No. OF BUNDLES	TOTAL WEIGHT LBS.	TOTAL WEIGHT LBS/2	LBS/FT	INDICATOR READING	Δ"	COMMENTS
0	0	0	0	4.076	0	
160	1600	800	100	3.985	.091	
320	3200	1600	200	3.892	.184	
480	4800	2400	300	3.763	.313	
640	6400	3200	400	3.650	.426	
800	8000	4000	500	3.558	.518	
960	9600	4800	600	3.423	.653	
1120	11200	5600	700	3.311	.765	
1280	12800	6400	800	3.234	.842	
1440	14400	7200	900	3.116	.960	
1600	16000	8000	1000	3.024	1.052	
1760	17600	8800	1100	2.991	1.185	
1920	19200	9600	1200	2.747	1.329	
2080	20800	10400	1300	2.546	1.530	
2240	22400	11200	1400	2.282	1.794	
2268	22680	11340	1418	—	—	COLLAPSE

CERTIFIED CYPRUS

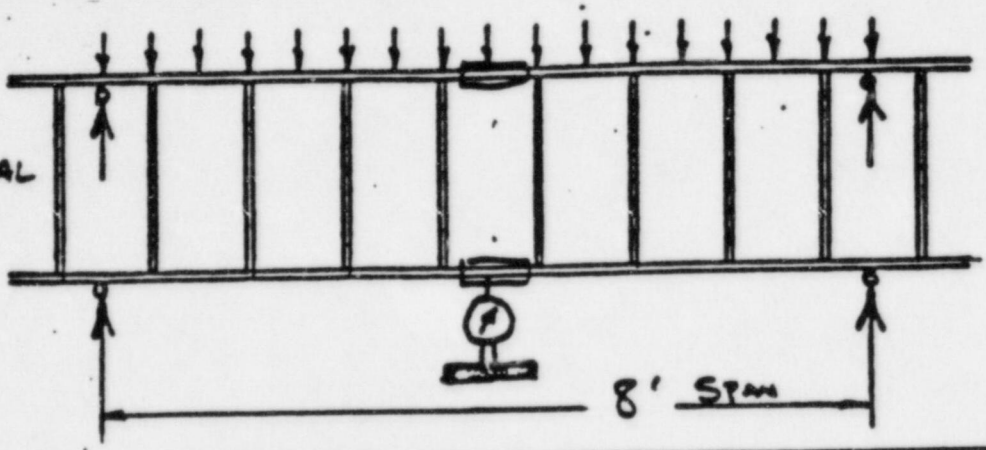
TRAY DESCRIPTION: JM-36SL-12

36" TROF 6¼ x 1¼ .126A 50KSI

SPLICE @ MIDSPAN

01-CP-18
TEST NO.
1-31-79
DATE
BB
BY

HORIZONTAL TEST



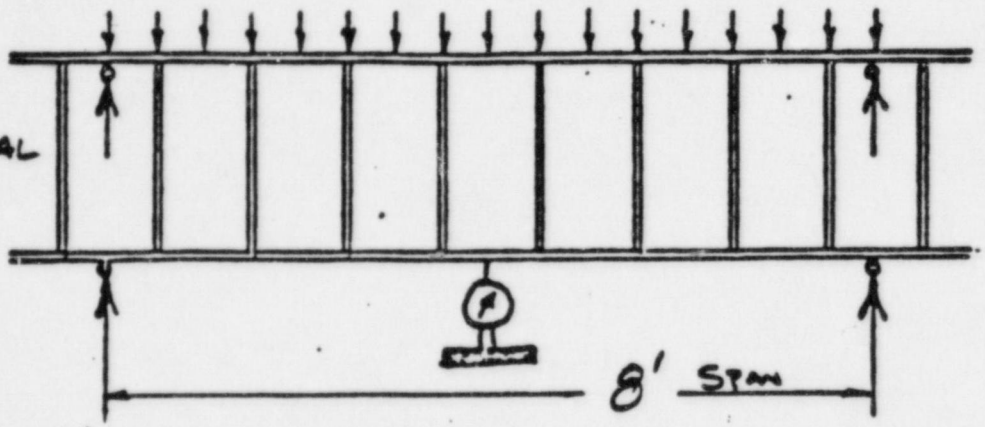
No. of Bundles	Total Weight Lbs.	Total Weight Lbs/2	Lbs/Ft	Indicator Reading	Δ"	Comments
0	0	0	0	3.954	0	
160	1600	800	100	3.757	.097	
320	3200	1600	200	3.653	.201	
480	4800	2400	300	3.514	.340	
640	6400	3200	400	3.392	.462	
800	8000	4000	500	3.271	.583	
960	9600	4800	600	3.165	.689	
1120	11200	5600	700	3.050	.804	
1280	12800	6400	800	2.954	.900	
1440	14400	7200	900	2.828	1.026	
1600	16000	8000	1000	2.703	1.151	
1760	17600	8800	1100	2.559	1.295	
1920	19200	9600	1200	2.363	1.491	
2080	20800	10400	1300	2.138	1.716	
2123	21230	10615	1327	—	—	COLLAPSE

CERTIFIED CYPRUS

TRAY DESCRIPTION: JTM-30SL-12
6"x1¼-12GA 50 KST TROF
SPLICE at MIDSPAN

01-CP-15
TEST NO.
1-27-94
DATE
BB
BY

HORIZONTAL TEST



No. of Bundles	Total Weight Lbs.	Total Weight Lbs/2	Lbs/Ft	Indicator Reading	Δ"	Comments
0	0	0	0	3.743	0	
160	1600	800	100	3.684	.059	
320	3200	1600	200	3.604	.140	
480	4800	2400	300	3.496	.247	
640	6400	3200	400	3.394	.349	
800	8000	4000	500	3.287	.456	
960	9600	4800	600	3.153	.590	
1120	11200	5600	700	3.046	.697	
1280	12800	6400	800	2.962	.781	
1440	14400	7200	900	2.889	.854	
1600	16000	8000	1000	2.771	.972	
1760	17600	8800	1100	2.650	1.093	
1920	19200	9600	1200	2.537	1.206	
2080	20800	10400	1300	2.265	1.478	
2240	22400	11200	1400	2.139	1.604	
2273	22730	11365	1421	—	—	COLLAPSE

CYPRUS

CALCULATIONS FOR CYPRUS CABLE TRAY CAT NO. JM-30SL-12
TO COMANCHE PEAK S.E.S. SPECIFICATION # 2323-ES-19

SPECIFICATION LOADING IS 88 LBS/FT.

6"x1½"x12' T_x=4.39

FROM ATTACHED TEST DATA:

F_m = 1275 + 1289 + 1291 ÷ 3 = 1285

F_x = 1421 + 1414 + 1389 ÷ 3 = 1408

USING EULERS COLUMN FORMULA: $P = \frac{\pi^2 I E}{l^2}$

WHERE: P = ALLOWABLE LOAD LBS

E = 29.5×10^6

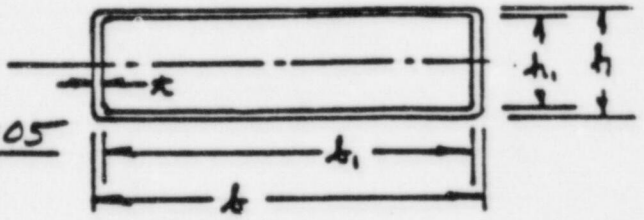
l = 96"

I = ½[b h³ − b_1 h_1^3]

b = 6.25 h = 2.5 t = .105

I = 2.09

3.163, 3.296, 3.450 } 3.303



$F_l = \frac{P}{8} = \frac{\pi^2 \times 2.09 \times 29.5 \times 10^6}{8 \times 96^2}$ = 8247

THE FOLLOWING CRITERIA MUST BE MET:

$$1.6\left[\frac{f'_m}{F_m} + \sqrt{\left(\frac{f_m}{F_m}\right)^2 + \left(\frac{f_x}{F_x}\right)^2 + \left(\frac{f_l}{F_l}\right)^2}\right] \leq 1.0$$

WHERE f'_m = SPECIFICATION LOADING
f_m = 2.5 × SPECIFICATION LOADING
f_x = 4.0 × SPECIFICATION LOADING
f_l = 4.0 × SPECIFICATION LOADING

CYPRUS

$$\frac{88}{1285} + \sqrt{\left(\frac{2.5 \times 88}{1285}\right)^2 + \left(\frac{4.0 \times 88}{1408}\right)^2 + \left(\frac{4.0 \times 88}{8247}\right)^2} \leq .625$$

$$.068 + \sqrt{.029 + .0625 + .0018} \leq .625$$

$$.068 + .305 \leq .625$$

$$.373 \leq .625 \therefore \text{OK}$$

TRAY DESCRIPTION: JM-300L-12 01-CP-1[illegible]

LADDER 30"w 6 1/4 x 1 1/4 12-GA-50KSI

CONN @ MIDSPAN

TEST NO.

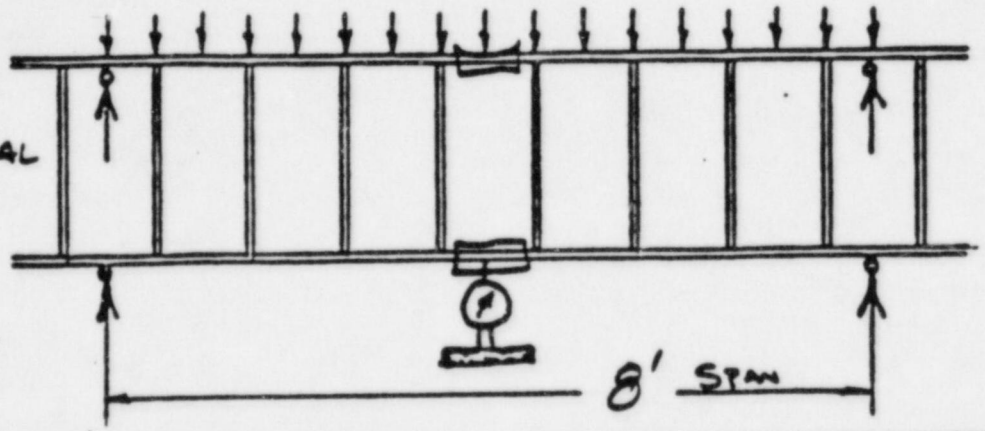
1-25-[illegible]

DATE

BB

BY

HORIZONTAL TEST



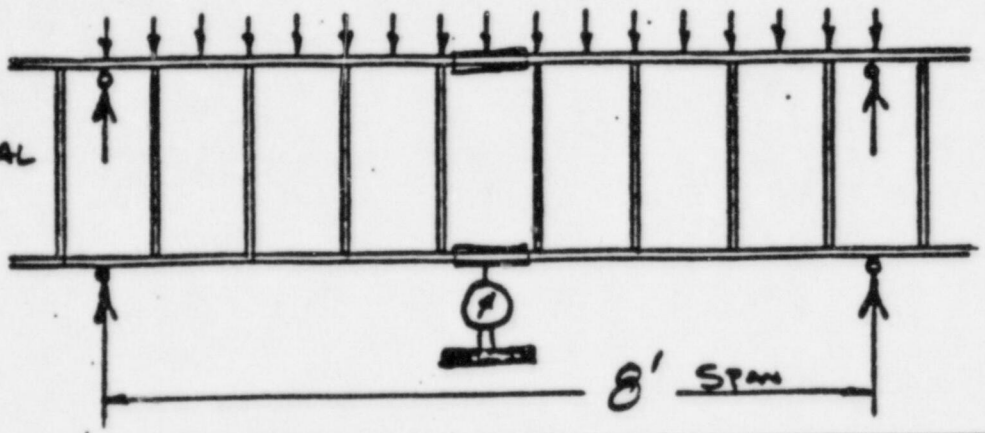
No. of Bundles	Total Weight Lbs.	Total Weight Lbs/2	Lbs/Ft	Indicator Reading	Δ"	Comments
0	0	0	0	3.977	0	
160	1600	800	100	3.905	.072	
320	3200	1600	200	3.808	.169	
480	4800	2400	300	3.701	.276	
640	6400	3200	400	3.584	.393	
800	8000	4000	500	3.480	.497	
960	9600	4800	600	3.346	.631	
1120	11200	5600	700	3.229	.748	
1280	12800	6400	800	3.145	.832	
1440	14400	7200	900	3.033	.944	
1600	16000	8000	1000	2.931	1.046	
1760	17600	8800	1100	2.810	1.167	
1920	19200	9600	1200	2.655	1.312	
2080	20800	10400	1300	2.468	1.509	
2222	22220	11110	1389	—	—	COLLAPSE

CERTIFIED CYPRUS

TRAY DESCRIPTION: JTM-30SL-12
6 1/4 x 1 1/4 12 GA. 50 KSI SOLID TROF
CONN at MIDSPAN

01-CP-
TEST NO.
1-26-78
DATE
BB
BY

HORIZONTAL TEST



No. of Bundles	Total Weight Lbs.	Total Weight Lbs/2	Lbs/Ft	Indicator Reading	Δ"	Comments
0	0	0	0	4.206	0	
160	1600	800	100	4.218	.068	
320	3200	1600	200	4.138	.148	
480	4800	2400	300	4.031	.255	
640	6400	3200	400	3.919	.367	
800	8000	4000	500	3.805	.481	
960	9600	4800	600	3.683	.603	
1120	11200	5600	700	3.567	.719	
1280	12800	6400	800	3.484	.802	
1440	14400	7200	900	3.386	.900	
1600	16000	8000	1000	3.288	.998	
1760	17600	8800	1100	3.169	1.117	
1920	19200	9600	1200	3.022	1.264	
2080	20800	10400	1300	2.846	1.440	
2240	22400	11200	1400	2.615	1.671	
2262	22620	11310	1414	—	—	COLLAPSE

CYPRUS

CALCULATIONS FOR CYPRUS CABLE TRAY CAT NO. TM-24SL-12
TO COMANCHE PEAK S.E.S. SPECIFICATION # 2323-ES-19

SPECIFICATION LOADING IS 70 LBS/FT.

6 1/4 x 1 1/4 x 12 G. $I_x = 4.39$ △

FROM ATTACHED TEST DATA:

$F_m = 1275 + 1289 + 1291 \div 3 = 1285$

$F_x = 1438 + 1497 + 1526 \div 3 = 1487$

USING EULERS COLUMN FORMULA: $P = \frac{\pi^2 I E}{l^2}$

WHERE: P = ALLOWABLE LOAD LBS

$E = 29.5 \times 10^6$

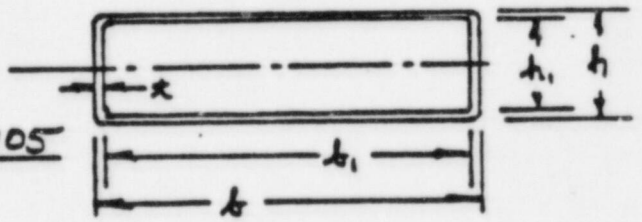
$l = 96''$

3.764, 3.631, 3.420 } 3.605 > △

$I = 1/2 [b h^3 - b_1 h_1^3]$

$b = 6.25$ $h = .25$ $t = .105$

$I = 2.09$



$F_l = \frac{P}{8} = \frac{\pi^2 \times 2.09 \times 29.5 \times 10^6}{8 \times 96^2} = 8247$

THE FOLLOWING CRITERIA MUST BE MET:

$$1.6\left[\frac{f'_m}{F_m} + \sqrt{\left(\frac{f_m}{F_m}\right)^2 + \left(\frac{f_x}{F_x}\right)^2 + \left(\frac{f_l}{F_l}\right)^2}\right] \leq 1.0$$

CYPRUS

WHERE f'_m = SPECIFICATION LOADING
f_m = 2.5 x SPECIFICATION LOADING
f_x = 4.0 x SPECIFICATION LOADING
f_l = 4.0 x SPECIFICATION LOADING

$$\frac{70}{1285} + \sqrt{\left(\frac{2.5 \times 70}{1285}\right)^2 + \left(\frac{4.0 \times 70}{1487}\right)^2 + \left(\frac{4.0 \times 70}{8247}\right)^2} \leq .625$$

$$.054 + \sqrt{.019 + .035 + .0012} \leq .625$$

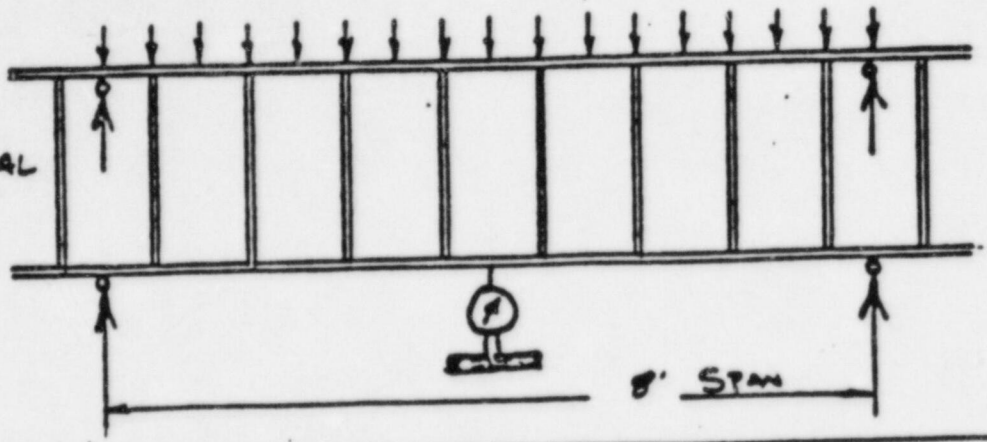
$$.054 + .235 \leq .625$$

$$.289 \leq .625 \quad \therefore \text{OK}$$

TRAY DESCRIPTION. JM-24SL-12
24" W. SOLID TROP 6 1/4 x 1 1/4 12GA 50KSI
CONN at MIDSPAN

01-CP-10
TEST NO.
1-20-78
DATE
BY

HORIZONTAL TEST



No. of Bundles	Total Weight Lbs.	Total Weight Lbs/2	Lbs/Ft	Indicator Reading	Δ"	Comments
0	0	0	0	4.149	0	
160	1600	800	100	4.104	.045	
320	3200	1600	200	4.019	.130	
480	4800	2400	300	3.915	.234	
640	6400	3200	400	3.818	.331	
800	8000	4000	500	3.753	.396	
960	9600	4800	600	3.601	.548	
1120	11200	5600	700	3.510	.639	
1280	12800	6400	800	3.417	.732	
1440	14400	7200	900	3.329	.820	
1600	16000	8000	1000	3.240	.909	
1760	17600	8800	1100	3.145	1.004	
1920	19200	9600	1200	3.041	1.108	
2080	20800	10400	1300	2.889	1.260	
2240	22400	11200	1400	2.714	1.435	
2400	24000	12000	1500	2.468	1.681	
2441	24410	12205	1526	—	—	COLLAPSE

CERTIFIED CYPRUS

TRAY DESCRIPTION: JM-24SL-12

24" WIDE SOLID FLOF 6"x4"x1"x4 12GA 50KSI

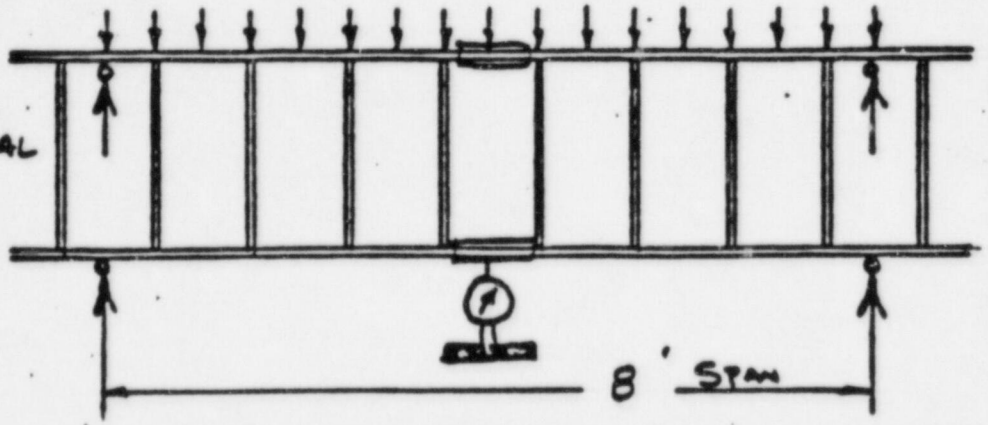
CONN. AT MIDSPAN

01-CP-11
TEST NO.

1-23-78
DATE

BB
BY

HORIZONTAL TEST

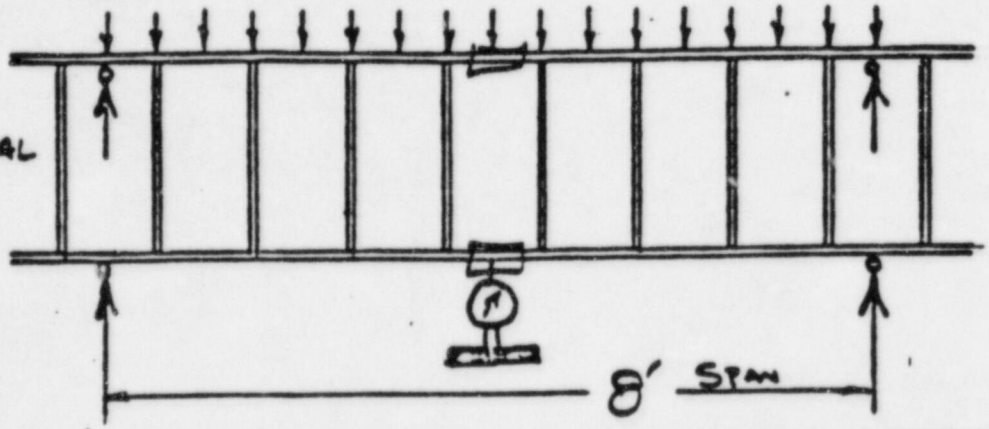


No. OF BUNDLES	TOTAL WEIGHT LBS.	TOTAL WEIGHT LBS/2	LBS/FT	INDICATOR READING	Δ"	COMMENTS
0	0	0	0	4.619	0	
160	1600	800	100	4.571	.048	
320	3200	1600	200	4.483	.136	
480	4800	2400	300	4.376	.243	
640	6400	3200	400	4.269	.350	
800	8000	4000	500	4.168	.451	
960	9600	4800	600	4.043	.576	
1120	11200	5600	700	3.951	.668	
1280	12800	6400	800	3.859	.760	
1440	14400	7200	900	3.772	.847	
1600	16000	8000	1000	3.678	.941	
1760	17600	8800	1100	3.570	1.049	
1920	19200	9600	1200	3.454	1.165	
2080	20800	10400	1300	3.288	1.331	
2240	22400	11200	1400	3.082	1.537	
2395	23950	11975	1497	—	—	COLLAPSE

CERTIFIED CYPRUS

TRAY DESCRIPTION: JM-2452-12 01-CP-12
TEST NO.
24"W TROF 6¼ x 1¼ 12GA 50KSI
COMM @ MIDSPAN
1-24-78
DATE
BB
BY

HORIZONTAL TEST



No. of Bundles	Total Weight Lbs.	Total Weight Lbs/2	Lbs/Ft	Indicator Reading	Δ"	Comments
0	0	0	0	3.741	0	
160	1600	800	100	3.686	.055	
320	3200	1600	200	3.592	.149	
480	4800	2400	300	3.491	.250	
640	6400	3200	400	3.382	.359	
800	8000	4000	500	3.269	.472	
960	9600	4800	600	3.145	.596	
1120	11200	5600	700	3.042	.699	
1280	12800	6400	800	2.948	.793	
1440	14400	7200	900	2.851	.890	
1600	16000	8000	1000	2.753	.988	
1760	17600	8800	1100	2.635	1.106	
1920	19200	9600	1200	2.522	1.219	
2080	20800	10400	1300	2.339	1.402	
2240	22400	11200	1400	2.089	1.652	
2300	23000	11500	1438	—	—	COLLAPSE

CERTIFIED CYPRUS

CALCULATIONS FOR CYPRUS CABLE TRAY CAT NO. JTM-18SL-12
TO COMANCHE PEAK S.E.S. SPECIFICATION # 2323-ES-19

SPECIFICATION LOADING IS 53 LBS/FT.

$6\frac{1}{4} \times 1\frac{1}{4} \times 14$ GA.
$I_x = 3.89$
Δ

FROM ATTACHED TEST DATA:

$F_m = 746 + 791 + 779 \div 3 = 772$

$F_t = 821 + 869 + 839 \div 3 = 843$

USING EULERS COLUMN FORMULA: $P = \frac{\pi^2 I E}{l^2}$

WHERE: P = ALLOWABLE LOAD LBS

$E = 29.5 \times 10^6$

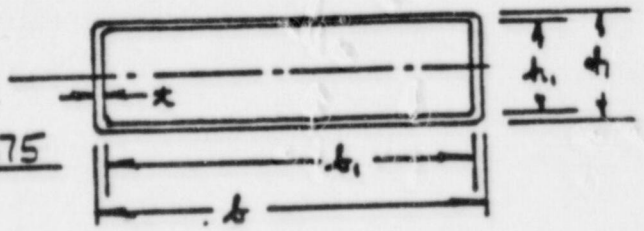
$l = 96''$

$I = \frac{1}{12}[b h^3 - b_1 h_1^3]$

$b = 6.25$ $h = 2.5$ $t = .075$

$I = 1.54$

1.861
1.771
1.865
1.832 >
Δ



$$F_l = \frac{P}{8} = \frac{\pi^2 \times 1.54 \times 29.5 \times 10^6}{8 \times 96^2} = 6082$$

THE FOLLOWING CRITERIA MUST BE MET:

$$1.6\left[\frac{f'_m}{F_m} + \sqrt{\left(\frac{f_m}{F_m}\right)^2 + \left(\frac{f_t}{F_t}\right)^2 + \left(\frac{f_l}{F_l}\right)^2}\right] \leq 1.0$$

CERTIFIED CYPRUS

WHERE f'_m = SPECIFICATION LOADING
f_m = 2.5 × SPECIFICATION LOADING
f_t = 4.0 × SPECIFICATION LOADING
f_l = 4.0 × SPECIFICATION LOADING

$$\frac{53}{772} + \sqrt{\left(\frac{2.5 \times 53}{772}\right)^2 + \left(\frac{4.0 \times 53}{843}\right)^2 + \left(\frac{4.0 \times 53}{6082}\right)^2} \leq .625$$

$$.068 + \sqrt{.029 + .063 + .0012} \leq .625$$

$$.068 + .305 \leq .625$$

$$.373 \leq .625 \therefore OK$$

TRAY DESCRIPTION: TM-18SL-1

18"W SOLID TROF 146A-50 KSI

6"x1¼ SIDE MEMBER

Conn at MIDSPAN

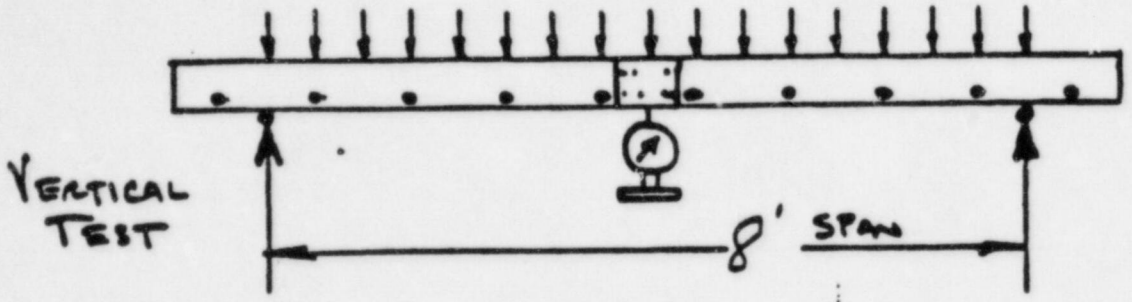
D1-CP-

TEST NO.

1-11-78

DATE

BY



NO. OF BUNDLES	TOTAL WEIGHT LBS.	LBS/FT.	#1 INDICATOR READING	#1 INDICATOR Δ"	#2 INDICATOR READING	#2 INDICATOR Δ"	MAX AVERAGE Δ"	COMMENTS
0	0	0	3.928	0	3.704	0	0	
80	800	100	3.662	.266	3.428	.276	.271	
160	1600	200	3.438	.490	3.190	.514	.502	
240	2400	300	3.251	.677	2.987	.717	.697	
320	3200	400	3.086	.842	2.800	.904	.873	
400	4000	500	2.882	1.046	2.568	1.136	1.091	
480	4800	600	2.643	1.285	2.294	1.410	1.348	
560	5600	700	2.353	1.575	1.961	1.743	1.659	
623	6230	779	—	—	—	—	—	COLLAPSE

CYPRUS

TRAY DESCRIPTION, TM-18SL-12

18"W SOLID TROF (CORR.) 14GA-50KSI

6"1/4 x 1"1/4 SIDERAILS

CONN @ MIDSPAN

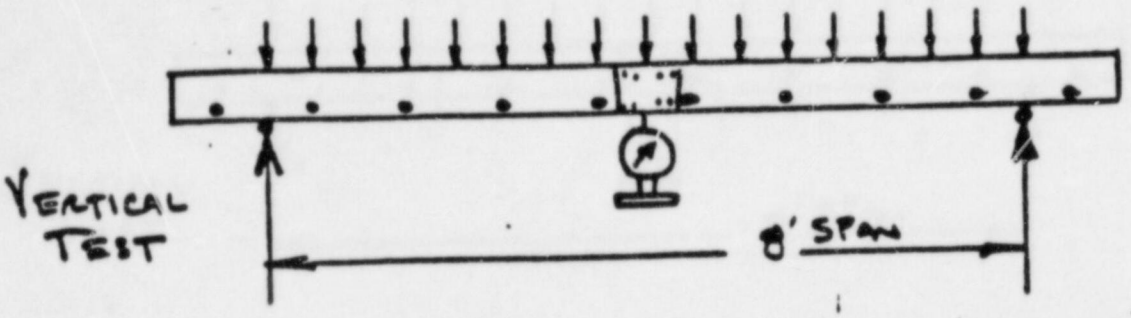
01-CP-01

TEST NO.

1-12-75

DATE

BY



NO. OF BUNDLES	TOTAL WEIGHT LBS.	LBS/FT.	#1 INDICATOR READING	#1 INDICATOR Δ"	#2 INDICATOR READING	#2 INDICATOR Δ"	MAX AVERAGE Δ"	COMMENT
0	0	0	4.178	0	4.352	0	0	
80	800	100	3.927	.251	4.109	.243	.247	
160	1600	200	3.676	.502	3.868	.484	.493	
240	2400	300	3.491	.687	3.699	.653	.670	
320	3200	400	3.316	.862	3.547	.805	.834	
400	4000	500	3.090	1.088	3.349	1.003	1.046	
480	4800	600	2.823	1.355	3105	1.247	1.301	
560	5600	700	2.503	1.675	2.813	1.539	1.607	
633	6330	791	—	—	—	—	—	COLLAPSE

CERTIFIED CYPRUS

TRAY DESCRIPTIO. TM-118SL-12

18"W SOLID TROF 14GA-50KSI

C6 1/4 X 1 1/4 SIDERAILS

SPLICE at MIDSPAN

01-CP-05

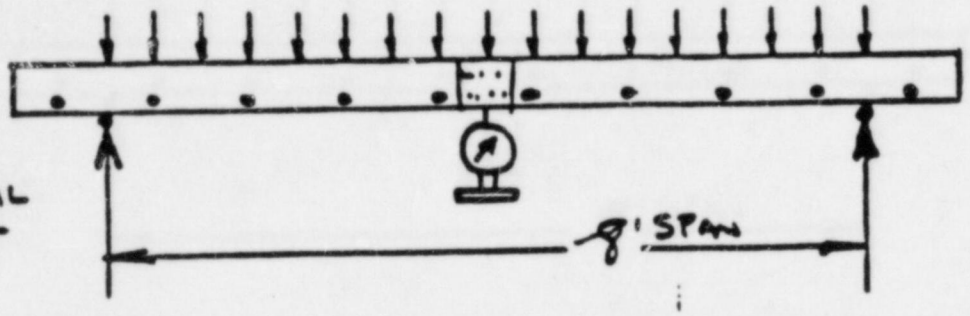
TEST NO.

1-12-78

DATE

BB

BY



NO. OF BUNDLES	TOTAL WEIGHT LBS.	LBS/FT.	#1 INDICATOR READING	#1 INDICATOR Δ"	#2 INDICATOR READING	#2 INDICATOR Δ"	MAX AVERAGE Δ"	COMMENTS
0	0	0	4.545	0	4.723	0	0	
80	800	100	4.158	.387	4.322	.401	.394	
160	1600	200	4.049	.496	4.213	.510	.510	
240	2400	300	3.857	.688	3.988	.735	.712	
320	3200	400	3.685	.860	3.788	.935	.898	
400	4000	500	3.469	1.076	3.537	1.186	1.131	
480	4800	600	3.235	1.310	3.264	1.459	1.385	
560	5600	700	2.924	1.621	2.901	1.822	1.722	
597	5970	746	—	—	—	—	—	COLLAPSE

CYPRUS

TRAY DESCRIPTION: JTM-18SL-12

18"W SOLID CORR TROF 6'/4 x 1'/4 14GA-50 KSI

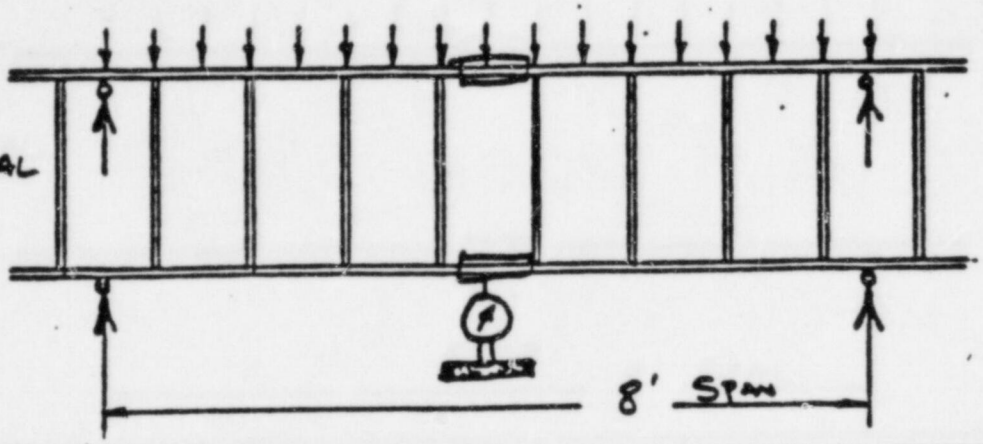
Conn. @ Midspan

01-CP-07
TEST NO.

1-19-7?
DATE

BB
BY

HORIZONTAL TEST



No. of Bundles	Total Weight Lbs.	Total Weight Lbs/2	Lbs/Ft	Indicator Reading	Δ"	Comments
0	0	0	0	4.437	0	
160	1600	800	100	4.285	.152	
320	3200	1600	200	4.131	.306	
480	4800	2400	300	3.948	.489	
640	6400	3200	400	3.801	.636	
800	8000	4000	500	3.596	.841	
960	9600	4800	600	3.381	1.056	
1120	11200	5600	700	3.153	1.284	
1280	12800	6400	800	2.847	1.590	
1390	13900	6950	869	—	—	COLLAPSE

CERTIFIED CYPRUS

TRAY DESCRIPTION: JM-18SL-12

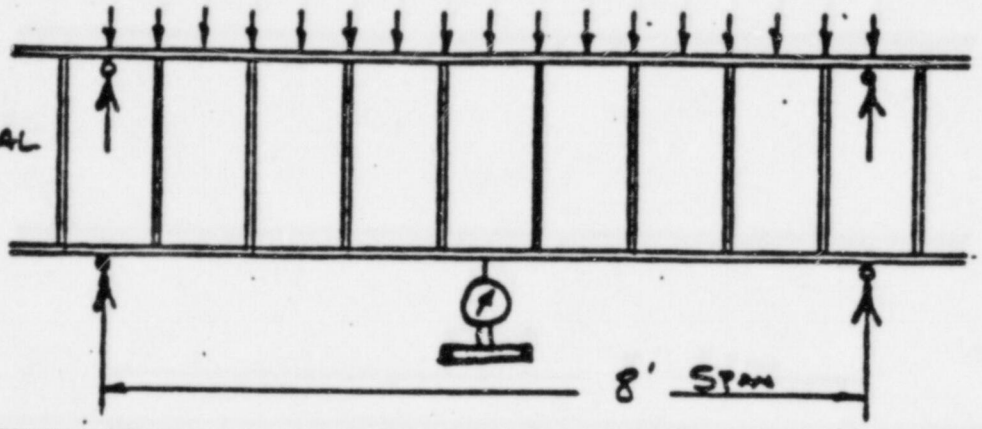
18" W SOLID CORR. TROP

6 1/4 x 1 1/4 14GA - 50KSI

CONN AT MIDSPAN

01-CP-U
TEST
1-19-76
DATE
BB
BY

HORIZONTAL TEST



No. of Bundles	Total Weight Lbs.	Total Weight Lbs/2	Lbs/Ft	Indicator Reading	Δ"	Comments
0	0	0	0	4.198	0	
160	1600	800	100	4.030	.168	
320	3200	1600	200	3.876	.322	
480	4800	2400	300	3.694	.504	
640	6400	3200	400	3.512	.686	
800	8000	4000	500	3.317	.981	
960	9600	4800	600	3.105	1.093	
1120	11200	5600	700	2.874	1.324	
1280	12800	6400	800	2.579	1.619	
1342	13420	6710	839	—	—	COLLAPSE

CERTIFIED CYPRUS

CALCULATIONS FOR CYPRUS CABLE TRAY CAT NO. 66-365L-12-06-C
TO COMANCHE PEAK S.E.S. SPECIFICATION # 2323-ES-19

SPECIFICATION LOADING IS 105 LBS/FT.

4 x 1¼ x 12 GA.
I_x = 3.4
Δ

FROM ATTACHED TEST DATA:

F_m = 981 + 991 + 997.5 ÷ 3 = 990

F_x = 1116 + 1077 + 1083 ÷ 3 = 1092

USING EULERS COLUMN FORMULA: $P = \frac{\pi^2 I E}{l^2}$

WHERE: P = ALLOWABLE LOAD LBS

3.540
3.413
3.858
} 3.604 Δ

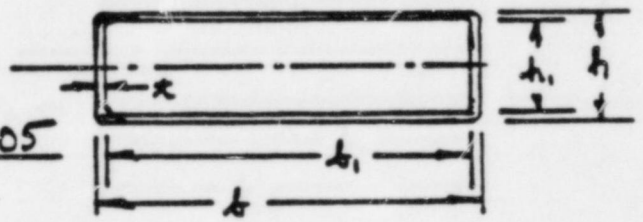
$E = 29.5 \times 10^6$

$l = 96''$

$I = \frac{1}{12}[b h^3 - b_1 h_1^3]$

b = 4.00 h = 2.50 t = .105

I = 1.415



$F_l = \frac{P}{8} = \frac{\pi^2 \times 1.415 \times 29.5 \times 10^6}{8 \times 96^2} =$ 5588

THE FOLLOWING CRITERIA MUST BE MET:

$$1.6\left[\frac{f'_m}{F_m} + \sqrt{\left(\frac{f_m}{F_m}\right)^2 + \left(\frac{f_x}{F_x}\right)^2 + \left(\frac{f_l}{F_l}\right)^2}\right] \leq 1.0$$

CYPRUS

WHERE f'_m = SPECIFICATION LOADING
f_m = 2.5 x SPECIFICATION LOADING
f_x = 4.0 x SPECIFICATION LOADING
f_l = 4.0 x SPECIFICATION LOADING

$$\frac{105}{990} + \sqrt{\left(\frac{2.5 \times 105}{990}\right)^2 + \left(\frac{4.0 \times 105}{1092}\right)^2 + \left(\frac{4.0 \times 105}{5588}\right)^2} \leq .625$$

$$.106 + \sqrt{.070 + .148 + .0056} \leq .625$$

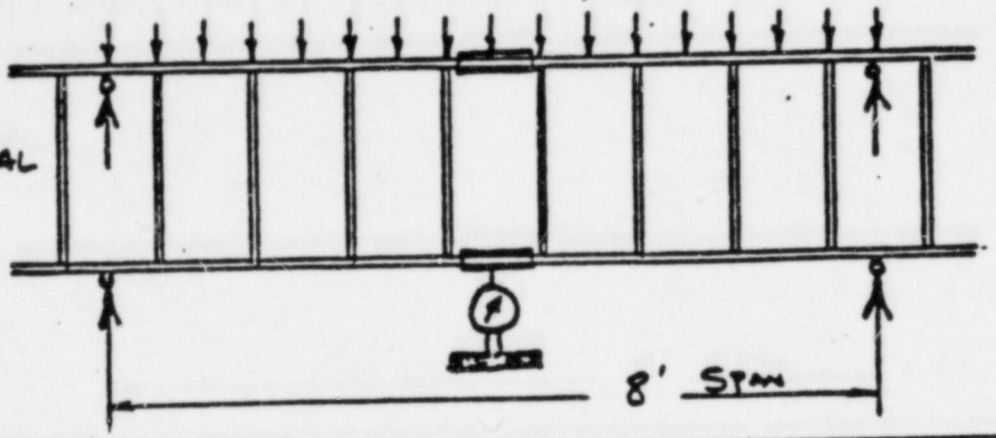
$$.106 + .473 \leq .625$$

$$.579 \leq .625 \quad \therefore \text{OK}$$

TRAY DESCRIPTION: JTM-18SL-12
18" WIDE SOLID CORR TROF
6"x4"x1/4" - 14GA-50KSI
CONN @ MIDSPAN

01-CP-01
TEST NO.
1-20-78
DATE
BB
BY

HORIZONTAL TEST

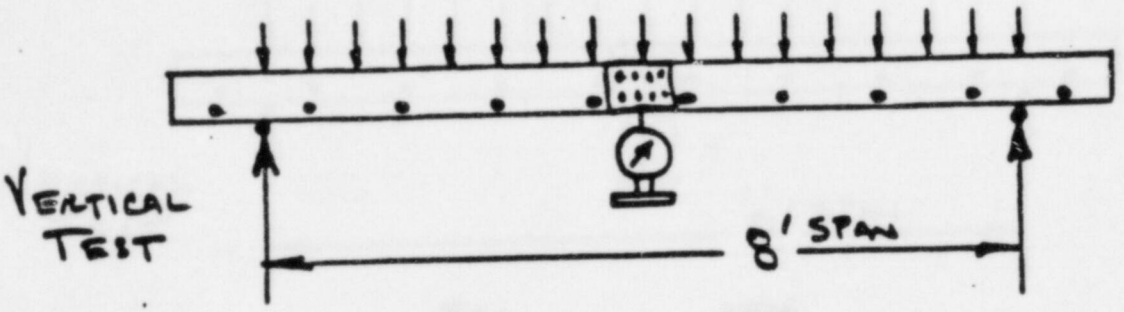


No. of Bundles	Total Weight Lbs.	Total Weight Lbs/2	Lbs/Ft	Indicator Reading	Δ"	Comments
0	0	0	0	4.713	0	
160	1600	800	100	4.617	.096	
320	3200	1600	200	4.352	.361	
480	4800	2400	300	4.175	.538	
640	6400	3200	400	4.002	.711	
800	8000	4000	500	3.797	.916	
960	9600	4800	600	3.593	1.120	
1120	11200	5600	700	3.326	1.387	
1280	12800	6400	800	3.021	1.692	
1313	13130	6565	821	—	—	COLLAPSE

CYPRUS

TRAY DESCRIPTION: GG-3656-12-06
12 GA 50 KSI SIDE MEMBER 16 GA. RUNG
SPLICE AT MID-SPAN

05-CP-0
TEST NO.
5/12/
DATE
B.B.
BY



NO. OF BUNDLES	TOTAL WEIGHT LBS.	LBS/FT.	FAR #1 INDICATOR READING	FAR #1 INDICATOR Δ"	NEAR #2 INDICATOR READING	NEAR #2 INDICATOR Δ"	MAX AVERAGE Δ"	COMMENTS
0	0	0	4.150	0	3.699	0	0	
40	400	50	4.104	.046	3.652	.047	.046	
80	800	100	4.051	.099	3.592	.107	.103	
120	1200	150	3.998	.152	3.536	.163	.158	
160	1600	200	3.939	.211	3.456	.243	.227	
200	2000	250	3.884	.266	3.409	.290	.278	
240	2400	300	3.818	.332	3.314	.385	.359	
280	2800	350	3.744	.406	3.258	.441	.424	
320	3200	400	3.660	.490	3.167	.532	.511	
360	3600	450	3.599	.551	3.084	.615	.583	
400	4000	500	3.507	.643	3.012	.697	.665	
440	4400	550	3.422	.728	2.909	.790	.759	
480	4800	600	3.341	.809	2.833	.866	.838	
520	5200	650	3.237	.913	2.738	.961	.937	
560	5600	700	3.125	1.025	2.606	1.093	1.059	
600	6000	750	3.039	1.111	2.499	1.202	1.157	
640	6400	800	2.938	1.212	2.365	1.334	1.273	
680	6800	850	2.778	1.372	2.216	1.483	1.428	
720	7200	900	2.556	1.594	2.028	1.671	1.633	
760	7600	950	2.334	1.816	1.757	1.942	1.879	
785	7850	981	—	—	—	—	—	COLLAPSE

CYPRUS

TRAY DESCRIPTION GG-36SL42-06

12 GA 50 KSI SIDE MEMBER 16 GA. RUNG

SPLICE AT MID-SPAN

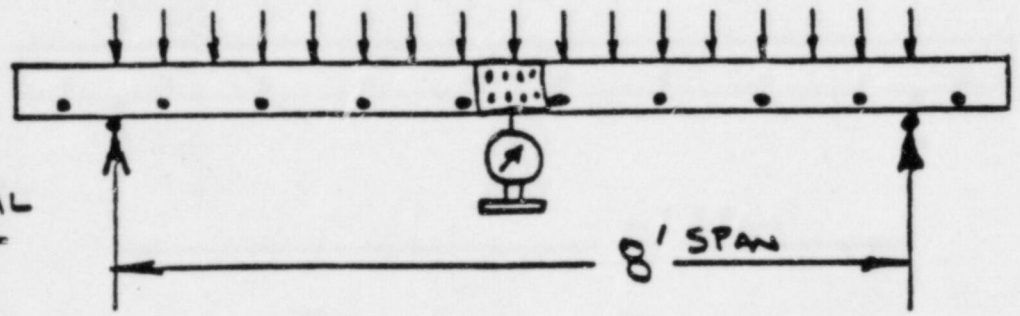
05-CP-081

TEST NO. 5/13/97

DATE

B.B.

BY



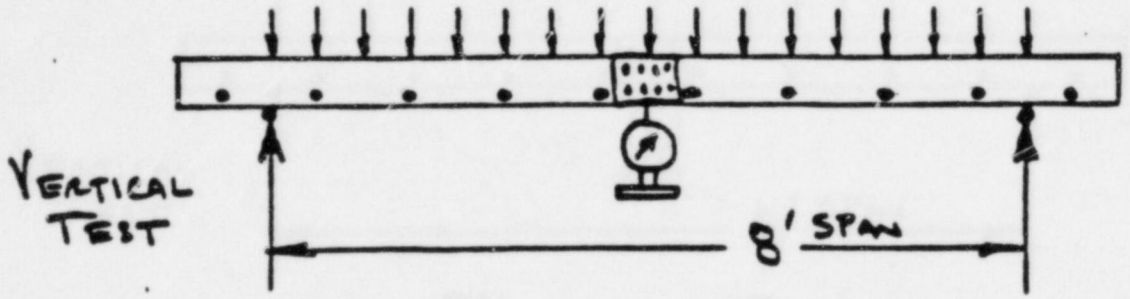
			FAR		NEAR			
NO. OF BUNDLES	TOTAL WEIGHT LBS.	LBS/FT.	#1 INDICATOR READING	#1 INDICATOR Δ"	#2 INDICATOR READING	#2 INDICATOR Δ"	MAX AVERAGE Δ"	COMMENTS
0	0	0	4.150	0	3.726	0	0	
40	400	50	4.109	.041	3.681	.045	.043	
80	800	100	4.060	.090	3.620	.106	.098	
120	1200	150	4.015	.135	3.569	.159	.146	
160	1600	200	3.944	.206	3.494	.232	.219	
200	2000	250	3.887	.263	3.424	.302	.282	
240	2400	300	3.827	.323	3.359	.367	.345	
280	2800	350	3.753	.397	3.293	.433	.415	
320	3200	400	3.691	.459	3.199	.527	.493	
360	3600	450	3.602	.548	3.130	.596	.572	
400	4000	500	3524	.626	3.038	.688	.657	
440	4400	550	3.418	.702	2.959	.767	.734	
480	4800	600	3.382	.768	2.872	.854	.811	
520	5200	650	3.302	.848	2.748	.978	.913	
560	5600	700	3.231	.919	2.653	1.073	.996	
600	6000	750	3.131	1.019	2.538	1.188	1.104	
640	6400	800	3.010	1.140	2.412	1.314	1.227	
680	6800	850	2.908	1.242	2.236	1.490	1.366	
720	7200	900	2.770	1.380	2.048	1.678	1.529	
760	7600	950	2.537	1.613	1.803	1.923	1.768	
793	7930	991	—	—	—	—	—	COLLAPSE

(54)

CYPRUS

TRAY DESCRIPTION: GG-36SL42-06
12 GA 50 KSI SIDE MEMBER 16 GA. RUNG
SPLICE AT MID-SPAN

05-CP-GG
TEST NO.
5/16/97
DATE
B.B.
BY



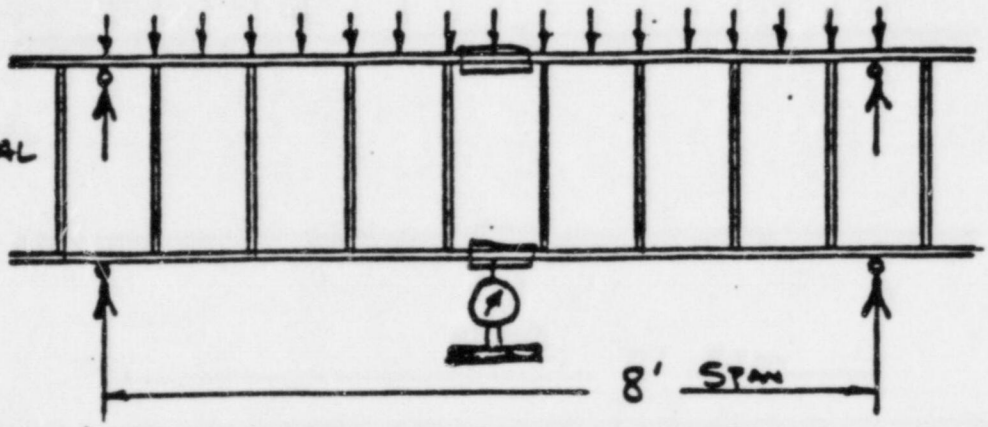
NO. OF BUNDLES	TOTAL WEIGHT LBS.	LBS/FT.	FAR #1 INDICATOR READING	FAR #1 INDICATOR Δ"	NEAR #2 INDICATOR READING	NEAR #2 INDICATOR Δ"	MAX AVERAGE Δ"	COMMENTS
0	0	0	4.062	0	3.300	0	0	
40	400	50	4.020	.042	3.261	.039	.040	
80	800	100	3.963	.099	3.210	.090	.094	
120	1200	150	3.920	.142	3.164	.136	.139	
160	1600	200	3.852	.210	3.105	.195	.202	
200	2000	250	3.771	.291	3.039	.261	.276	
240	2400	300	3.713	.349	2.979	.321	.335	
280	2800	350	3.641	.421	2.915	.385	.403	
320	3200	400	3.565	.497	2.841	.459	.478	
360	3600	450	3.473	.589	2.754	.546	.567	
400	4000	500	3.401	.661	2.693	.606	.634	
440	4400	550	3.314	.748	2.616	.684	.716	
480	4800	600	3.261	.801	2.543	.757	.779	
520	5200	650	3.166	.896	2.468	.832	.864	
560	5600	700	3.063	.999	2.377	.922	.961	
600	6000	750	2.955	1.107	2.291	1.029	1.068	
640	6400	800	2.793	1.269	2.164	1.136	1.203	
680	6800	850	2.649	1.413	2.071	1.229	1.321	
720	7200	900	2.494	1.568	1.924	1.376	1.472	
760	7600	950	2.271	1.791	1.790	1.530	1.660	
798	7980	997.5	—	—	—	—	—	COLLAPSE

CYPRUS

TRAY DESCRIPTION: GG-36SL-12-0G-CP
36" WIDE LADDER- 4x 1¼ 12GA 50KSI

05-CP-30
TEST NO.
11-15-77
DATE
BB
BY

HORIZONTAL TEST



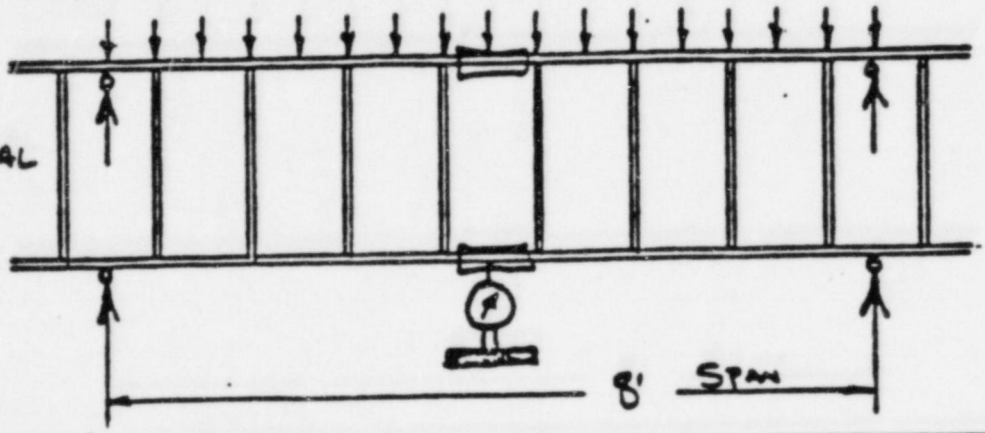
No. OF BUNDLES	TOTAL WEIGHT LBS.	TOTAL WEIGHT LBS/2	LBS/FT	INDICATOR READING	Δ"	COMMENTS
0	0	0	0	4.746	0	
216	2160	1080	135	4.627	.119	
288	2880	1440	180	4.577	.169	
60	3600	1800	225	4.529	.217	
432	4320	2160	270	4.470	.276	
504	5040	2520	315	4.444	.302	
576	5760	2880	360	4.423	.323	
648	6480	3240	405	4.392	.354	
720	7200	3600	450	4.359	.387	
792	7920	3960	495	4.342	.404	
864	8640	4320	540	4.316	.430	
936	9360	4680	585	4.272	.474	
1008	10080	5040	630	4.233	.513	
1080	10800	5400	675	4.189	.557	
1152	11520	5760	720	4.145	.601	
1224	12240	6120	765	4.081	.665	
1296	12960	6480	810	3.988	.758	
1368	13680	6840	855	3.899	.847	
1440	14400	7200	900	3.790	.956	
1512	15120	7560	945	3.730	1.106	
584	15840	7920	990	3.428	1.318	
656	16560	8280	1035	3.164	1.582	
1728	17280	8640	1080	2.835	1.911	
1732	17320	8660	1083	—	—	COLLAPSE

CERTIFIED CYPRUS

TRAY DESCRIPTION: GG-36SC-12
12 GA 50 KSI LADDER 4x1 1/4
CONN @ MIDSPAN
16 GA. RUNG.

05-CP-6
TEST NO.
11-16-71
DATE
BB
BY

HORIZONTAL TEST



No. OF BUNDLES	TOTAL WEIGHT LBS.	TOTAL WEIGHT LBS/2	LBS/FT	INDICATOR READING	Δ"	COMMENTS
0	0	0	0	4.237	0	
216	2160	1080	135	4.115	.122	
288	2880	1440	180	4.063	.174	
360	3600	1800	225	4.008	.229	
432	4320	2160	270	3.946	.291	
504	5040	2520	315	3.928	.309	
576	5760	2880	360	3.902	.335	
648	6480	3240	405	3.868	.369	
720	7200	3600	450	3.835	.402	
792	7920	3960	495	3.819	.418	
864	8640	4320	540	3.787	.450	
936	9360	4680	585	3.731	.506	
1008	10080	5040	630	3.708	.529	
1080	10800	5400	675	3.650	.587	
1152	11520	5760	720	3.609	.628	
1224	12240	6120	765	3.531	.706	
1296	12960	6480	810	3.439	.798	
1368	13680	6840	855	3.340	.897	
1440	14400	7200	900	3.223	1.014	
1512	15120	7560	945	3.056	1.181	
1584	15840	7920	990	2.808	1.429	
1656	16560	8280	1035	2.557	1.680	
1723	17230	8615	1077	—	—	COLLAPSE

CYPRUS

TRAY DESCRIPTION: GG-36SL-12

36"w LADDER 12 GA 50KSI 4x1½

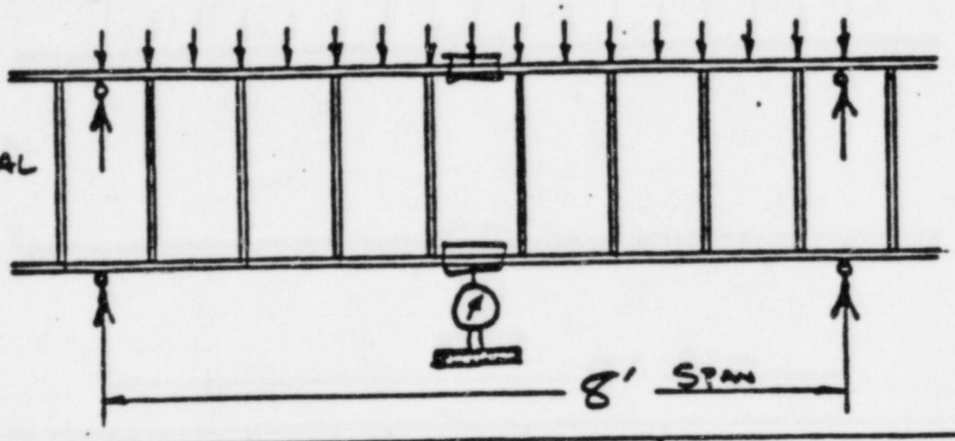
CONN. @ midspan

05-CP-32
TEST NO.

11-18-77
DATE

BS
BY

HORIZONTAL TEST



No. OF BUNDLES	TOTAL WEIGHT LBS.	TOTAL WEIGHT LBS/2	LBS/FT	INDICATOR READING	Δ"	COMMENTS
0	0	0	0	4.156	0.	
216	2160	1080	135	4.050	.106	
288	2880	1440	180	4.007	.149	
360	3600	1800	225	3.955	.201	
432	4320	2160	270	3.908	.248	
504	5040	2520	315	3.874	.282	
576	5760	2880	360	3.852	.304	
648	6480	3240	405	3.833	.323	
720	7200	3600	450	3.799	.357	
792	7920	3960	495	3.776	.380	
864	8640	4320	540	3.748	.408	
936	9360	4680	585	3.715	.441	
1008	10080	5040	630	3.663	.493	
1080	10800	5400	675	3.645	.511	
1152	11520	5760	720	3.594	.562	
1224	12240	6120	765	3.551	.605	
1296	12960	6480	810	3.455	.701	
1368	13680	6840	855	3.362	.794	
1440	14400	7200	900	3.257	.899	
1512	15120	7560	945	3.133	1.023	
1584	15840	7920	990	2.949	1.207	
1656	16560	8280	1035	2.714	1.442	
1728	17280	8640	1080	2.452	1.704	
1785	17850	8925	1116	—	—	COLLAPSE

CYPRUS

CALCULATIONS FOR CYPRUS CABLE TRAY CAT NO. GG-24SL-12-06-C
TO COMANCHE-PEAK S.E.S. SPECIFICATION # 2323-ES-19

SPECIFICATION LOADING IS 70 LBS/FT.

4 x 12 x 14 GA
I_x = 2.166 Δ

FROM ATTACHED TEST DATA:

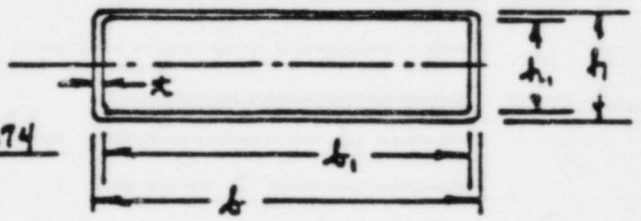
F_m = 685 + 666 + 675 ÷ 3 = 675

F_x = 679 + 620 + 651 ÷ 3 = 650

USING EULERS COLUMN FORMULA: $P = \frac{\pi^2 I E}{l^2}$

WHERE: P = ALLOWABLE LOAD LBS
E = 29.5×10^6
l = 96"
$I = 1/2 [b h^3 - b_1 h_1^3]$
b = 4.00 h = 2.50 t = .074
I = 1.032

2.174
1.991
2.334
} 2.166 > Δ



$F_l = \frac{P}{8'} = \frac{\pi^2 \times 1.032 \times 29.5 \times 10^6}{8 \times 96^2}$ = 4,075

THE FOLLOWING CRITERIA MUST BE MET:

$$1.6 \left[\frac{f'_m}{F_m} + \sqrt{\left(\frac{f_m}{F_m}\right)^2 + \left(\frac{f_x}{F_x}\right)^2 + \left(\frac{f_l}{F_l}\right)^2} \right] \leq 1.0$$

WHERE f'_m = SPECIFICATION LOADING
f_m = 2.5 X SPECIFICATION LOADING
f_x = 4.0 X SPECIFICATION LOADING
f_l = 4.0 X SPECIFICATION LOADING

CYPRUS

$$\frac{70}{675} + \sqrt{\left(\frac{70 \times 2.5}{675}\right)^2 + \left(\frac{70 \times 4.0}{650}\right)^2 + \left(\frac{70 \times 40}{4,075}\right)^2} \leq .625$$

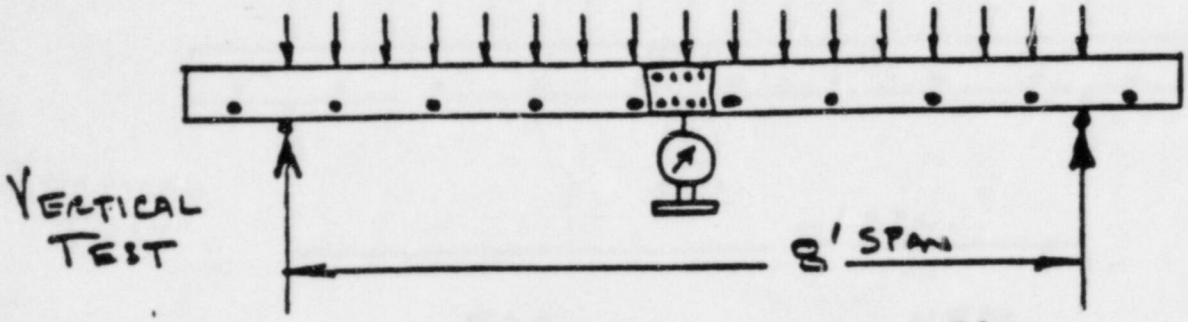
$$.10370 + \sqrt{.06721 + .18556 + .00005} \leq .625$$

$$.10370 + .50281 \leq .625$$

$$.60651 \leq .625 \therefore \text{O.K.}$$

TRAY DESCRIPTION: GG-24SL-12-06-CY
14 GA 50 KS. SIDE MEMBER 16 GA [illegible]
SPLICE AT MID-SPAN

[illegible]
TEST NO.
5/27/77
DATE
B.B.
BY

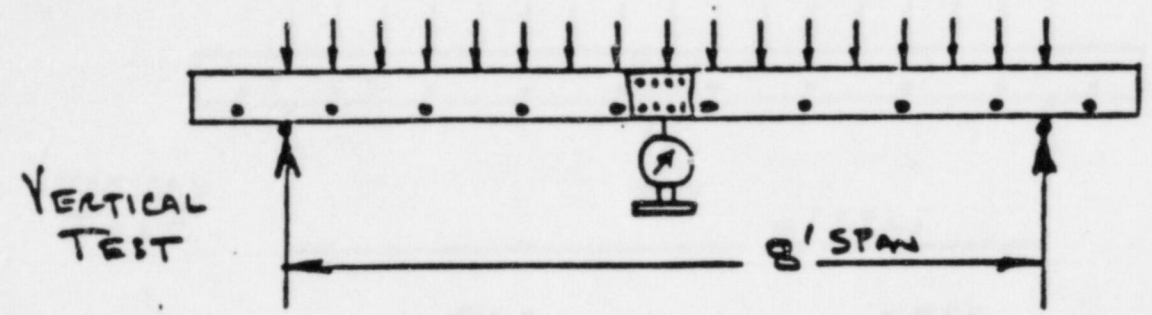


			FAR		NEAR			
NO. OF BUNDLES	TOTAL WEIGHT LBS.	LBS/FT.	#1 INDICATOR READING	#1 INDICATOR Δ"	#2 INDICATOR READING	#2 INDICATOR Δ"	MAX AVERAGE Δ"	COMMENTS
0	0	0	4.250	0	4.100	0	0	
32	320	40	4.159	.091	4.008	.092	.091	
64	640	80	4.085	.165	3.931	.169	.167	
96	960	120	4.029	.221	3.869	.231	.226	
128	1280	160	3.943	.307	3.780	.320	.313	
160	1600	200	3.879	.371	3.707	.393	.382	
192	1920	240	3.808	.442	3.629	.471	.457	
224	2240	280	3.738	.512	3.554	.546	.529	
256	2560	320	3.681	.569	3.489	.611	.590	
288	2880	360	3.600	.649	3.411	.689	.669	
320	3200	400	3.503	.747	3.303	.797	.772	
352	3520	440	3.413	.837	3.202	.898	.867	
384	3840	480	3.334	.916	3.086	1.014	.965	
416	4160	520	3.213	1.037	2.938	1.162	1.099	
448	4480	560	3.107	1.143	2.797	1.303	1.223	
480	4800	600	2.969	1.281	2.638	1.462	1.371	
512	5120	640	2.801	1.449	2.440	1.660	1.554	
540	5400	675	—	—	—	—	—	COLLAPSE

CYPRUS

TRAY DESCRIPTION: GG-245L-12-06-CP
14 GA 50 KSI SIDE MEMBER 16 GA RUNG
SPLICE AT MID-SPAN

05-CP-04
TEST NO.
5/11/77
DATE
B.B.
BY



			FAR		NEAR			
NO. OF BUNDLES	TOTAL WEIGHT LBS.	LBS/FT.	#1 INDICATOR READING	#1 INDICATOR Δ"	#2 INDICATOR READING	#2 INDICATOR Δ"	MAX AVERAGE Δ"	COMMENTS
0	0	0	4.217	0	3.760	0	0	
32	320	40	4.118	.099	3.665	.095	.097	
64	640	80	4.039	.178	3.590	.170	.174	
96	960	120	3.977	.240	3.531	.229	.234	
128	1280	160	3.877	.340	3.438	.322	.331	
160	1600	200	3.800	.417	3369	.391	.404	
192	1920	240	3.719	.498	3.298	.462	.480	
224	2240	280	3.632	.585	3.221	.539	.562	
256	2560	320	3.561	.650	3.172	.588	.619	
288	2880	360	3.496	.721	3.121	.639	.680	
320	3200	400	3.376	.851	3.018	.742	.796	
352	3520	440	3.283	.934	2.930	.830	.882	
384	3840	480	3.167	1.050	2.824	.936	.993	
416	4160	520	3.031	1.186	2.703	1.059	1.121	
448	4480	560	2.879	1.338	2.562	1.198	1.268	
480	4800	600	2.695	1.522	2.424	1.336	1.429	
512	5120	640	2.460	1.757	2.193	1.567	1.662	
533	5330	666	—	—	—	—	—	COLLAPSE

(69)

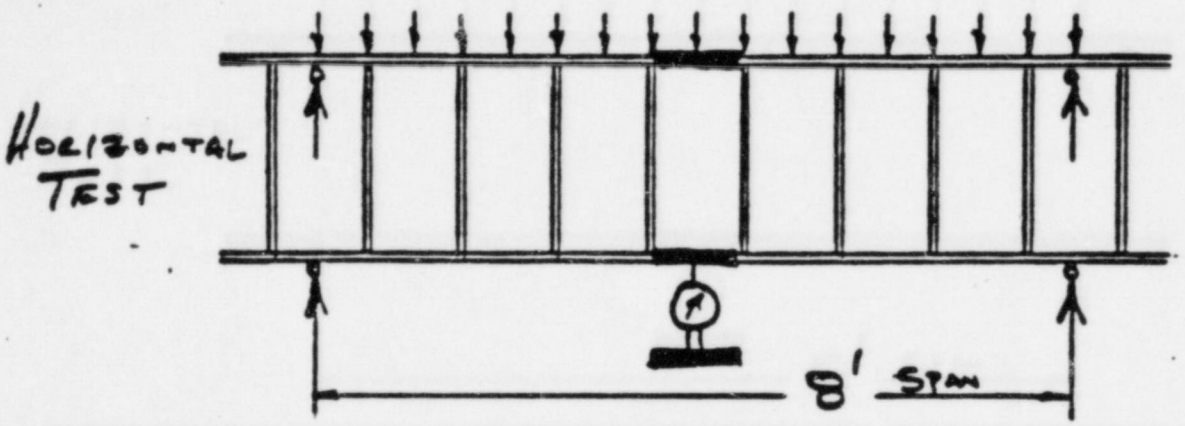
CYPRUS

TRAY DESCRIPTION: GG-24SL-12-06

19 GA. 50KSI SIDE RAIL 16 GA. RUNG

SPLICE AT MIDSPAN

05-CP-19
TEST NO.
5/27/77
DATE
B.B.
BY



No. OF BUNDLES	TOTAL WEIGHT LBS.	TOTAL WEIGHT LBS/2	LBS/FT	INDIMATH READING	Δ"	COMMENTS
0	0	0	0	3.150	0	
60	600	300	37.5	3.069	.081	
120	1200	600	75	3.017	.133	
180	1800	900	112.5	2.958	.192	
240	2400	1200	150	2.910	.240	
300	3000	1500	187.5	2.852	.298	
360	3600	1800	225	2.816	.334	
420	4200	2100	262.5	2.780	.370	
480	4800	2400	300	2.748	.402	
540	5400	2700	337.5	2.721	.429	
600	6000	3000	375	2.674	.476	
660	6600	3300	412.5	2.615	.535	
720	7200	3600	450	2.553	.597	
780	7800	3900	487.5	2.477	.673	
840	8400	4200	525	2.350	.800	
900	9000	4500	562.5	2.204	.946	
960	9600	4800	600	1.994	1.156	
1020	10,200	5100	637.5	1.707	1.443	
1042	10420	5210	651	—	—	COLLAPSE

CYPRUS

TRAY DESCRIPTION: GG-24SL-12-00

14 GA. 50KSI SIDE RAIL 16 GA. RUNG

SPLICE AT MIDSPAN.

05-CP-11

TEST NO.

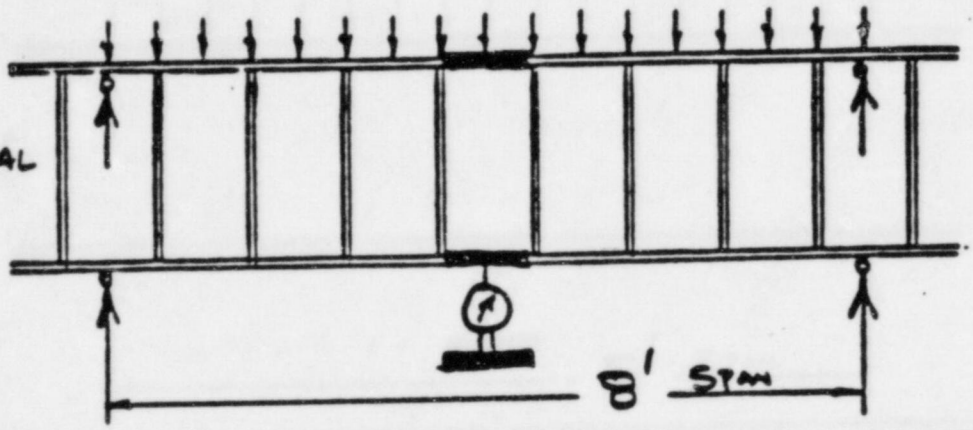
5/17/71

DATE

B. B.

BY

HORIZONTAL TEST



No. OF BUNDLES	TOTAL WEIGHT LBS.	TOTAL WEIGHT LBS/2	LBS/FT	INDICATOR READING	Δ"	COMMENTS
0	0	0	0	4.200	0	
60	600	300	37.5	4.120	.080	
120	1200	600	75	4.072	.128	
180	1800	900	112.5	4.024	.176	
240	2400	1200	150	3.981	.219	
300	3000	1500	187.5	3.930	.270	
360	3600	1800	225	3.887	.313	
420	4200	2100	262.5	3.851	.349	
480	4800	2400	300	3.826	.374	
540	5400	2700	337.5	3.783	.417	
600	6000	3000	376	3.749	.451	
660	6600	3300	412.5	3.698	.502	
720	7200	3600	450	3.645	.555	
780	7800	3900	487.5	3.580	.620	
840	8400	4200	525	3.464	.736	
900	9000	4500	562.5	3.332	.868	
960	9600	4800	600	3.133	1.067	
1020	10200	5100	637.5	2.873	1.327	
1080	10800	5400	675	2.434	1.766	
1087	10870	5435	679	—	—	COLLAPSE

CYPRUS

CALCULATIONS FOR CYPRUS CABLE TRAY CAT NO. GG-30SL-12-06-C
TO COMANCHE PEAK S.E.S. SPECIFICATION # 2323-ES-19

SPECIFICATION LOADING IS 88 LBS/FT.

4 x 1 1/4 x 12...
$I_x = 3.4$...

FROM ATTACHED TEST DATA:

$$F_m = 981 + 991 + 997.5 \div 3 = 990$$

$$F_t = 1111 + 1124 + 1101 \div 3 = 1112$$

USING EULERS COLUMN FORMULA: $P = \dfrac{\pi^2 I E}{l^2}$

WHERE: P = ALLOWABLE LOAD LBS

$E = 29.5 \times 10^6$

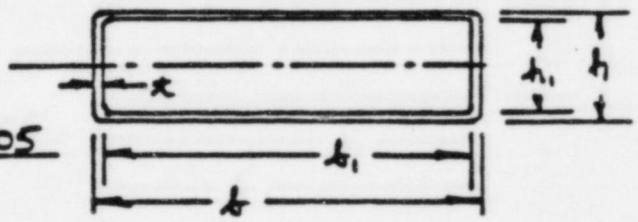
$l = 96''$

$I = \frac{1}{12}\left[b h^3 - b_1 h_1^3\right]$

$b = 4.00$ $h = 2.50$ $t = .105$

$I = 1.415$

3.740
4.073 } 3.770 >
3.496 Δ



$$F_l = \frac{P}{8'} = \frac{\pi^2 \times 1.415 \times 29.5 \times 10^6}{8 \times 96^2} = 5588$$

THE FOLLOWING CRITERIA MUST BE MET:

$$1.6\left[\frac{f'_m}{F_m} + \sqrt{\left(\frac{f_m}{F_m}\right)^2 + \left(\frac{f_t}{F_t}\right)^2 + \left(\frac{f_l}{F_l}\right)^2}\right] \leq 1.0$$

CERTIFIED CYPRUS

WHERE f'_m = SPECIFICATION LOADING

f_m = 2.5 × SPECIFICATION LOADING

f_t = 4.0 × SPECIFICATION LOADING

f_l = 4.0 × SPECIFICATION LOADING

$$\frac{88}{990} + \sqrt{\left(\frac{2.5 \times 88}{990}\right)^2 + \left(\frac{4.0 \times 88}{1112}\right)^2 + \left(\frac{4.0 \times 88}{5{,}588}\right)^2} \leq .625$$

$$.08889 + \sqrt{.04938 + .10020 + .00397} \leq .625$$

$$.08889 + .39185 \leq .625$$

$$.48074 \leq .625 \quad \therefore \text{O.K.}$$

TRAY DESCRIPTION: GG-3054-12-06-CP

12GA 50ICSI SIDE RAIL 16 GA. RUNG

SPLICE AT MIDSPAN

05-CY-04

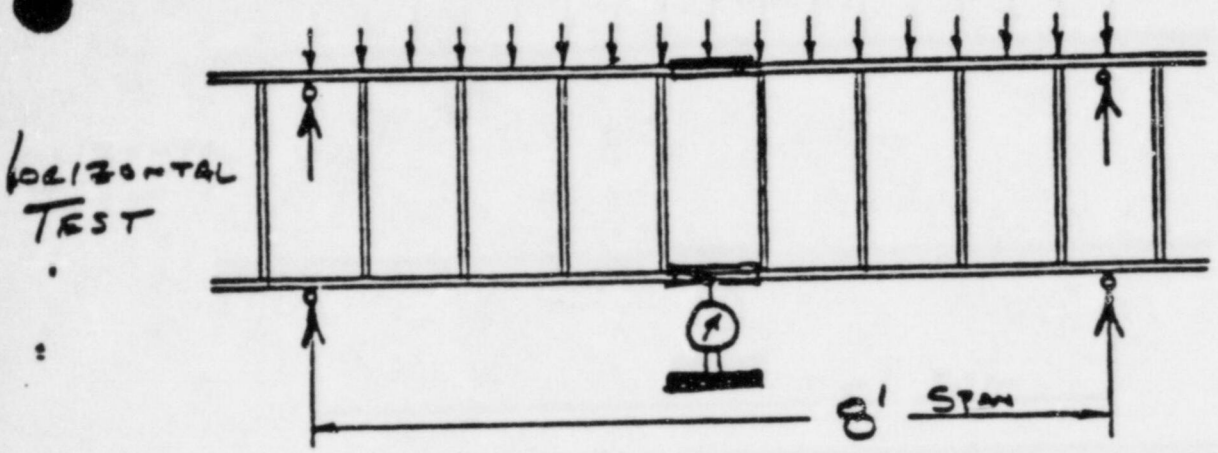
TEST NO.

5/16/77

DATE

B.B.

BY



No. of Bundles	Total Weight Lbs.	Total Weight Lbs/2	Lbs/Ft	Indicator Reading	Δ"	Comments
0	0	0	0	4.050	0	
216	2160	1080	135	3.929	.121	
288	2880	1440	180	3.893	.157	
360	3600	1900	225	3.871	.179	
432	4320	2160	270	3.816	.234	
504	5040	2520	315	3.761	.289	
576	5760	2880	360	3.729	.321	
648	6480	3240	405	3.688	.362	
720	7200	3600	450	3.669	.381	
792	7920	3960	495	3.648	.402	
864	8640	4320	540	3.622	.428	
936	9360	4680	585	3.597	.453	
1008	10080	5040	630	3.573	.477	
1080	10800	5400	675	3.518	.532	
1152	11520	5760	720	3.484	.566	
1224	12240	6,120	765	3.432	.618	
1296	12960	6480	810	3.356	.694	
1368	13680	6840	855	3.273	.777	
1440	14,400	7,200	900	3.188	.862	
1512	15120	7560	945	3.063	.987	
1584	15840	7920	990	2.992	1.158	
1656	16560	8280	1035	2.695	1.355	
1728	17280	8640	1080	2.413	1.637	
1762	17620	8810	1101	—	—	COLLAPSE

CYPRUS

62

TRAY DESCRIPTION: GG-30SL-12-06-CP

12 GA 50 KSI SIDE RAIL 16 GA. RUNG

SPLICE AT MIDSPAN

05-CP-06

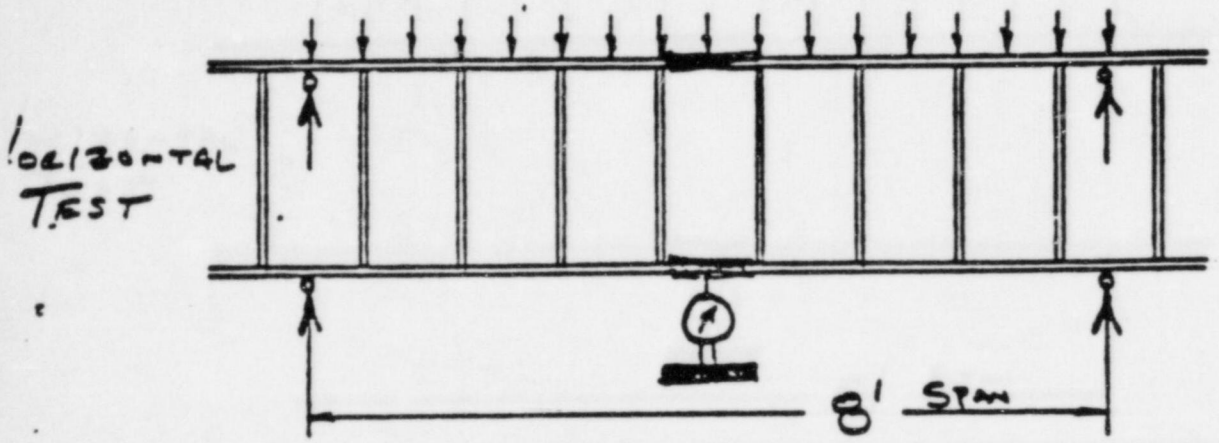
TEST NO.

5/12/11

DATE

B.B.

BY



No. of Bundles	Total Weight Lbs.	Total Weight Lbs/2	Lbs/Ft	Indicator Reading	Δ"	Comments
0	0	0	0	3.700	0	
216	2160	1080	135	3.585	.115	
288	2880	1440	180	3.549	.151	
360	3600	1900	225	3.517	.183	
432	4320	2160	270	3.471	.229	
504	5040	2520	315	3.430	.270	
576	5760	2880	360	3.397	.303	
648	6480	3240	405	3.372	.328	
720	7200	3600	450	3.358	.342	
792	7920	3960	495	3.329	.371	
864	8640	4320	540	3.306	.394	
936	9360	4680	585	3.283	.417	
1008	10080	5040	630	3.256	.444	
1080	10800	5400	675	3.217	.483	
1152	11520	5760	720	3.174	.526	
1224	12240	6,120	765	3.121	.579	
1296	12960	6480	810	3.069	.633	
1368	13680	6840	855	2.981	.719	
1440	14,400	7,200	900	2.882	.818	
1512	15120	7560	945	2.769	.931	
1584	15840	7920	990	2.619	1.081	
1656	16560	8280	1035	2.426	1.274	
1728	17280	8640	1080	2.224	1.476	
1778	17780	8890	1111	—	—	COLLAPSE

CYPRUS

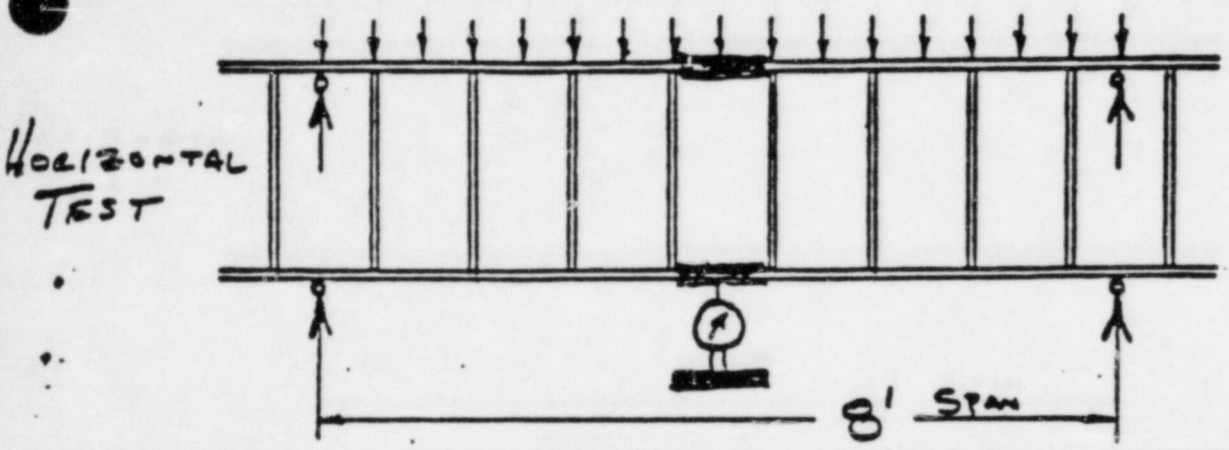
63

TRAY DESCRIPTION # GG-305L-12-06-CP

12GA 50KSI S.E RAIL 16 GA. RU. 6

SPLICE AT MIDSPAN

05-CP-01
TEST NO.
5/13/77
DATE
B.B.
BY



No. of Bundles	Total Weight Lbs.	Total Weight Lbs/2	Lbs/Ft	Indicator Reading	Δ"	Comments
0	0	0	0	2.845	0	
216	2160	1080	135	2.746	.099	
288	2880	1440	180	2.704	.141	
360	3600	1900	225	2.669	.176	
432	4320	2160	270	2.648	.197	
504	5040	2520	315	2.622	.223	
576	5760	2880	360	2.596	.249	
648	6480	3240	405	2.543	.302	
720	7200	3600	450	2.517	.328	
792	7920	3960	495	2.499	.347	
864	8640	4320	540	2.476	.369	
936	9360	4680	585	2.445	.400	
1008	10080	5040	630	2.418	.427	
1080	10800	5400	675	2.387	.458	
1152	11520	5760	720	2.352	.493	
1224	12240	6,120	765	2.301	.544	
1296	12960	6480	810	2.249	.596	
1368	13680	6840	855	2.188	.657	
1440	14,400	7,200	900	2.082	.763	
1512	15120	7560	945	1.974	.871	
1584	15840	7920	990	1.849	.996	
1656	16560	8280	1035	1.708	1.137	
1728	17280	8640	1080	1.536	1.309	
1799	17990	8995	1124	—	—	COLLAPSE

CYPRUS

64

CALCULATIONS FOR CYPRUS CABLE TRAY CAT NO. GG-18SL-12-06-C
TO COMANCHE PEAK S.E.S. SPECIFICATION # 2323-ES-19

4x1½x14G
$I_x = 2.46$ △

SPECIFICATION LOADING IS 53 LBS/FT.

FROM ATTACHED TEST DATA:

$F_m = 695 + 666 + 675 \div 3 = 675$

$F_x = 660 + 670.6 + 677 \div 3 = 669$

USING EULERS COLUMN FORMULA: $P = \frac{\pi^2 I E}{l^2}$

WHERE: P = ALLOWABLE LOAD LBS

$E = 29.5 \times 10^6$

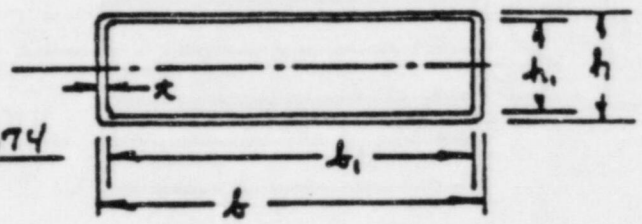
$l = 96''$

$I = \frac{1}{12}[bh^3 - b_1 h_1^3]$

2.842
2.357
2.500
} 2.360 △

$b = 4.00$ $h = 2.50$ $t = .074$

$I = 1.032$



$F_l = \frac{P}{8} = \frac{\pi^2 \times 1.032 \times 29.5 \times 10^6}{8 \times 96^2} = 4,075$

THE FOLLOWING CRITERIA MUST BE MET:

$$1.6\left[\frac{f'_m}{F_m} + \sqrt{\left(\frac{f_m}{F_m}\right)^2 + \left(\frac{f_x}{F_x}\right)^2 + \left(\frac{f_l}{F_l}\right)^2}\right] \leq 1.0$$

CYPRUS

WHERE f'_m = SPECIFICATION LOADING
f_m = 2.5 x SPECIFICATION LOADING
f_x = 4.0 x SPECIFICATION LOADING
f_l = 4.0 x SPECIFICATION LOADING

$$\frac{53}{675} + \sqrt{\left(\frac{2.5 \times 53}{675}\right)^2 + \left(\frac{4.0 \times 53}{669}\right)^2 + \left(\frac{4.0 \times 53}{4,075}\right)^2} \leq .625$$

$$.07852 + \sqrt{.03853 + .10042 + .00271} \leq .625$$

$$.07852 + .37638 \leq .625$$

$$.45490 \leq .625 \therefore \text{O.K.}$$

TRAY DESCRIPTION GG-18SL-12-0'-CP

19GA. 50KSI SIDE RAIL 16 GA. RUNG

SPLICE AT MIDSPAN

05-CP-14

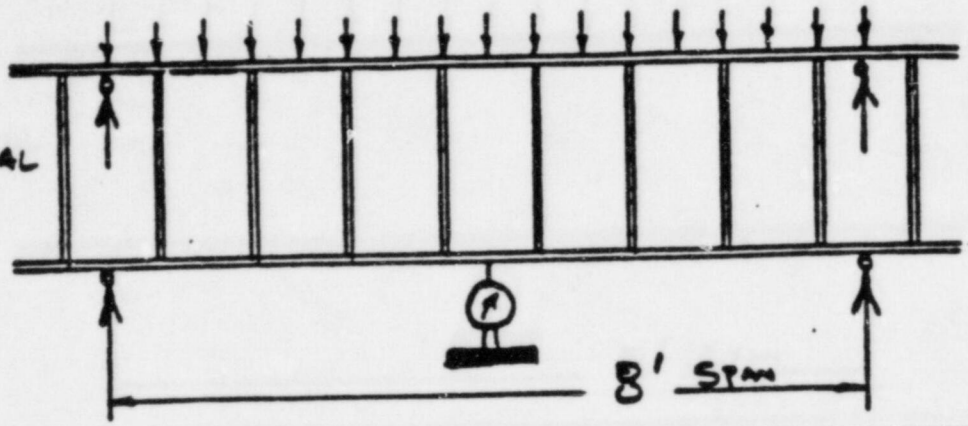
TEST NO.

5/20/77

DATE

B.B.

BY



No. of Bundles	Total Weight Lbs.	Total Weight Lbs/2	Lbs/Ft	Indicator Reading	Δ"	Comments
0	0	0	0	3.900	0	
60	600	300	37.5	3.827	.093	
120	1200	600	75	3.784	.116	
180	1800	900	112.5	3.727	.193	
240	2400	1200	150	3.679	.221	
300	3000	1500	187.5	3.636	.264	
360	3600	1800	225	3.583	.317	
420	4200	2100	262.5	3.549	.351	
480	4800	2400	300	3.510	.390	
540	5400	2700	337.5	3.473	.427	
600	6000	3000	375	3.419	.481	
660	6600	3300	412.5	3.374	.526	
720	7200	3600	450	3.305	.595	
780	7800	3900	487.5	3.239	.661	
840	8400	4200	525	3.106	.794	
900	9000	4500	562.5	2.942	.958	
960	9600	4800	600	2.748	1.152	
1020	10,200	5100	637.5	2.494	1.406	
1056	10,560	5280	660	—	—	COLLAPSE

CERTIFIED CYPRUS

TRAY DESCRIPTION # GG-18SL-12-06-CP

14GA. 50KSI SIDE RAIL 16 GA. RUNG

SPLICE AT MIDSPAN

05-CP-15

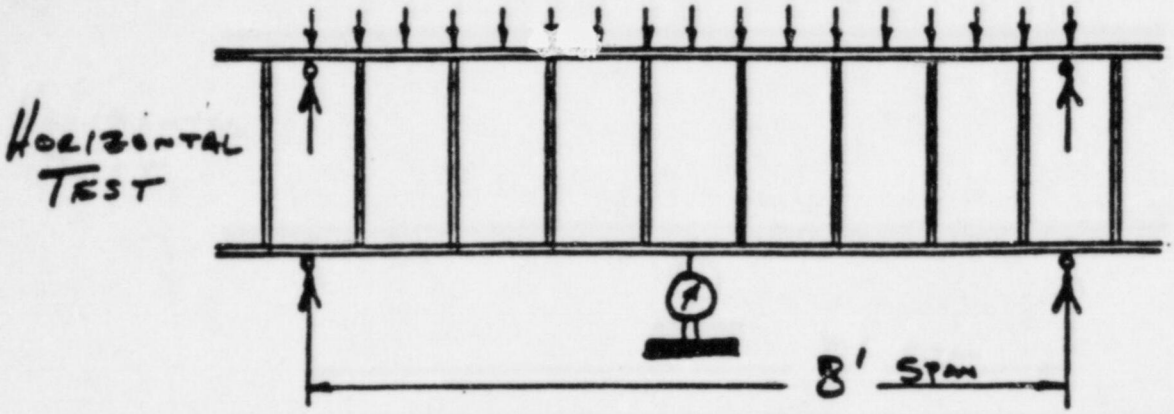
TEST #

5/23/

DATE

B.B.

BY



No. of Bundles	Total Weight Lbs.	Total Weight Lbs/2	Lbs/Ft	Indicator Reading	Δ"	Comments
0	0	0	0	3.300	0	
60	600	300	37.5	3.229	.071	
120	1200	600	75	3.187	.113	
180	1800	900	112.5	3.131	.169	
240	2400	1200	150	3.086	.214	
300	3000	1500	187.5	3.049	.251	
360	3600	1800	225	3.001	.299	
420	4200	2100	262.5	2.977	.323	
480	4800	2400	300	2.925	.375	
540	5400	2700	337.5	2.896	.404	
600	6000	3000	375	2.850	.450	
660	6600	3300	412.5	2.792	.508	
720	7200	3600	450	2.741	.559	
780	7800	3900	487.5	2.673	.627	
840	8400	4200	525	2.562	.738	
900	9000	4500	562.5	2.431	.869	
960	9600	4800	600	2.210	1.090	
1020	10200	5100	637.5	1.979	1.321	
1073	10730	5365	670.6	—	—	COLLAPSE

CYPRUS

TRAY DESCRIPTION: GG-18SL-12-0-CP

14GA. 50KSI SIDE RAIL 16 GA. RUNG

SPLICE AT MIDSPAN

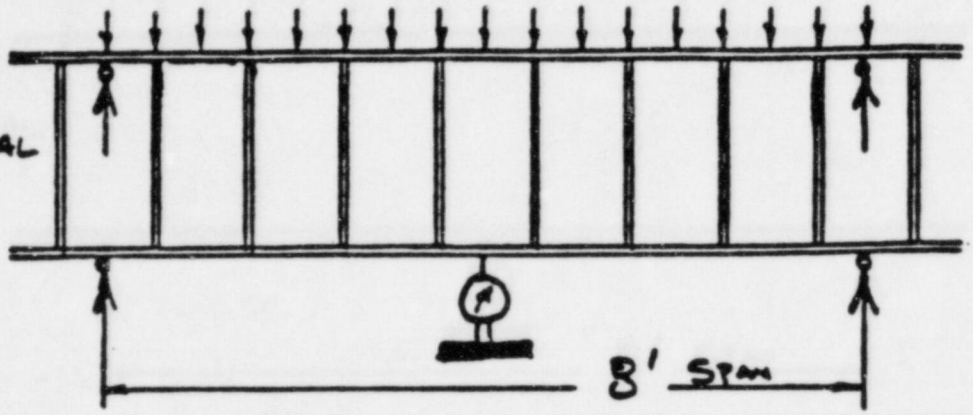
05-CP-16

TEST # 5/24

DATE

BB.

BY



No. OF BUNDLES	TOTAL WEIGHT LBS.	TOTAL WEIGHT LBS/2	LBS/FT	INDICATOR READING	Δ"	COMMENTS
0	0	0	0	2.840	0	
60	600	300	37.5	2.771	.069	
120	1200	600	75	2.728	.112	
180	1800	900	112.5	2.682	.158	
240	2400	1200	150	2.637	.203	
300	3000	1500	187.5	2.603	.237	
360	3600	1800	225	2.556	.284	
420	4200	2100	262.5	2.514	.326	
480	4800	2400	300	2.497	.343	
540	5400	2700	337.5	2.454	.386	
600	6000	3000	375	2.421	.419	
660	6600	3300	412.5	2.373	.467	
720	7200	3600	450	2.320	.520	
780	7800	3900	487.5	2.255	.585	
840	8400	4200	525	2.163	.677	
900	9000	4500	562.5	2.030	.810	
960	9600	4800	600	1.842	.998	
1020	10200	5100	637.5	1.609	1.231	
1080	10800	5400	675.0	1.338	1.502	
1083	10830	5415	677	—	—	COLLAPSE

CERTIFIED CYPRUS

CALCULATIONS FOR CYPRUS CABLE TRAY CAT NO. GG-12SL-12-06-C1
TO COMANCHE PEAK S.E.S. SPECIFICATION # 2323-ES-19

SPECIFICATION LOADING IS 35 LBS/FT.

FROM ATTACHED TEST DATA:

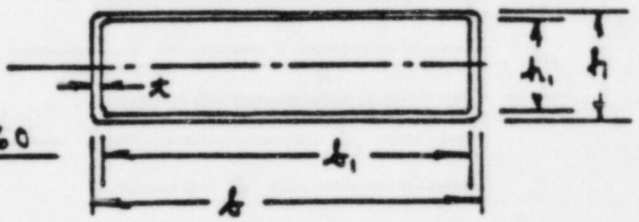
$F_m = \frac{\cancel{310}\ 269 + \cancel{319.5}\ 262.5 + \cancel{324.4}\ 265}{3} = \cancel{320}\ 265$ ★

$F_x = \frac{429.4 + 423 + 420}{3} = 424$ ✓

USING EULERS COLUMN FORMULA: $P = \frac{\pi^2 I E}{l^2}$

WHERE: P = ALLOWABLE LOAD LBS
E = 29.5×10^6
l = 96"
$I = \frac{1}{12}[b h^3 - b_1 h_1^3]$
b = 4.0 h = 1.626 t = .060
I = .328

1.396
1.383 } 1.40 ✓
1.421



$F_l = \frac{P}{8} = \frac{\pi^2 \times .328 \times 29.5 \times 10^6}{8 \times 96^2} = 1{,}295$

THE FOLLOWING CRITERIA MUST BE MET:

$$1.6\left[\frac{f'_m}{F_m} + \sqrt{\left(\frac{f_m}{F_m}\right)^2 + \left(\frac{f_x}{F_x}\right)^2 + \left(\frac{f_l}{F_l}\right)^2}\right] \leq 1.0$$

WHERE f'_m = SPECIFICATION LOADING
f_m = 2.5 × SPECIFICATION LOADING
f_x = 4.0 × SPECIFICATION LOADING
f_l = 4.0 × SPECIFICATION LOADING

CYPRUS

$$\frac{35}{\cancel{320}\ 265} + \sqrt{\left(\frac{2.5\times35}{\cancel{320}\ 265}\right)^2 + \left(\frac{4.0\times35}{424}\right)^2 + \left(\frac{4.0\times35}{1{,}295}\right)^2} \leq .625$$

$$\cancel{.10937}\ .13208 + \sqrt{\cancel{.09477}\ .10902 + .10902 + .01169} \leq .625$$

$$\cancel{.10937}\ .13208 + \cancel{.44215}\ .47930 \leq .625$$

$$\cancel{.55152}\ .61138 \leq .625 \quad \therefore \text{ O.K.}$$

TRAY DESCRIPTION: GG-1256-12-S's-CP

16 GA 33 KSI SIDE MEMBER 16 GA RUNG

SPLICE AT MID-SPAN

05-CP-13

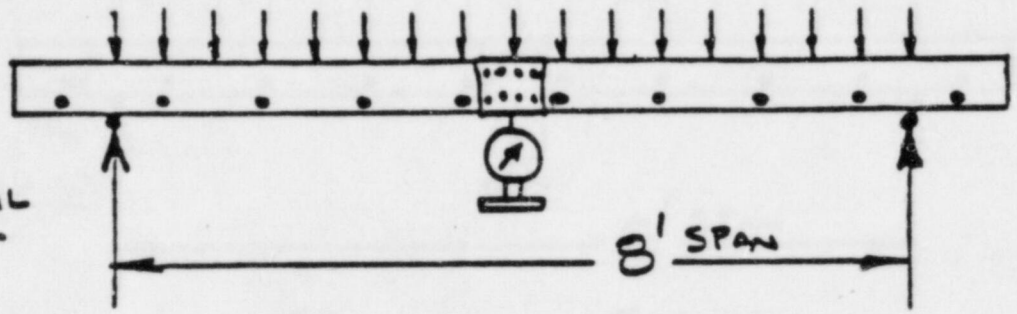
TEST NO.

5/19/77

DATE

B.B.

BY



VERTICAL TEST

			FAR		NEAR			
NO. OF BUNDLES	TOTAL WEIGHT LBS.	LBS/FT.	#1 INDICATOR READING	#1 INDICATOR Δ"	#2 INDICATOR READING	#2 INDICATOR Δ"	MAX AVERAGE Δ"	COMMENTS
0	0	0	4.250	0	3.700	0	0	
12	120	15	4.201	.049	3.635	.065	.057	
24	240	30	4.141	.109	3.577	.123	.116	
36	360	45	4.098	.152	3.515	.185	.168	
48	480	60	4.046	.204	3.454	.246	.225	
60	600	75	3.993	.257	3.409	.291	.274	
72	720	90	3.934	.316	3.337	.363	.339	
84	840	105	3.886	.364	3.262	.438	.401	
96	960	120	3.829	.421	3.209	.491	.456	
108	1080	135	3.791	.459	3.135	.565	.512	
120	1200	150	3.720	.530	3.096	.604	.567	
132	1320	165	3.677	.573	3.011	.689	.631	
144	1440	180	3.604	.646	2.946	.754	.700	
156	1560	195	3.527	.723	2.867	.833	.778	
168	1680	210	3.456	.794	2.748	.952	.873	
180	1800	225	3.348	.902	2.640	1.060	.981	
192	1920	240	3.206	1.044	2.489	1.212	1.128	
204	2040	255	3.087	1.163	2.347	1.353	1.258	
212	2120	265	—	—	—	—	—	COLLAPSE

CYPRUS

50

TRAY DESCRIPTION: GG-1256-12-E, -CP

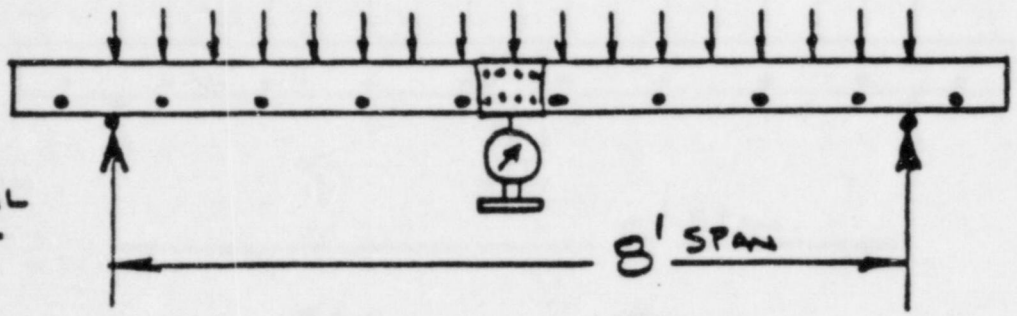
16 GA 33 KSI SIDE MEMBER 16 GA RUNG

SPLICE AT MID-SPAN

05-CP-12
TEST NO.

5/19/77
DATE

B.B.
BY



VERTICAL TEST

			FAR		NEAR			
NO. OF BUNDLES	TOTAL WEIGHT LBS.	LBS/FT.	#1 INDICATOR READING	#1 INDICATOR Δ"	#2 INDICATOR READING	#2 INDICATOR Δ"	MAX AVERAGE Δ"	COMMENTS
0	0	0	3.550	0	2.300	0	0	
12	120	15	3.499	.051	2.231	.069	.060	
24	240	30	3.439	.111	2.167	.133	.122	
36	360	45	3.384	.166	2.112	.188	.177	
48	480	60	3.371	.179	2.039	.261	.248	
60	600	75	3.275	.275	1.983	.317	.296	
72	720	90	3.205	.345	1.911	.389	.367	
84	840	105	3.144	.406	1.850	.450	.428	
96	960	120	3.109	.441	1.779	.521	.481	
108	1080	135	3.050	.500	1.714	.586	.543	
120	1200	150	3.024	.526	1.642	.658	.592	
132	1320	165	2.927	.623	1.576	.724	.673	
144	1440	180	2.885	.665	1.501	.799	.732	
156	1560	195	2.800	.750	1.412	.888	.819	
168	1680	210	2.675	.875	1.303	.999	.936	
180	1800	225	2.538	1.012	1.158	1.142	1.077	
192	1920	240	2.415	1.135	1.019	1.281	1.208	
204	2040	255	2.271	1.279	.877	1.423	1.351	
210	2100	262.5	—	—	—	—	—	COLLAPSE

CYPRUS

51

TRAY DESCRIPTION: GG-1256-12-06-CP

16 GA 33 KSI SIDE MEMBER 16 GA R.N6

SPLICE AT MID-SPAN

05-C-10

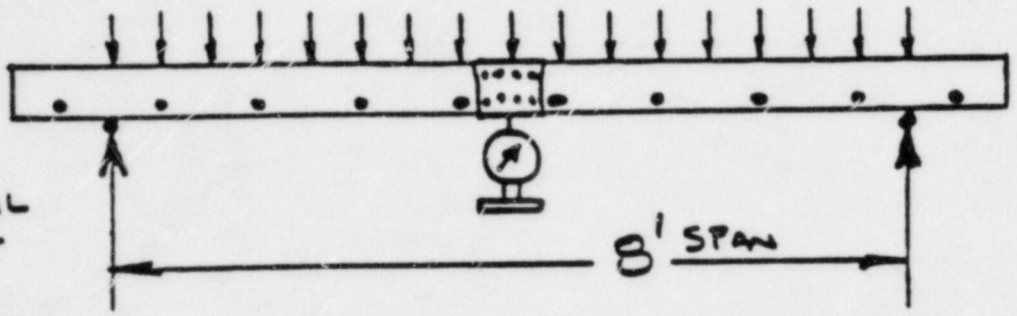
TEST NO.

5/17/77

DATE

B.B.

BY



VERTICAL TEST

			FAR		NEAR			
NO. OF BUNDLES	TOTAL WEIGHT LBS.	LBS/FT.	#1 INDICATOR READING	#1 INDICATOR Δ"	#2 INDICATOR READING	#2 INDICATOR Δ"	MAX AVERAGE Δ"	COMMENTS
0	0	0	2.937	0	3.400	0	0	
12	120	15	2.880	.057	3.346	.054	.056	
24	240	30	2.814	.123	3.289	.111	.117	
36	360	45	2.759	.178	3.238	.162	.170	
48	480	60	2.709	.230	3.191	.209	.220	
60	600	75	2.648	.289	3.157	.243	.266	
72	720	90	2.588	.349	3.087	.313	.331	
84	840	105	2.523	.414	3.036	.364	.389	
96	960	120	2.466	.471	2.975	.425	.448	
108	1080	135	2.412	.525	2.921	.479	.502	
120	1200	150	2.350	.587	2.879	.521	.554	
132	1320	165	2.285	.652	2.812	.588	.620	
144	1440	180	2.228	.709	2.747	.653	.681	
156	1560	195	2.142	.795	2.669	.731	.763	
168	1680	210	2.052	.885	2.581	.819	.852	
180	1800	225	1.956	.981	2.513	.889	.934	
192	1920	240	1.791	1.146	2.368	1.032	1.089	
204	2040	255	1.671	1.266	2.224	1.176	1.221	
215	2150	269	—	—	—	—	—	COLLAPSE

CERTIFIED CYPRUS

52

DESCRIPTION GG-12SL-12-06

16 GA. SIDE RAIL 33KSI 16 GA. RUNG

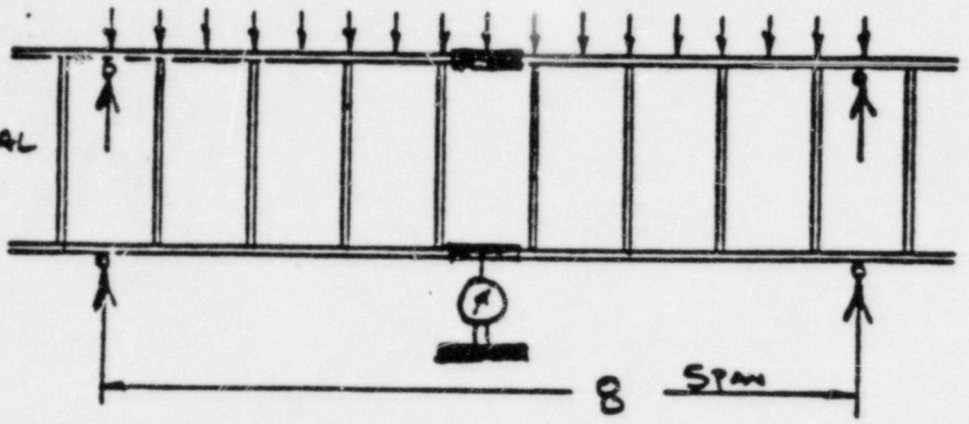
SPLICE AT MID-SPAN

TEST NO.

5/25/77
DATE

B.B.
BY

HORIZONTAL TEST



No. OF BUNDLES	TOTAL WEIGHT LBS.	TOTAL WEIGHT LBS/2	LBS/FT	INDICATOR READING	Δ"	COMMENTS
0	0	0	0	2.600	0	
90	900	450	56.25	2.500	.100	
120	1200	600	75	2.469	.131	
150	1500	750	93.75	2.426	.174	
180	1800	900	112.5	2.386	.214	
210	2100	1050	131.25	2.357	.243	
240	2400	1200	150	2.303	.297	
270	2700	1350	168.75	2.259	.341	
300	3000	1500	187.5	2.218	.382	
330	3300	1650	206.25	2.153	.447	
360	3600	1800	225	2.091	.509	
390	3900	1950	243.75	2.007	.593	
420	4200	2100	262.5	1.926	.674	
450	4500	2250	281.25	1.822	.778	
480	4800	2400	300	1.734	.866	
510	5100	2550	318.75	1.629	.971	
540	5400	2700	337.50	1.504	1.096	
570	5700	2850	356.25	1.379	1.221	
600	6000	3000	375	1.228	1.372	
630	6300	3150	393.75	1.041	1.559	
660	6600	3300	412.5	.795	1.805	
672	6720	3360	420	—	—	COLLAPSE

CERTIFIED CYPRUS

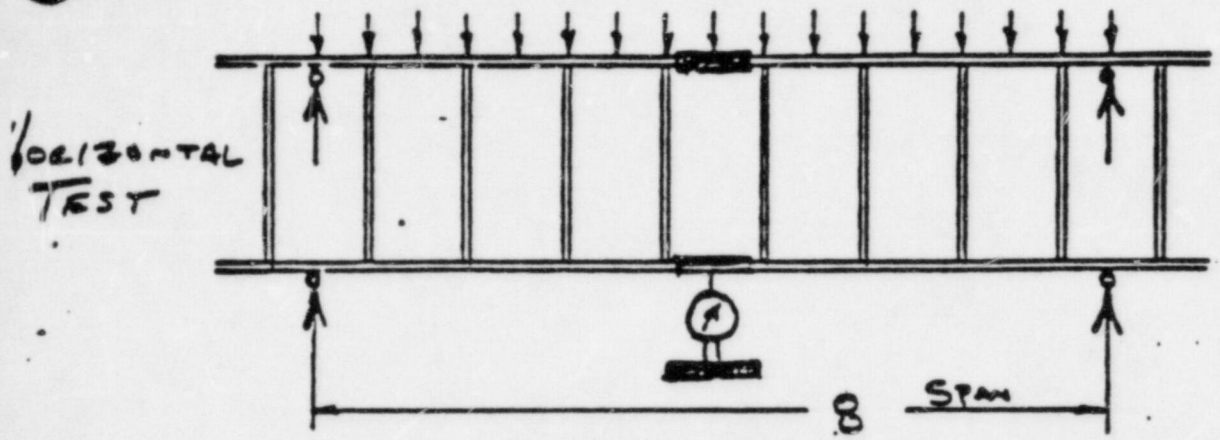
59

TRAY DESCRIPTION: GG-12SL-12-06-[illegible]

16 GA. SIDE RAIL 33KSI 16 GA. RUNG

SPLICE AT MID-SPAN

05-CP-03
TEST NO.
5/10/77
DATE
B.B.
BY



No. OF BUNDLES	TOTAL WEIGHT LBS.	TOTAL WEIGHT LBS/2	LBS/FT	INDICATOR READING	Δ"	COMMENTS
0	0	0	0	4.370	0	
90	900	450	56.25	4.269	.101	
120	1200	600	75	4.238	.132	
150	1500	750	93.75	4.193	.177	
180	1800	900	112.5	4.154	.216	
210	2100	1050	131.25	4.122	.248	
240	2400	1200	150	4.067	.303	
270	2700	1350	168.75	4.023	.347	
300	3000	1500	187.5	3.981	.389	
330	3300	1650	206.25	3.914	.456	
360	3600	1800	225	3.849	.521	
390	3900	1950	243.75	3.769	.601	
420	4200	2100	262.5	3.681	.689	
450	4500	2250	281.25	3.578	.792	
480	4800	2400	300	3.471	.899	
510	5100	2550	318.75	3.361	1.009	
540	5400	2700	337.50	3.253	1.117	
570	5700	2850	356.25	3.102	1.268	
600	6000	3000	375	2.954	1.416	
630	6300	3150	393.75	2.764	1.606	
660	6600	3300	412.5	2.503	1.867	
17	6770	3385	423	—	—	COLLAPSE

CYPRUS

60

TRAY DESCRIPTION: GG-12SL-12-06-LP

16 GA. SIDE RAIL 33KSI 16 GA. RUNG

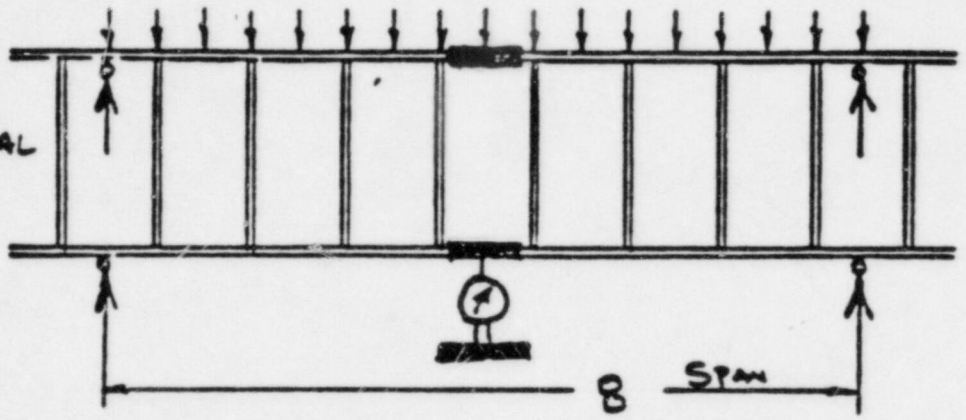
SPLICE AT MID-SPAN

TEST NO. C2-LT-01

DATE 5/9/77

BY

HORIZONTAL TEST



No. OF BUNDLES	TOTAL WEIGHT LBS.	TOTAL WEIGHT LBS/2	LBS/FT	INDICATOR READING	Δ"	COMMENTS
0	0	0	0	3.975	0	
90	900	450	56.25	3.872	.103	
120	1200	600	75	3.840	.135	
150	1500	750	93.75	3.796	.179	
180	1800	900	112.5	3.754	.221	
210	2100	1050	131.25	3.720	.255	
240	2400	1200	150	3.664	.311	
270	2700	1350	168.75	3.615	.360	
300	3000	1500	187.5	3.573	.402	
330	3300	1650	206.25	3.504	.471	
360	3600	1800	225	3.436	.539	
390	3900	1950	243.75	3.353	.622	
420	4200	2100	262.5	3.260	.715	
450	4500	2250	281.25	3.146	.829	
480	4800	2400	300	3.043	.932	
510	5100	2550	318.75	2.914	1.061	
540	5400	2700	337.50	2.792	1.183	
570	5700	2850	356.25	2.641	1.334	
600	6000	3000	375	2.490	1.485	
630	6300	3150	393.75	2.262	1.713	
660	6600	3300	412.50	1.991	1.984	
687	6870	3435	429.4	—	—	COLLAPSE

CYPRUS

61

CALCULATIONS FOR CYPRUS CABLE TRAY CAT NO. GG-06SL-12-06
TO COMANCHE PEAK S.E.S. SPECIFICATION # 2323-ES-19

SPECIFICATION LOADING IS 18 LBS/FT.

4 x 13/16 x 11...
I_x = 1.1...

FROM ATTACHED TEST DATA:

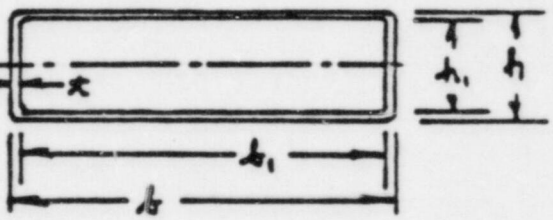
$$F_m = \frac{269}{310} + \frac{262.5}{319.5} + \frac{265}{324.4} \div 3 = \frac{265}{320}$$

$$F_x = \frac{269}{318} + \frac{262.5}{317.5} + \frac{265}{324.4} \div 3 = \frac{265}{320}$$

1.102
1.135
1.124
} 1.122 >

USING EULERS COLUMN FORMULA: $P = \frac{\pi^2 I E}{l^2}$

WHERE: P = ALLOWABLE LOAD LBS
$E = 29.5 \times 10^6$
$l = 96''$
$I = 1/2 [b h^3 - b_1 h_1^3]$
$b = 4.0$ $h = 1.626$ $t = .060$
$I = .328$



$$F_l = \frac{P}{8'} = \frac{\pi^2 \times .328 \times 29.5 \times 10^6}{8 \times 96^2} = 1{,}295$$

THE FOLLOWING CRITERIA MUST BE MET:

$$1.6\left[\frac{f'_m}{F_m} + \sqrt{\left(\frac{f_m}{F_m}\right)^2 + \left(\frac{f_x}{F_x}\right)^2 + \left(\frac{f_l}{F_l}\right)^2}\right] \leq 1.0$$

CYPRUS

WHERE f'_m = SPECIFICATION LOADING
f_m = 2.5 x SPECIFICATION LOADING
f_x = 4.0 x SPECIFICATION LOADING
f_l = 4.0 x SPECIFICATION LOADING

$$\frac{18}{265} + \sqrt{\left(\frac{2.5 \times 18}{265}\right)^2 + \left(\frac{4 \times 18}{320}\right)^2 + \left(\frac{4 \times 18}{1{,}295}\right)^2} \leq .625$$

$$.0679 + \sqrt{.02808^2 + .0506^2 + .00309} \leq .625$$

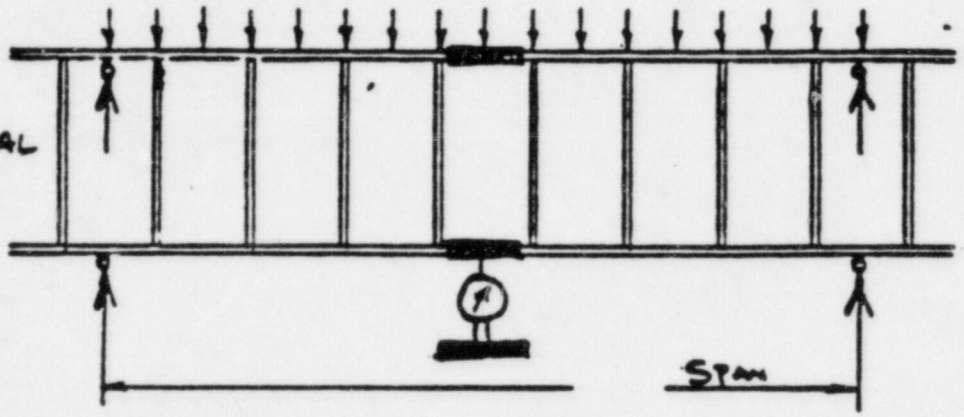
$$.0679 + .3551 \leq .625$$

$$.4230 \leq .625 \quad \therefore \text{O.K.}$$

TRAY DESCRIPTION: GG-065L-12-06-CP
16 GA. SIDE RAIL 33 KSI 16 GA. RUNG
SPLICE AT MIDSPAN

OS-CP-21
TEST NO.
5/31/71
DATE
B.B.
BY

HORIZONTAL TEST



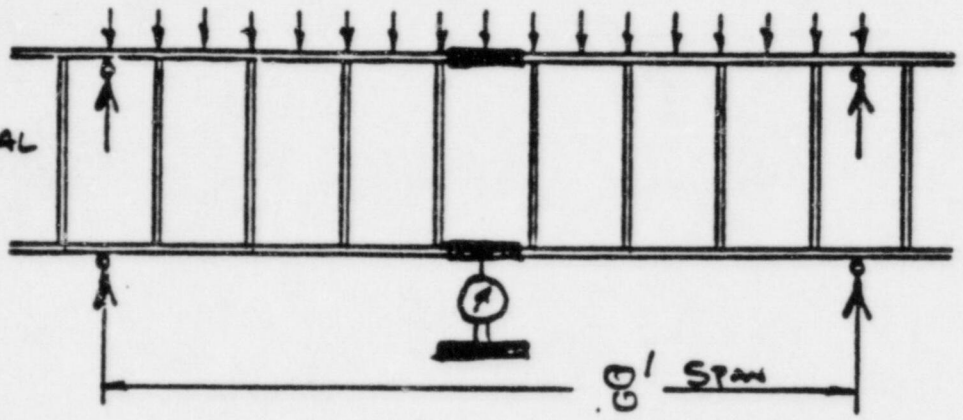
No. OF BUNDLES	TOTAL WEIGHT LBS.	TOTAL WEIGHT LBS/2	LBS/FT	INDICATOR READING	Δ"	COMMENTS
0	0	0	0	3.220	0	
60	600	300	37.5	3.124	.096	
90	900	450	56.25	3.073	.147	
120	1200	600	75.0	3.032	.188	
150	1500	750	93.75	2.995	.225	
180	1800	900	112.5	2.951	.269	
210	2100	1050	131.25	2.893	.327	
240	2400	1200	150	2.820	.400	
270	2700	1350	168.75	2.750	.470	
300	3000	1500	187.5	2.661	.559	
330	3300	1650	206.25	2.542	.678	
360	3600	1800	225	2.436	.784	
390	3900	1950	243.75	2.301	.919	
420	4200	2100	262.5	2.150	1.070	
450	4500	2250	281.25	2.002	1.218	
480	4800	2400	300	1.849	1.371	
509	5090	2545	318	—	—	COLLAPSE

CERTIFIED CYPRUS LIMITED

56

TRAY DESCRIPTION: GG-065L-12-06-CP
16 GA. SIDE RAIL 37 KSI 16 GA. RUNG
SPLICE AT MIDSPAN

05-CP-22
TEST NO.
6/1/77
DATE
B.P.
BY



No. of Bundles	Total Weight Lbs.	Total Weight Lbs/2	Lbs/Ft	Indicator Reading	Δ"	Comments
0	0	0	0	3.645	0	
60	600	300	37.5	3.548	.097	
90	900	450	56.25	3.496	.149	
120	1200	600	75.0	3.454	.191	
150	1500	750	93.75	3.415	.230	
180	1800	900	112.5	3.367	.278	
210	2100	1050	131.25	3.313	.332	
240	2400	1200	150	3.246	.399	
270	2700	1350	168.75	3.162	.483	
300	3000	1500	187.5	3.065	.580	
330	3300	1650	206.25	2.930	.715	
360	3600	1800	225	2.836	.809	
390	3900	1950	243.75	2.678	.967	
420	4200	2100	262.5	2.512	1.133	
450	4500	2250	281.25	2.359	1.286	
480	4800	2400	300	2.207	1.438	
508	5080	2540	317.5		—	COLLAPSE

CERTIFIED CYPRUS LIMITED

57

IRAY DESCRIPTION: GG-06SL-12-06-C1

16 GA. SIDE RAIL 33 KSI 16 GA. RUNG

SPLICE AT MIDSPAN

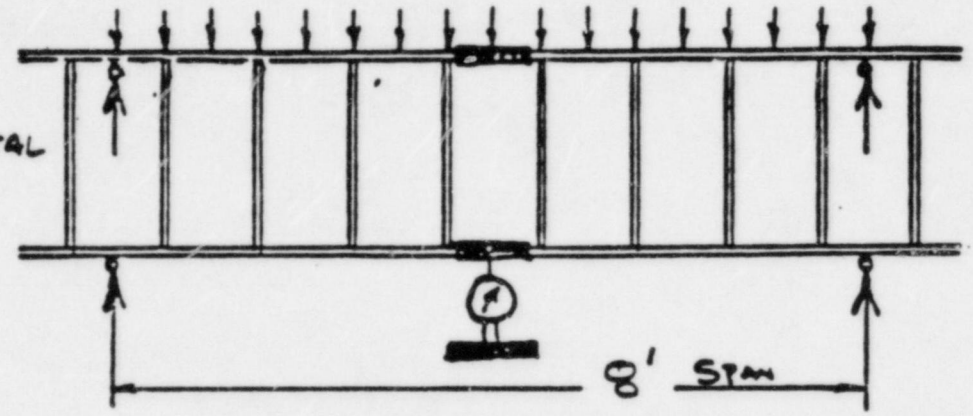
TEST NO. 6/1/99

DATE

B.B.

BY

HORIZONTAL TEST



No. of Bundles	Total Weight Lbs.	Total Weight Lbs/2	Lbs/Ft	Indicator Reading	Δ"	Comments
0	0	0	0	4.179	0	
60	600	300	37.5	4.085	.094	
90	900	450	56.25	4.036	.143	
120	1200	600	75.0	3.999	.180	
150	1500	750	93.75	3.960	.219	
180	1800	900	112.5	3.916	.263	
210	2100	1050	131.25	3.861	.318	
240	2400	1200	150	3.798	.381	
270	2700	1350	168.75	3.710	.469	
300	3000	1500	187.5	3.637	.542	
330	3300	1650	206.25	3.526	.653	
360	3600	1800	225	3.431	.748	
390	3900	1950	243.75	3.299	.880	
420	4200	2100	262.5	3.148	1.031	
450	4500	2250	281.25	3.000	1.179	
480	4800	2400	300	2.883	1.296	
510	5100	2550	318.75	2.776	1.403	
519	5190	2595	324.4	—	—	COLLAPSE

CERTIFIED CYPRUS

58

CALCULATIONS FOR CYPRUS CABLE TRAY CAT NO. GF-36SL
TO COMANCHE PEAK S.E.S. SPECIFICATION # 2323-ES-19

SPECIFICATION LOADING IS 105 LBS/FT.

4 x 1½ x 12 GA
I_x = 3.40 Δ

FROM ATTACHED TEST DATA:

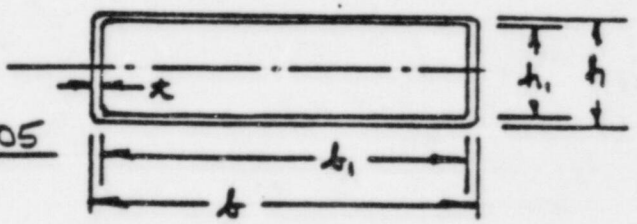
F_m = 898 + 931 + 946 ÷ 3 = 925

F_x = 1032 + 977 + 1021 ÷ 3 = 1,010

USING EULERS COLUMN FORMULA: $P = \frac{\pi^2 I E}{l^2}$

WHERE: P = ALLOWABLE LOAD LBS
E = 29.5×10^6
l = 96"
$I = \frac{1}{12}[b h^3 - b_1 h_1^3]$
b = 4.0 h = 2.5 t = .105
I = 1.415

3.073
2.961 } 2.966 Δ
2.914



$F_l = \frac{P}{8} = \frac{\pi^2 \times 1.415 \times 29.5 \times 10^6}{8 \times 96^2} = 5,588$

THE FOLLOWING CRITERIA MUST BE MET:

$$1.6\left[\frac{f'_m}{F_m} + \sqrt{\left(\frac{f_m}{F_m}\right)^2 + \left(\frac{f_x}{F_x}\right)^2 + \left(\frac{f_l}{F_l}\right)^2}\right] \leq 1.0$$

WHERE f'_m = SPECIFICATION LOADING
f_m = 2.5 x SPECIFICATION LOADING
f_x = 4.0 x SPECIFICATION LOADING
f_l = 4.0 x SIFCIFICATION LOADING

CYPRUS

$$\frac{105}{925} + \sqrt{\left(\frac{2.5 \times 105}{925}\right)^2 + \left(\frac{4.0 \times 105}{1010}\right)^2 + \left(\frac{4.0 \times 105}{5588}\right)^2} \leq .625$$

$$.113 + \sqrt{.080 + .173 + .0056} \leq .625$$

$$.113 + .508 \leq .625$$

$$.621 \leq .625 \therefore \text{OK}$$

TRAY DESCRIPT. NO. GF-36SL-12

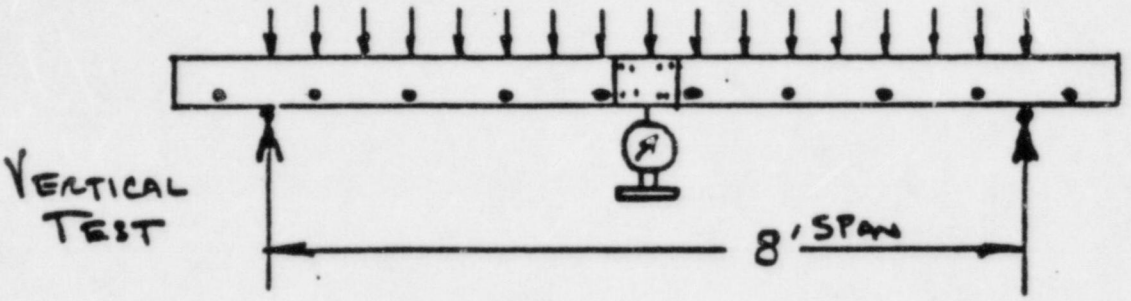
36"W Solid Trof — conn @ MID-SPAN

12GA. 50 KSI — 4x1¼ SIDES

TEST NO. 12-13-77

DATE

BY BLB



NO. OF BUNDLES	TOTAL WEIGHT LBS.	LBS/FT.	#1 INDICATOR READING	#1 INDICATOR Δ"	#2 INDICATOR READING	#2 INDICATOR Δ"	MAX AVERAGE Δ"	COMMENTS
0	0	0	4.677	0	4.734	0	0	
40	400	50	4.596	.081	4.649	.085	.083	
80	800	100	4.519	.158	4.568	.166	.162	
120	1200	150	4.464	.213	4.508	.226	.220	
160	1600	200	4.397	.280	4.441	.293	.287	
200	2000	250	4.337	.340	4.378	.356	.348	
240	2400	300	4.265	.412	4.302	.432	.422	
280	2800	350	4.176	.501	4.208	.526	.514	
320	3200	400	4.089	.588	4.116	.618	.603	
360	3600	450	3.991	.686	4.016	.718	.702	
400	4000	500	3.896	.781	3.921	.813	.797	
440	4400	550	3.794	.883	3.815	.919	.901	
480	4800	600	3.690	.987	3.703	1.031	1.009	
520	5200	650	3.574	1.103	3.582	1.152	1.128	
560	5600	700	2.945	1.232	3.444	1.290	1.261	
600	6000	750	3.326	1.351	3.319	1.415	1.383	
640	6400	800	3.165	1.512	3.150	1.584	1.548	
680	6800	850	2.970	1.707	2.949	1.785	1.746	
720	7200	900	2.738	1.939	2.710	2.024	1.982	
745	7450	931	—	—	—	—	—	COLLAPSE

CYPRUS

GF-36SL

GF-36SL

137

TRAY DESCRIPT. NO. GF-365L-72

36" WIDE SOLID TROF CONN. @ MID-SPAN

12 GA. 50KSI 4x1½ SIDE

1 - 1 - 1

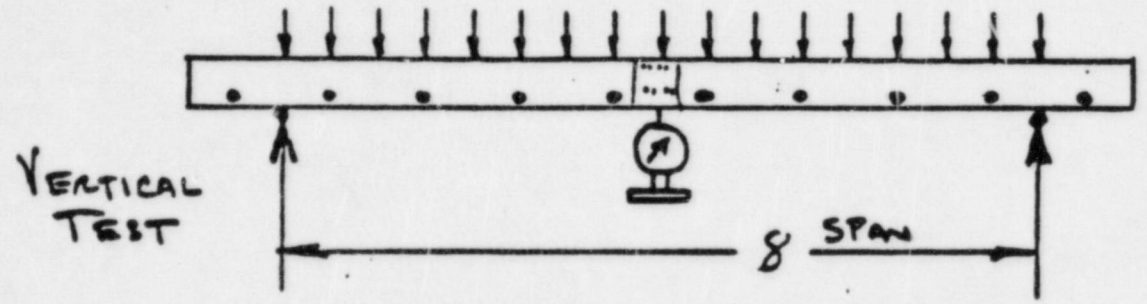
TEST NO.

12-12-77

DATE

BB

BY



NO. OF BUNDLES	TOTAL WEIGHT LBS.	LBS/FT.	#1 INDICATOR READING	#1 INDICATOR Δ"	#2 INDICATOR READING	#2 INDICATOR Δ"	MAX AVERAGE Δ"	COMMENTS
0	0	0	4.414	0	3.915	0	0	
40	400	50	4.325	.089	3.822	.093	.091	
80	800	100	4.246	.168	3.745	.170	.169	
120	1200	150	4.187	.227	3.684	.231	.229	
160	1600	200	4.120	.294	3.614	.301	.298	
200	2000	250	4.058	.356	3.549	.366	.361	
240	2400	300	3.985	.429	3.475	.440	.435	
280	2800	350	3.895	.519	3.380	.535	.527	
320	3200	400	3.904	.610	3.285	.630	.620	
360	3600	450	3.696	.718	3.171	.744	.731	
400	4000	500	3.610	.804	3.083	.832	.818	
440	4400	550	3.507	.907	3.024	.891	.923	
480	4800	600	3.399	1.015	2862	1.053	1.034	
520	5200	650	3.277	1.137	2.738	1.177	1.157	
560	5600	700	3.153	1.261	2.609	1.306	1.284	
600	6000	750	3.017	1.397	2.468	1.447	1.422	
640	6400	800	2.837	1.577	2.280	1.635	1.606	
680	6800	850	2.601	1.813	2.036	1.879	1.846	
718	7180	898	—	—	—	—	—	COLLAPSE

CERTIFIED CYPRUS

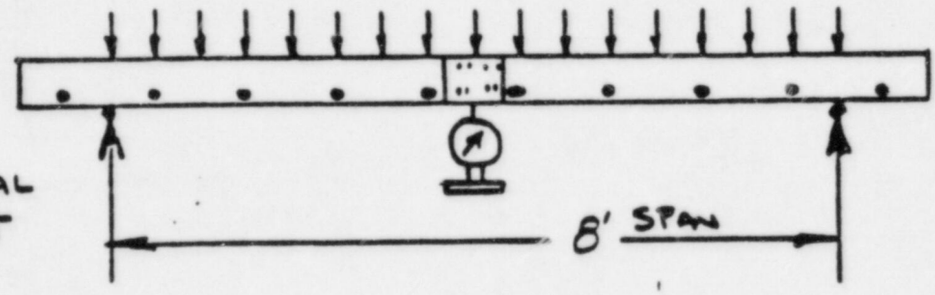
GF-365L

TRAY DESCRIPTION: GF-36SL-12

36"W SOLID TROF — SPLICE at MID SPAN

12 GA — 50 KSI — 4x1¼ SIDERAILS

12-CP-08
TEST NO.
12-13-77
DATE
BB
BY



NO. OF BUNDLES	TOTAL WEIGHT LBS.	LBS/FT.	#1 INDICATOR READING	#1 INDICATOR Δ"	#2 INDICATOR READING	#2 INDICATOR Δ"	MAX AVERAGE Δ"	COMMENTS
0	0	0	4.410	0	4.075	0	0	
40	400	50	4.326	.084	3.995	.080	.082	
80	800	100	4.246	.164	3.919	.156	.160	
120	1200	150	4.188	.222	3.861	.214	.218	
160	1600	200	4.121	.289	3.798	.277	.283	
200	2000	250	4.060	.350	3.741	.334	.342	
240	2400	300	3.987	.423	3.673	.402	.413	
280	2800	350	3.894	.516	3.590	.485	.501	
320	3200	400	3.804	.606	3.503	.572	.589	
360	3600	450	3.706	.704	3.411	.664	.684	
400	4000	500	3.611	.799	3.318	.757	.778	
440	4400	550	3.507	.903	3.226	.849	.876	
480	4900	600	3.401	1.009	3.128	.947	.978	
520	5200	650	3.277	1.133	3.012	1.063	1.098	
560	5600	700	3.151	1.259	2.888	1.187	1.223	
600	6000	750	3.032	1.378	2.772	1.303	1.341	
640	6400	800	2.871	1.539	2.628	1.447	1.493	
680	6800	850	2.675	1.735	2.438	1.637	1.686	
720	7200	900	2.441	1.969	2.216	1.859	1.914	
757	7570	946	—	—	—	—	—	COLLAPSE

CERTIFIED CYPRUS

CALCULATIONS FOR CYPRUS CABLE TRAY CAT NO. F-30SL
TO COMANCHE PEAK S.E.S. SPECIFICATION # 2323-ES-19

SPECIFICATION LOADING IS 88 LBS/FT.

4 x 1/4 x 12GA. I_x = 3.640 △

FROM ATTACHED, TEST DATA:

F_m = 898 + 931 + 946 ÷ 3 = 925

F_x = 1026 + 1070 + 1042 ÷ 3 = 1046

USING EULERS COLUMN FORMULA: $P = \frac{\pi^2 I E}{l^2}$

WHERE: P = ALLOWABLE LOAD LBS

E = 29.5×10^6

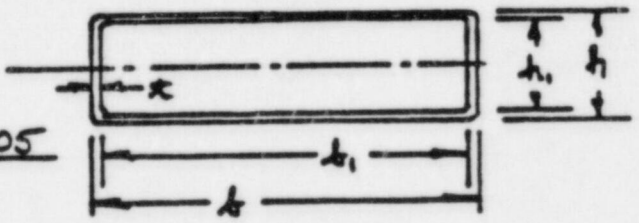
l = 96"

$I = \frac{1}{2}[bh^3 - b_1 h_1^3]$

b = 4.0 h = 2.5 t = .105

I = 1.415

3.787, 3.630, 3.502 } 3.640 > △



$F_l = \frac{P}{8} = \frac{\pi^2 \times 1.415 \times 29.5 \times 10^6}{8 \times 96^2} = 5588$

THE FOLLOWING CRITERIA MUST BE MET:

$$1.6\left[\frac{f'_m}{F_m} + \sqrt{\left(\frac{f_m}{F_m}\right)^2 + \left(\frac{f_x}{F_x}\right)^2 + \left(\frac{f_l}{F_l}\right)^2}\right] \leq 1.0$$

WHERE f'_m = SPECIFICATION LOADING

f_m = 2.5 x SPECIFICATION LOADING

f_x = 4.0 x SPECIFICATION LOADING

f_l = 4.0 x SPECIFICATION LOADING

$$\frac{88}{925} + \sqrt{\left(\frac{2.5 \times 88}{925}\right)^2 + \left(\frac{4.0 \times 88}{1046}\right)^2 + \left(\frac{4.0 \times 88}{5588}\right)^2} \leq .625$$

$$.095 + \sqrt{.056 + .113 + .0039} \leq .625$$

$$.095 + .416 \leq .625$$

$$.511 \leq .625 \quad \therefore \text{OK}$$

TRAY DESCRIPTION: GF-30SL-12

30" WIDE CORR. SOLID TROF - CONN at MID-SPAN

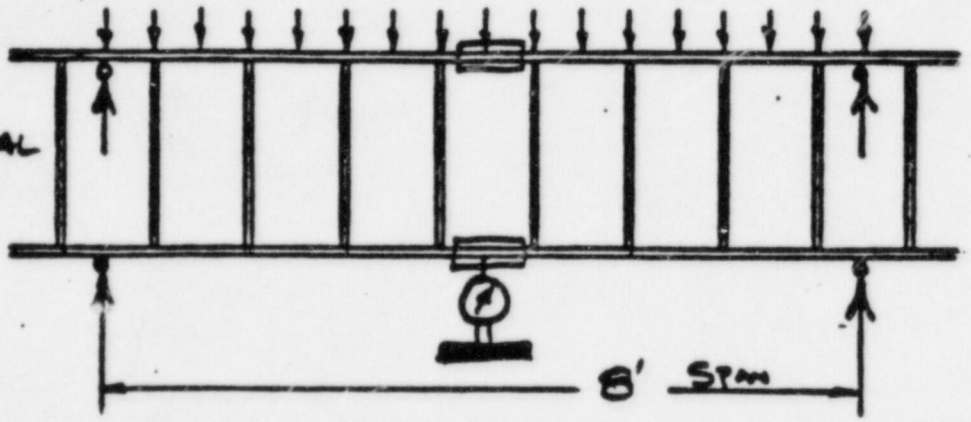
RGA. 50KSI 4x1¼ SIDERAILS

12-CP-
TEST NO.

1-4-79
DATE

BB
BY

HORIZONTAL TEST



No. of Bundles	Total Weight Lbs.	Total Weight Lbs/2	Lbs/Ft	Indicator Reading	Δ"	Comments
0	0	0	0	4.009	0	
360	3600	1800	225	3.823	.186	
432	4320	2160	270	3.783	.226	
504	5040	2520	315	3.754	.255	
576	5760	2880	360	3.720	.289	
648	6480	3240	405	3.689	.320	
720	7200	3600	450	3.649	.360	
792	7920	3960	495	3.614	.395	
864	8640	4320	540	3.582	.427	
936	9360	4680	585	3.546	.463	
1008	10080	5040	630	3.498	.511	
1080	10800	5400	675	3.447	.562	
1152	11520	5760	720	3.416	.593	
1224	12240	6120	765	3.336	.673	
1296	12960	6480	810	3.298	.721	
1368	13680	6840	855	3.228	.781	
1440	14400	7200	900	3.157	.852	
1512	15120	7560	945	3.073	.936	
1584	15840	7920	990	2.965	1.044	
1656	16560	8280	1035	2.772	1.237	
1712	17120	8560	1070	—	—	COLLAPSE

CERTIFIED CYPRUS

TRAY DESCRIPTION: GF-30SL-12

30" WIDE CORR. SOLID TRAF — CONN @ MID-SPAN

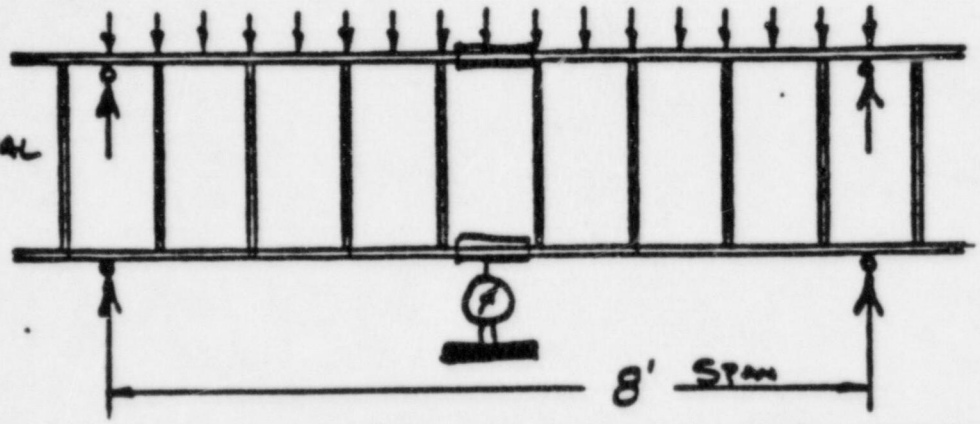
12 GA. — 50KSI 4x1¼ SIDES.

12-CP-2
TEST NO.

1-5-78
DATE

BB
BY

HORIZONTAL TEST



No. of Bundles	Total Weight Lbs.	Total Weight Lbs/2	Lbs/Ft	Indicator Reading	Δ"	Comments
0	0	0	0	3.307	0	
360	3600	1800	225	3.116	.191	
432	4320	2160	270	3.075	.232	
504	5040	2520	315	3.042	.265	
576	5760	2880	360	3.006	.301	
649	6480	3240	405	2.971	.336	
720	7200	3600	450	2.928	.379	
792	7920	3960	495	2.891	.416	
864	8640	4320	540	2.856	.451	
936	9360	4680	585	2.819	.488	
1008	10080	5040	630	2.765	.542	
1080	10800	5400	675	2.711	.596	
1152	11520	5760	720	2.676	.631	
1224	12240	6120	765	2.593	.714	
1296	12960	6480	810	2.541	.766	
1368	13680	6840	855	2.478	.829	
1440	14400	7200	900	2.399	.908	
1512	15120	7560	945	2.309	.998	
1584	15840	7920	990	2.195	1.112	
1656	16560	8280	1035	1.998	1.309	
1667	16670	8335	1042	—	—	COLLAPSE

CERTIFIED CYPRUS

TRAY DESCRIPTION: GF-30SL-12

30"W SOLID CORR. TROF CONN. at MID-SPAN

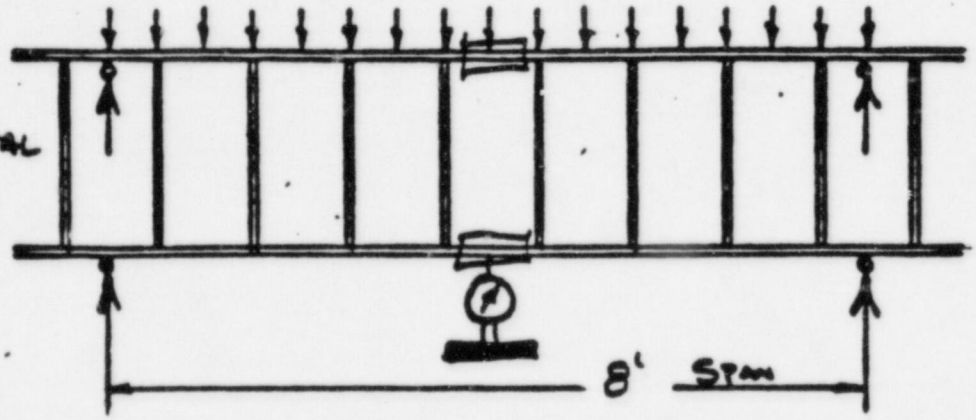
12GA. 50KSI 4x1¼ SIDERAILS

12-CP-(
TEST NO.

1-5-7(
DATE

BB
BY

HORIZONTAL TEST



No. of Bundles	Total Weight Lbs.	Total Weight Lbs/2	Lbs/Ft	Indicator Reading	Δ"	Comments
0	0	0	0	3.743	0	
360	3600	1800	225	3.539	.204	
432	4320	2160	270	3.500	.243	
504	5040	2520	315	3.465	.278	
576	5760	2880	360	3.427	.316	
648	6480	3240	405	3.394	.349	
720	7200	3600	450	3.348	.395	
792	7920	3960	495	3.308	.435	
864	8640	4320	540	3.272	.471	
936	9360	4680	585	3.231	.512	
1008	10080	5040	630	3.174	.569	
1080	10800	5400	675	3.111	.632	
1152	11520	5760	720	3.067	.676	
1224	12240	6120	765	2.975	.768	
1296	12960	6480	810	2.919	.824	
1368	13680	6840	855	2.947	.896	
1440	14400	7200	900	2.760	.983	
1512	15120	7560	945	2.656	1.087	
1584	15840	7920	990	2.529	1.214	
1641	16410	8205	1026	—	—	COLLAPSE

CERTIFIED CYPRUS

CALCULATIONS FOR CYPRUS CABLE TRAY CAT NO. GF-24SL
TO COMANCHE PEAK S.E.S. SPECIFICATION # 2323-ES-19

SPECIFICATION LOADING IS 70 LBS/FT.

4 x 1¼ x 12 GA.
$I_x = 3.40$

FROM ATTACHED TEST DATA:

$F_m = 898 + 931 + 946 \div 3 = 925$

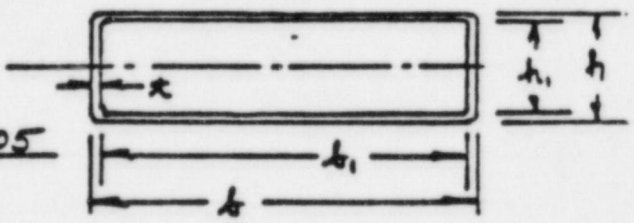
$F_x = 1059 + 1126 + 1139 \div 3 = 1108$

USING EULERS COLUMN FORMULA: $P = \frac{\pi^2 I E}{l^2}$

WHERE: P = ALLOWABLE LOAD LBS
$E = 29.5 \times 10^6$
$l = 96''$
$I = \frac{1}{12}[bh^3 - b_1 h_1^3]$

4.152
4.084 } 4.105
4.080

$b = 4.0$ $h = 2.5$ $t = .105$
$I = 1.415$



$F_l = \frac{P}{8} = \frac{\pi^2 \times 1.415 \times 29.5 \times 10^6}{8 \times 96^2} = 5588$

THE FOLLOWING CRITERIA MUST BE MET:

$$1.6\left[\frac{f'_m}{F_m} + \sqrt{\left(\frac{f_m}{F_m}\right)^2 + \left(\frac{f_x}{F_x}\right)^2 + \left(\frac{f_l}{F_l}\right)^2}\right] \leq 1.0$$

CYPRUS

WHERE f'_m = SPECIFICATION LOADING
f_m = 2.5 x SPECIFICATION LOADING
f_x = 4.0 x SPECIFICATION LOADING
f_l = 4.0 x SPECIFICATION LOADING

$$\frac{70}{925} + \sqrt{\left(\frac{2.5 \times 70}{925}\right)^2 + \left(\frac{4.0 \times 70}{1108}\right)^2 + \left(\frac{4.0 \times 70}{5588}\right)^2} \leq .625$$

$$.076 + \sqrt{.036 + .064 + .0025} \leq .625$$

$$.076 + .320 \leq .625$$

$$.396 \leq .625 \therefore \text{OK}$$

DESCRIPTION: GF-24SL-12

24"W SOLID TROF – SPLICE @ MIDSPAN

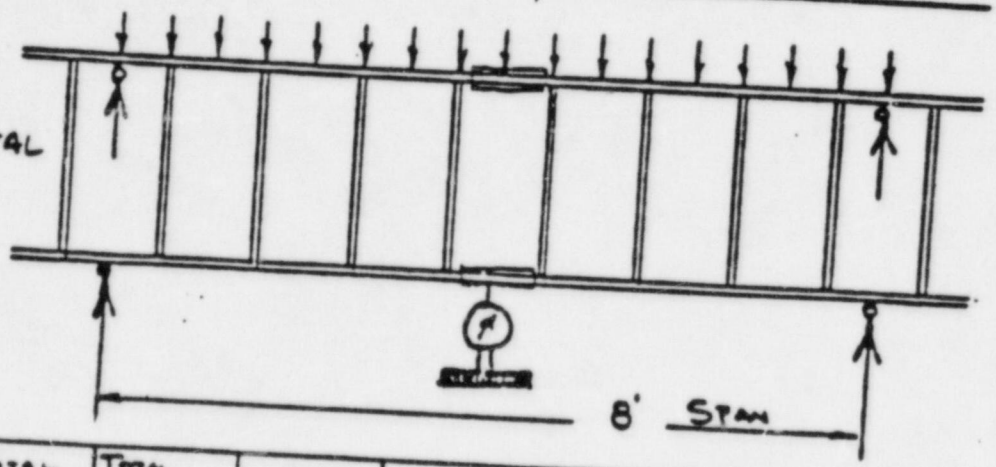
12GA-50 KSI 4x1¼ SIDERAILS

12-CP-21
TEST NO.

12-29-7
DATE

BB
BY

HORIZONTAL TEST



No. of Bundles	Total Weight Lbs.	Total Weight Lbs/2	Lbs/Ft	Indicator Reading	Δ"	Comments
0	0	0	0	4.427	0	
360	3600	1800	225	4.304	.123	
432	4320	2160	270	4.263	.164	
504	5040	2520	315	4.224	.203	
576	5760	2880	360	4.200	.227	
648	6480	3240	405	4.155	.272	
720	7200	3600	450	4.115	.312	
792	7920	3960	495	4.081	.346	
864	8640	4320	540	4.057	.370	
936	9360	4680	585	4.018	.409	
1008	10080	5040	630	3.996	.431	
1080	10800	5400	675	3.932	.495	
1152	11520	5760	720	3.897	.530	
1224	12240	6120	765	3.851	.576	
1296	12960	6480	810	3.806	.621	
1368	13680	6840	855	3.725	.702	
1440	14400	7200	900	3.663	.764	
1512	15120	7560	945	3.574	.853	
1584	15840	7920	990	3.468	.959	
1656	16560	8280	1035	3.358	1.069	
1728	17280	8640	1080	3.215	1.212	
1800	18000	9000	1125	3.078	1.349	
1801	18010	9005	1126	—	—	COLLAPSE

CERTIFIED CYPRUS

TRAY DESCRIPTION: GF-24SL-12

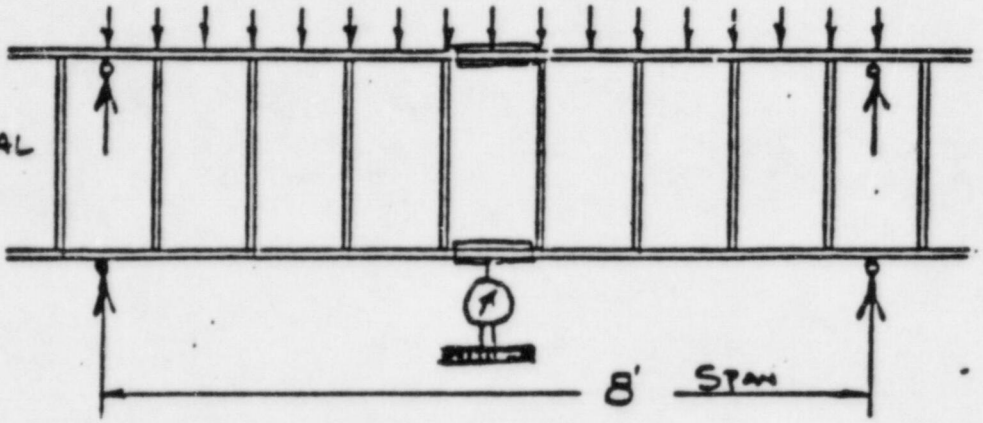
24 W Solid Tray - Splice at MIDSPAN

12 GA - 50 KSI 4 x 1¼ SIDERAILS

12-29-77
DATE

BB
BY

HORIZONTAL TEST



No. of Bundles	Total Weight Lbs.	Total Weight Lbs/2	Lbs/Ft	Indicator Reading	Δ"	Comments
0	0	0	0	4.021	0	
360	3600	1800	225	3.899	.122	
432	4320	2160	270	3.861	.160	
504	5040	2520	315	3.821	.200	
576	5760	2880	360	3.797	.224	
648	6480	3240	405	3.755	.266	
720	7200	3600	450	3.717	.304	
792	7920	3960	495	3.685	.336	
864	8640	4320	540	3.663	.358	
936	9360	4680	585	3.626	.395	
1008	10080	5040	630	3.607	.414	
1080	10800	5400	675	3.546	.475	
1152	11520	5760	720	3.515	.506	
1224	12240	6120	765	3.473	.548	
1296	12960	6480	810	3.429	.592	
1368	13680	6840	855	3.351	.670	
1440	14400	7200	900	3.292	.729	
1512	15120	7560	945	3.205	.816	
1584	15840	7920	990	3.104	.917	
1656	16560	8280	1035	2.999	1.022	
1728	17280	8640	1080	2.861	1.160	
1800	18000	9000	1125	2.729	1.292	
1822	18220	9110	1139	—	—	COLLAPSE

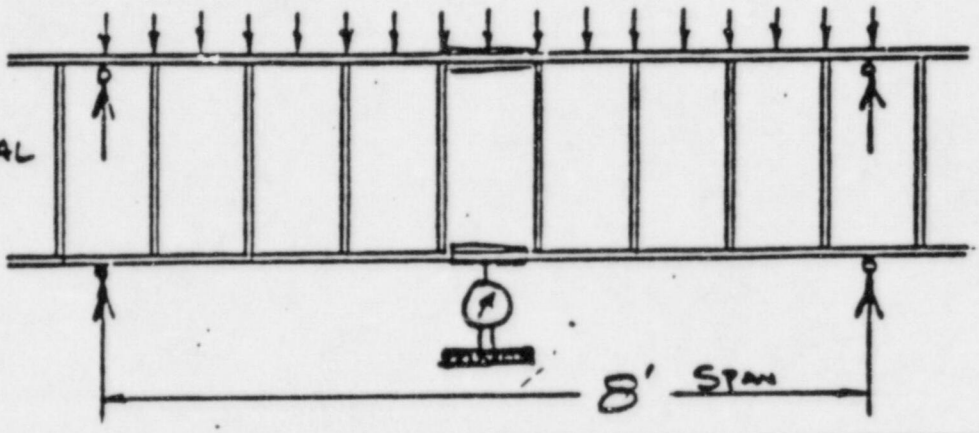
CERTIFIED CYPRUS

GF-24SL

TRAY DESCRIPTION : GF-24SL-12
24" wide Solid Corrug. TRUF. comm. mid-span
12GA - 50 KSI 4x1 1/4 SIDERAIL

12-27-77
DATE
BB
BY

HORIZONTAL TEST



No. of Bundles	Total Weight Lbs.	Total Weight Lbs/2	Lbs/Ft	Indicator Reading	Δ"	Comments
0	0	0	0	3.867	0	
360	3600	1800	225	3.740	.127	
432	4320	2160	270	3.698	.169	
504	5040	2520	315	3.656	.211	
576	5760	2880	360	3.630	.237	
648	6480	3240	405	3.581	.286	
720	7200	3600	450	3.538	.329	
792	7920	3960	495	3.503	.364	
864	8640	4320	540	3.474	.393	
936	9360	4680	585	3.431	.436	
1008	10080	5040	630	3.407	.460	
1080	10800	5400	675	3.328	.529	
1152	11520	5760	720	3.303	.564	
1224	12240	6120	765	3.254	.613	
1296	12960	6480	810	3.206	.661	
1368	13680	6840	855	3.122	.745	
1440	14400	7200	900	3.058	.809	
1512	15120	7560	945	2.966	.901	
1584	15840	7920	990	2.849	1.018	
1656	16560	8280	1035	2.710	1.157	
1694	16940	8470	1059	—	—	COLLAPSE GF-24SL

CERTIFIED CYPRUS

CALCULATIONS FOR CYPRUS CABLE TRAY CAT NO. GF-13SL
TO COMANCHE PEAK S.E.S. SPECIFICATION # 2323-ES-19

SPECIFICATION LOADING IS 53 LBS/FT.

4 x 1¼ x 14(
I_N = 2.4(

FROM ATTACHED TEST DATA:

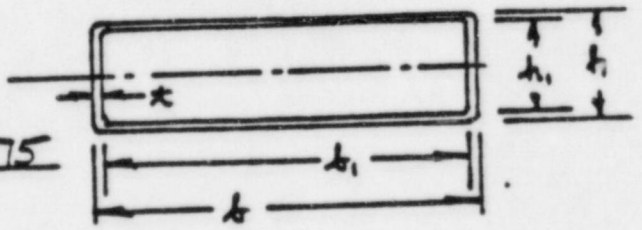
$F_m = 620 + 603 + 607 \div 3 = 610$

$F_x = 547 + 565 + 570 \div 3 = 560$

USING EULERS COLUMN FORMULA: $P = \frac{\pi^2 I E}{l^2}$

WHERE: P = ALLOWABLE LOAD LBS
E = 29.5×10^6
l = 96"
$I = \frac{1}{2}[b h^3 - b_1 h_1^3]$
b = 4.0 h = 2.5 t = .075
I = 1.045

1.150
1.168 } 1.157 >
1.153



$F_l = \frac{P}{8} = \frac{\pi^2 \times 1.045 \times 29.5 \times 10^6}{8 \times 96^2} = 4127$

THE FOLLOWING CRITERIA MUST BE MET:

$$1.6\left[\frac{f'_m}{F_m} + \sqrt{\left(\frac{f_m}{F_m}\right)^2 + \left(\frac{f_x}{F_x}\right)^2 + \left(\frac{f_l}{F_l}\right)^2}\right] \leq 1.0$$

CYPRUS

WHERE f'_m = SPECIFICATION LOADING
f_m = 2.5 x SPECIFICATION LOADING
f_x = 4.0 x SPECIFICATION LOADING
f_l = 4.0 x SPECIFICATION LOADING

$$\frac{53}{610} + \sqrt{\left(\frac{2.5 \times 53}{610}\right)^2 + \left(\frac{4.0 \times 53}{560}\right)^2 + \left(\frac{4.0 \times 53}{4127}\right)^2} \leq .625$$

$$.087 + \sqrt{.047 + .143 + .0026} \leq .625$$

$$.087 + .439 \leq .625$$

$$.526 \leq .625 \therefore \text{OK}$$

TRAY DESCRIPT #: GF-18SL-12

18"W SOLID CORR. TROF - CONN at MIDSPAN

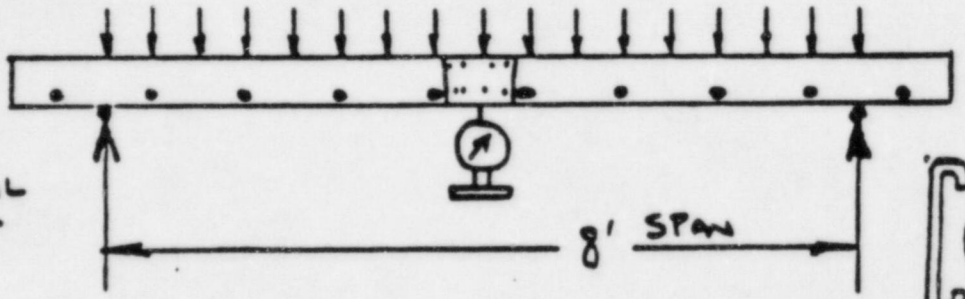
14GA 50KSI 4x1¼ SIDERAIL

L-1 C1

TEST NO. 12-7-77

DATE BB

BY



VERTICAL TEST

CYPRUS

NO. OF BUNDLES	TOTAL WEIGHT LBS.	LBS/FT.	#1 INDICATOR READING	#1 INDICATOR Δ"	#2 INDICATOR READING	#2 INDICATOR Δ"	MAX AVERAGE Δ"	COMMENTS
0	0	0	3.991	0	4.207	0	0	
24	240	30	3.917	.074	4.138	.069	.072	
48	480	60	3.853	.138	4.077	.130	.134	
72	720	90	3.772	.219	4.000	.207	.213	
96	960	120	3.713	.278	3.945	.262	.270	
120	1200	150	3.639	.352	3.876	.331	.342	
144	1440	180	3.563	.428	3.801	.406	.417	
168	1680	210	3.487	.504	3.733	.474	.489	
192	1920	240	3.402	.589	3.652	.555	.572	
216	2160	270	3.322	.669	3.580	.627	.648	
240	2400	300	3.240	.751	3.500	.707	.729	
264	2640	330	3.138	.853	3.409	.798	.826	
288	2880	360	3.055	.936	3.340	.867	.902	
312	3120	390	2.948	1.043	3.238	.969	1.006	
336	3360	420	2.857	1.134	3.160	1.047	1.091	
360	3600	450	2.744	1.247	3.060	1.147	1.197	
384	3840	480	2.641	1.350	2.971	1.236	1.293	
408	4080	510	2.532	1.459	2.876	1.331	1.395	
432	4320	540	2.415	1.576	2.781	1.426	1.501	
456	4560	570	2.274	1.717	2.665	1.542	1.630	
480	4800	600	2.134	1.857	2.538	1.669	1.762	
486	4860	607	—	—	—	—	—	COLLAPSE

(140)

TRAY DESCRIPTION: GF-18SL-12

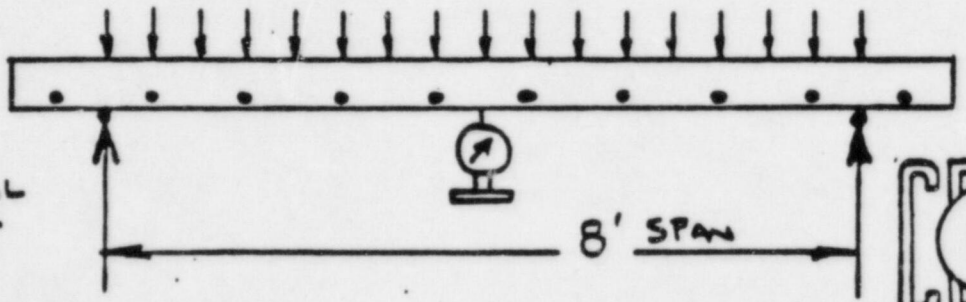
18"W SOLID COKE TROF - CONN @ MID SPAN

14 GA 50 KSI 4x1¼ SIDE MEMBERS

TEST NO. 12-8-77

DATE BB

BY



VERTICAL TEST

CYPRUS

NO. OF BUNDLES	TOTAL WEIGHT LBS.	LBS/FT.	#1 INDICATOR READING	#1 INDICATOR Δ"	#2 INDICATOR READING	#2 INDICATOR Δ"	MAX AVERAGE Δ"	COMMENTS
0	0	0	4.113		3.970	0	0	
24	240	30	4.044	.069	3.891	.079	.074	
48	480	60	3.982	.131	3.827	.143	.137	
72	720	90	3.905	.208	3.746	.224	.216	
96	960	120	3.849	.264	3.687	.283	.274	
120	1200	150	3.776	.337	3.615	.355	.346	
144	1440	180	3.705	.408	3.539	.431	.420	
168	1680	210	3.633	.480	3.464	.506	.493	
192	1920	240	3.550	.563	3.379	.591	.577	
216	2160	270	3.475	.638	3.303	.667	.653	
240	2400	300	3.392	.721	3.216	.754	.738	
264	2640	330	3.297	.816	3.122	.848	.832	
288	2880	360	3.221	.892	3.041	.929	.911	
312	3120	390	3.118	.995	2.931	1.039	1.017	
336	3360	420	3.033	1.080	2.838	1.132	1.106	
360	3600	450	2.934	1.179	2.733	1.237	1.208	
384	3840	480	2.837	1.276	2.631	1.339	1.308	
408	4080	510	2.737	1.376	2.527	1.443	1.410	
432	4320	540	2.632	1.481	2.489	1.551	1.516	
456	4560	570	2.503	1.610	2.287	1.683	1.647	
480	4800	600	2.372	1.741	2.149	1.821	1.781	
482	4820	603	—	—	—	—	—	COLLAPSE
								GF-18SL

GF-18SL-(141)

TRAY DESCRIPTION: GF-18SL-12

18" W. SOLID CORR TROF - CONN @ MID-SPAN

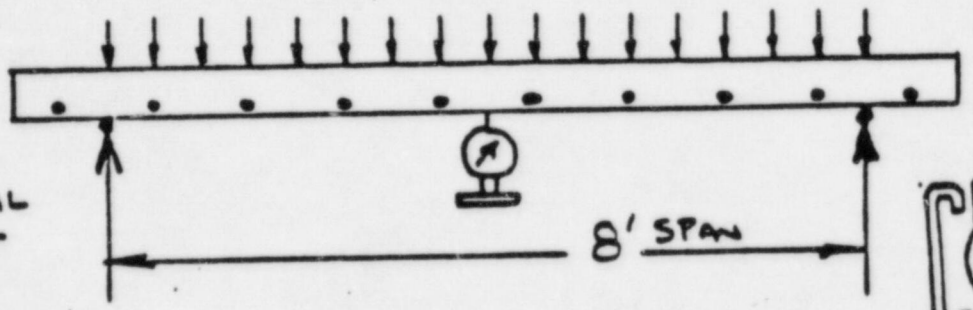
14 GA. 50KSI 4x1¼ SIDE MEMBERS

TEST NO. 12-9-77

DATE

SB

BY



VERTICAL TEST

CYPRUS

NO. OF BUNDLES	TOTAL WEIGHT LBS.	LBS/FT.	#1 INDICATOR READING	#1 INDICATOR Δ"	#2 INDICATOR READING	#2 INDICATOR Δ"	MAX AVERAGE Δ"	COMMENTS
0	0	0	3.864	0	3.960	0	0	
24	240	30	3.793	.071	3.887	.073	.072	
48	480	60	3.735	.129	3.825	.135	.132	
72	720	90	3.659	.205	3.747	.213	.209	
96	960	120	3.604	.260	3.690	.270	.265	
120	1200	150	3.534	.330	3.616	.344	.337	
144	1440	180	3.463	.401	3.539	.421	.411	
168	1680	210	3.395	.469	3.467	.493	.481	
192	1920	240	3.315	.549	3.384	.576	.563	
216	2160	270	3.244	.620	3.308	.652	.636	
240	2400	300	3.165	.699	3.228	.732	.716	
264	2640	330	3.068	.796	3.130	.830	.813	
288	2880	360	2.992	.872	3.052	.908	.890	
312	3120	390	2.893	.971	2.947	1.013	.992	
336	3360	420	2.812	1.052	2.864	1.096	1.074	
360	3600	450	2.709	1.155	2.755	1.205	1.180	
384	3840	480	2.616	1.248	2.660	1.300	1.274	
408	1080	510	2.520	1.344	2.558	1.402	1.373	
432	4320	540	2.418	1.446	2.452	1.508	1.477	
456	4560	570	2.294	1.570	2.322	1.638	1.604	
480	4800	600	2.166	1.698	2.188	1.772	1.735	
496	4960	620	—	—	—	—	—	COLLAPSE
								GF-18SL

GF-18-

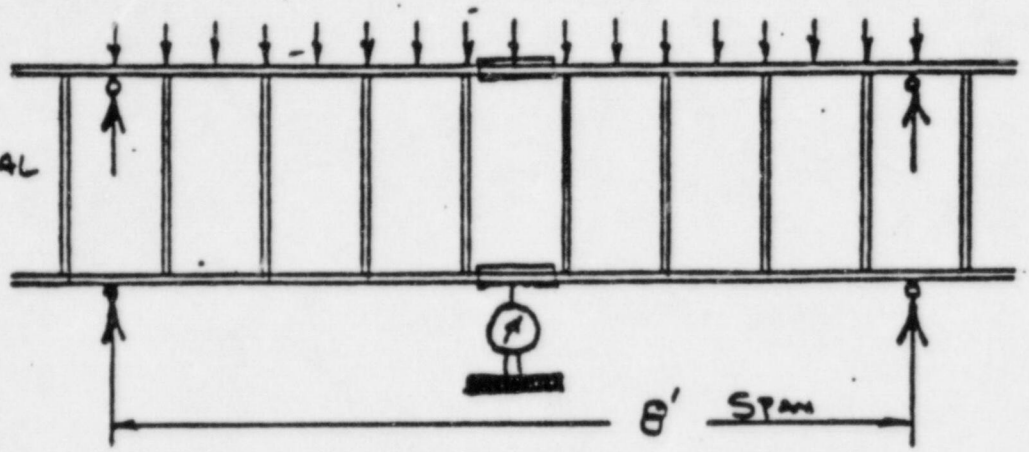
TRAY DESCRIPTION: GF-18SL12

18" W. SOLID CORR. BOTTOM W/CONN @ MID-SPAN

14GA. 50 KSI 4 x 1¼ SIDERAILS

12-[illegible]-18
TEST NO.
12-21-77
DATE
BB.
BY

HORIZONTAL TEST



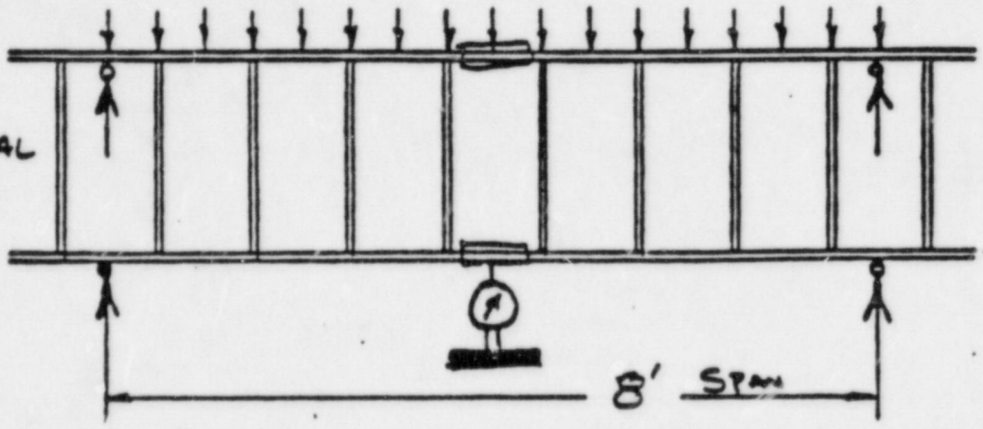
No. of Bundles	Total Weight Lbs.	Total Weight Lbs/2	Lbs/Ft	Indicator Reading	Δ"	Comments
0	0	0	0	3.605	0	
60	600	300	37.5	3.510	.095	
120	1200	600	75	3.426	.179	
180	1800	900	112.5	3.337	.268	
240	2400	1200	150	3.244	.361	
300	3000	1500	187.5	3.137	.468	
360	3600	1800	225	3.038	.567	
420	4200	2100	262.5	2.913	.692	
480	4800	2400	300	2.801	.804	
540	5400	2700	337.5	2.674	.931	
600	6000	3000	375	2.570	1.035	
660	6600	3300	412.5	2.444	1.161	
720	7200	3600	450	2.294	1.311	
780	7800	3900	487.5	2.167	1.438	
840	8400	4200	525	2.009	1.596	
900	9000	4500	562.5	1.829	1.776	
912	9120	4560	570	—	—	COLLAPSE

CERTIFIED CYPRUS

TRAY DESCRIPTION: GF-10JL-12
18" WIDE SOLID C R. TROF W/CONN at MID SPAN
14GA — 50 KSI — 4 x 1¼" SIDES

TEST NO.
12-22-77
DATE
BB
BY

HORIZONTAL TEST



No. of Bundles	Total Weight Lbs.	Total Weight Lbs/2	Lbs/Ft	Indicator Reading	Δ"	Comments
0	0	0	0	3.720	0	
60	600	300	37.5	3.619	.101	
120	1200	600	75	3.533	.187	
180	1800	900	112.5	3.445	.275	
240	2400	1200	150	3.351	.369	
300	3000	1500	187.5	3.242	.478	
360	3600	1800	225	3.137	.583	
420	4200	2100	262.5	3.011	.709	
480	4800	2400	300	2.905	.815	
540	5400	2700	337.5	2.775	.945	
600	6000	3000	375	2.671	1.049	
660	6600	3300	412.5	2.539	1.181	
720	7200	3600	450	2.386	1.334	
780	7800	3900	487.5	2.257	1.463	
840	8400	4200	525	2.083	1.637	
875	8750	4375	547	—	—	COLLAPSE

CERTIFIED CYPRUS

TRAY DESCRIPTION: GF-18SL-12

18" WIDE SOLID CORR TROF - W/CONN @ MID-SPAN

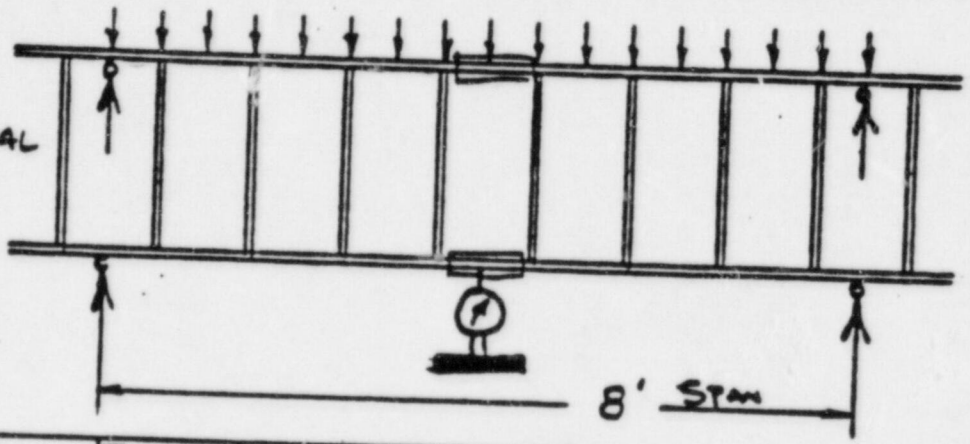
14GA 50KSI - 4x1¼ SIDERAILS

12-CP-1
TEST NO.

12-22-77
DATE

BB
BY

HORIZONTAL TEST



No. of Bundles	Total Weight Lbs.	Total Weight Lbs/2	Lbs/Ft	Indicator Reading	Δ"	Comments
0	0	0	0	4.016	0	
60	600	300	37.5	3.918	.098	
120	1200	600	75	3.832	.184	
180	1800	900	112.5	3.744	.272	
240	2400	1200	150	3.649	.367	
300	3000	1500	187.5	3.540	.476	
360	3600	1800	225	3.438	.578	
420	4200	2100	262.5	3.312	.704	
480	4800	2400	300	3.208	.808	
540	5400	2700	337.5	3.080	.936	
600	6000	3000	375	2.974	1.042	
660	6600	3300	412.5	2.847	1.169	
720	7200	3600	450	2.695	1.321	
780	7800	3900	487.5	2.569	1.447	
840	8400	4200	525	2.417	1.599	
900	9000	4500	562.5	2.232	1.784	
904	9040	4520	565	—	—	COLLAPSE

CERTIFIED CYPRUS

CALCULATIONS FOR CYPRUS CABLE TRAY CAT NO. GF-12SL
TO COMANCHE PEAK S.E.S. SPECIFICATION # 2323-ES-19

SPECIFICATION LOADING IS 35 LBS/FT.

4 x 1¼ x 14GA.
$I_x = 2.46$ Δ

FROM ATTACHED TEST DATA:

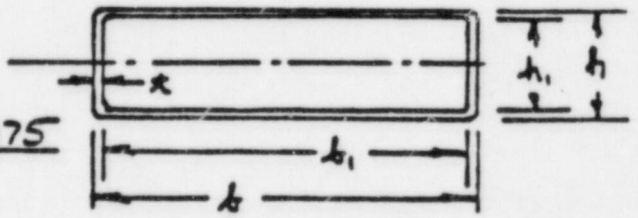
$F_m = 620 + 603 + 607 \div 3 = 610$

$F_x = 598 + 570 + 558 \div 3 = 575$

USING EULERS COLUMN FORMULA: $P = \frac{\pi^2 I E}{l^2}$

WHERE: P = ALLOWABLE LOAD LBS
E = 29.5×10^6
l = 96"
$I = \frac{1}{12}[b h^3 - b_1 h_1^3]$
b = 4.0 h = 2.5 t = .075
I = 1.045

1.149
1.161 } 1.166 >
1.187 } Δ



$F_l = \frac{P}{8} = \frac{\pi^2 \times 1.045 \times 29.5 \times 10^6}{8 \times 96^2} = 4127$

THE FOLLOWING CRITERIA MUST BE MET:

$$1.6\left[\frac{f'_m}{F_m} + \sqrt{\left(\frac{f_m}{F_m}\right)^2 + \left(\frac{f_x}{F_x}\right)^2 + \left(\frac{f_l}{F_l}\right)^2}\right] \leq 1.0$$

CYPRUS

WHERE f'_m = SPECIFICATION LOADING
f_m = 2.5 x SPECIFICATION LOADING
f_x = 4.0 x SPECIFICATION LOADING
f_l = 4.0 x SPECIFICATION LOADING

$$\frac{35}{610} + \sqrt{\left(\frac{2.5 \times 35}{610}\right)^2 + \left(\frac{4.0 \times 35}{575}\right)^2 + \left(\frac{4.0 \times 35}{4127}\right)^2} \leq .625$$

$$.057 + \sqrt{.021 + .059 + .0012} \leq .625$$

$$.057 + .285 \leq .625$$

$$.342 \leq .625 \quad \therefore \text{OK}$$

TRAY DESCRIPT. NO. CF-12SL-12

12"W SOLID TROF W/CONN AT MID-SPAN (8 HOLES)

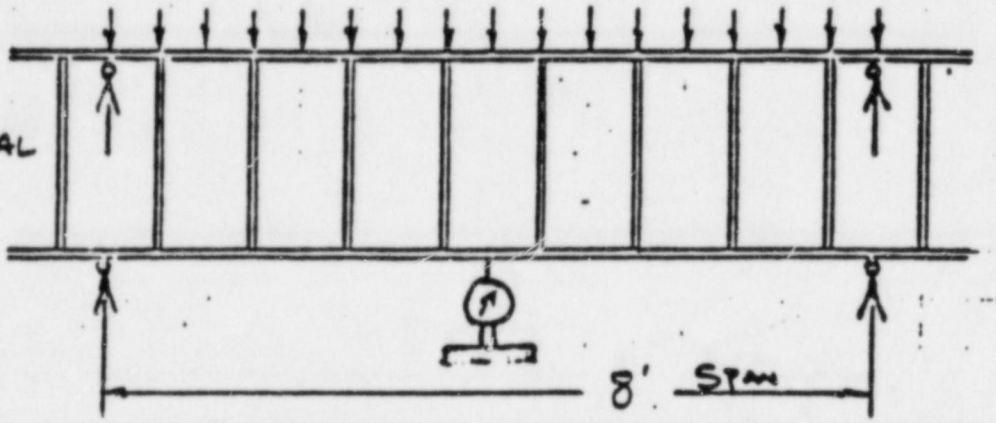
14GA-50KSI SIDEMEMBERS 4x1/4-Solid Corr. Do Horiz

TEST NO. 1 13

DATE 12-9-77

BY BB

HORIZONTAL TEST



No. of Bundles	Total Weight Lbs.	Total Weight Lbs/ft.	Lbs/Ft	Indicator Reading	Δ"	Comments
0	0	0	0	3.878	0	
60	600	300	37.5	3.784	.094	
120	1200	600	75	3.698	.180	
180	1800	900	112.5	3.609	.269	
240	2400	1200	150	3.516	.362	
300	3000	1500	187.5	3.414	.464	
360	3600	1800	225	3.320	.558	
420	4200	2100	262.5	3.202	.676	
480	4800	2400	300	3.093	.785	
540	5400	2700	337.5	2.975	.903	
600	6000	3000	375	2.867	1.011	
660	6600	3300	412.5	2.733	1.145	
720	7200	3600	450	2.598	1.280	
780	7800	3900	487.5	2.475	1.403	
840	8400	4200	525	2.340	1.538	
900	9000	4500	562.5	2.173	1.705	
956	9560	4780	598	—	—	COLLAPSE

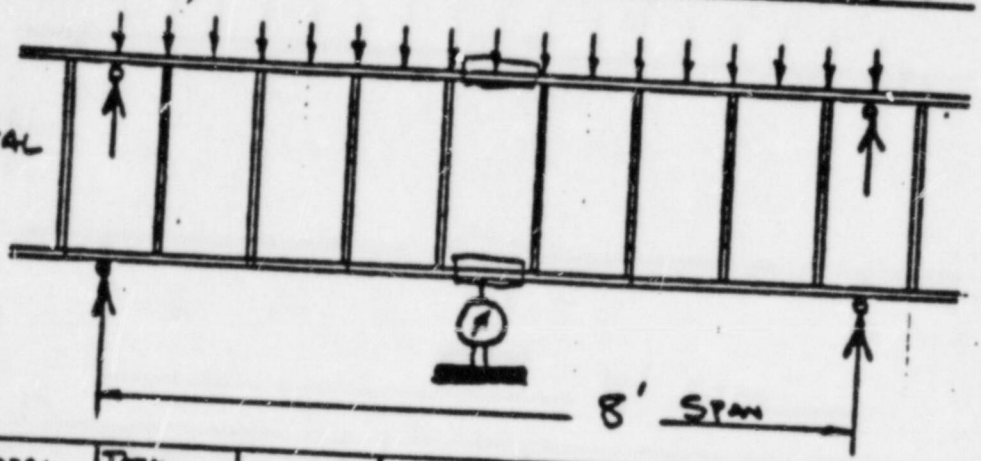
CYPRUS

TRAY DESCRIPTION: 6F-12SL-12

12"w Solid trof w/conn. at MIDSPAN 8-HOLES
146A-50 KSI SIDERAILS 4x1¼" - Solid Corr. Bottom

12-CP-
TRAY
12-20
DATE
7-13

HORIZONTAL TEST



No. of Bundles	Total Weight Lbs.	Total Weight Lbs/2	Lbs/Ft	Indicator Reading	Δ"	Comments
0	0	0	0	4.058	0	
60	600	300	37.5	3.959	.099	
120	1200	600	75	3.875	.183	
180	1800	900	112.5	3.784	.274	
240	2400	1200	150	3.690	.368	
300	3000	1500	187.5	3.586	.472	
360	3600	1800	225	3.489	.569	
420	4200	2100	262.5	3.368	.690	
480	4800	2400	300	3.260	.798	
540	5400	2700	337.5	3.134	.924	
600	6000	3000	375	3.031	1.027	
660	6600	3300	412.5	2.902	1.156	
720	7200	3600	450	2.753	1.305	
780	7800	3900	487.5	2.625	1.433	
840	8400	4200	525	2.466	1.592	
900	9000	4500	562.5	2.287	1.771	
912	9120	4560	570	—	—	COLLAPSE

CERTIFIED CYPRUS

MY DESCRIPTION NO. GF-12SL-12

12"W SOLID TROF W/CONN AT THIRD-SPAN (8 HOLES)

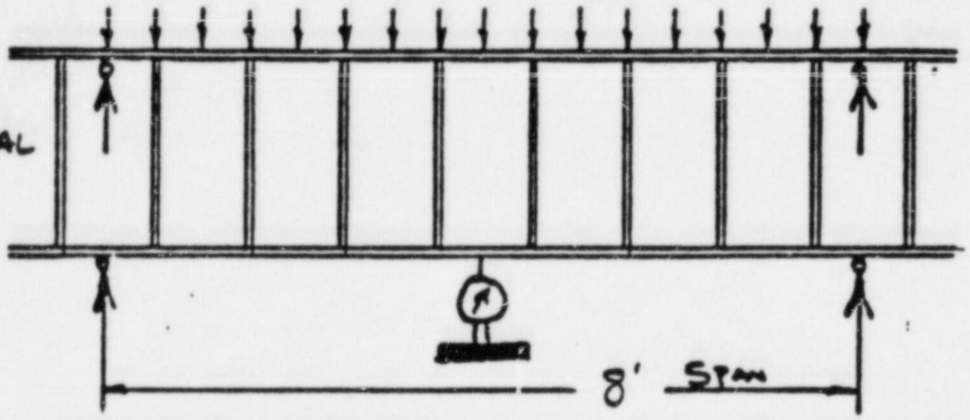
M 6A-50KSI SIDEMEMBERS 4x1/4-Solid Corr. Bottom

TEST NO. 12-20-77

DATE

BB

BY



No. OF BUNDLES	TOTAL WEIGHT LBS.	TOTAL WEIGHT LBS/2	LBS/FT	INDICATOR READING	Δ"	COMMENTS
0	0	0	0	4.420	0	
60	600	300	37.5	4.317	.103	
120	1200	600	75	4.234	.186	
180	1800	900	112.5	4.141	.279	
240	2400	1200	150	4.046	.374	
300	3000	1500	187.5	3.939	.481	
360	3600	1800	225	3.843	.577	
420	4200	2100	262.5	3.721	.699	
480	4800	2400	300	3.609	.811	
540	5400	2700	337.5	3.485	.935	
600	6000	3000	375	3.357	1.063	
660	6600	3300	412.5	3.232	1.188	
720	7200	3600	450	3.089	1.331	
780	7800	3900	487.5	2.951	1.469	
840	8400	4200	525	2.788	1.632	
892	8920	4460	558	—	—	COLLAPSE

CERTIFIED CYPRUS

CALCULATIONS FOR CYPRUS CABLE TRAY CAT NO. GF-06SL
TO COMANCHE PEAK S.E.S. SPECIFICATION # 2323-ES-19

4 x 13/16 x 16 GA
I_x = 1.56
Δ

SPECIFICATION LOADING IS 18 LBS/FT.

FROM ATTACHED TEST DATA:

F_m = 170 + 187 + 192 ÷ 3 = 183

F_x = 253 + 246 + 251 ÷ 3 = 250

USING EULERS COLUMN FORMULA: $P = \frac{\pi^2 I E}{l^2}$

WHERE: P = ALLOWABLE LOAD LBS

E = 29.5×10^6

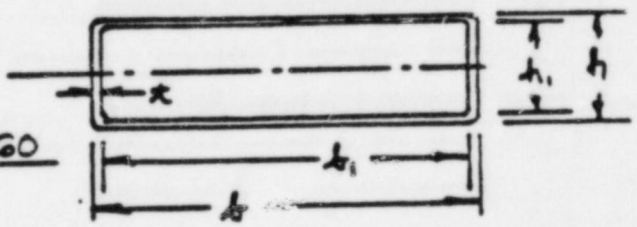
l = 96"

$I = \frac{1}{12}[b h^3 - b_1 h_1^3]$

b = 4.0 h = .625 t = .060

I = .328

.548
.548
.570
} .555
Δ



$F_l = \frac{P}{8} = \frac{\pi^2 \times .328 \times 29.5 \times 10^6}{8 \times 96^2} = 1295$

THE FOLLOWING CRITERIA MUST BE MET:

$$1.6\left[\frac{f'_m}{F_m} + \sqrt{\left(\frac{f_m}{F_m}\right)^2 + \left(\frac{f_x}{F_x}\right)^2 + \left(\frac{f_l}{F_l}\right)^2}\right] \leq 1.0$$

CYPRUS

WHERE f'_m = SPECIFICATION LOADING
f_m = 2.5 x SPECIFICATION LOADING
f_x = 4.0 x SPECIFICATION LOADING
f_l = 4.0 x SPECIFICATION LOADING

$$\frac{18}{183} + \sqrt{\left(\frac{2.5 \times 18}{183}\right)^2 + \left(\frac{4.0 \times 18}{250}\right)^2 + \left(\frac{4.0 \times 18}{1295}\right)^2} \leq .625$$

$$.098 + \sqrt{.06 + .08 + .003} \leq .625$$

$$.098 + .378 \leq .625$$

$$.476 \leq .625 \therefore \text{OK}$$

TRAY DESCRIPTION: GF-06SL-12

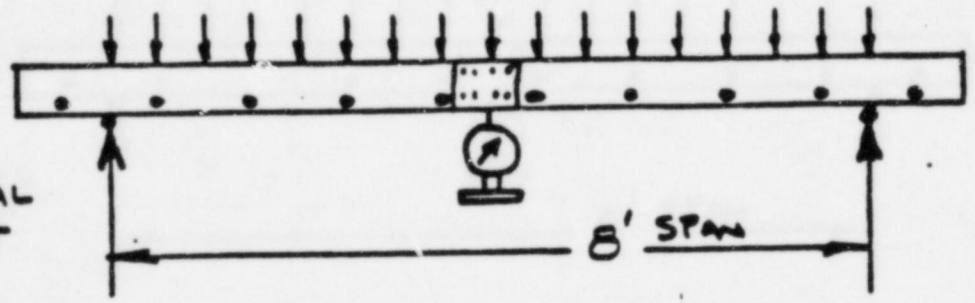
6"W. Solid Troof W/ CONN. AT MID-SPAN 8-HOLES

16 GA. SIDE MEMBERS 4x13/16 W/ SOLID CORR. BOTTOM

12-CP-01
TEST NO.

12-5-77
DATE

B.B.
BY



NO. OF BUNDLES	TOTAL WEIGHT LBS.	LBS/FT.	#1 INDICATOR READING	#1 INDICATOR Δ"	#2 INDICATOR READING	#2 INDICATOR Δ"	MAX AVERAGE Δ"	COMMENTS
0	0	0	3.715	0	4.144	0	0	
12	120	15	3.662	.053	4.101	.043	.048	
24	240	30	3.601	.114	4.044	.100	.107	
36	360	45	3.543	.172	3.988	.156	.164	
64	640	80	3.412	.303	3.865	.279	.291	
98	980	122.5	3.241	.474	3.698	.446	.460	
136	1360	170	2.995	.720	3.458	.686	.703	
150	1500	187	—	—	—	—	—	COLLAPSE

CYPRUS

TRAY DESCRIPTION: COMMANCHE PEAK

GF-06SL-12

6" WIDE SOLID TROF w/ CONNECTORS AT MID-SPAN (8 HOLES)

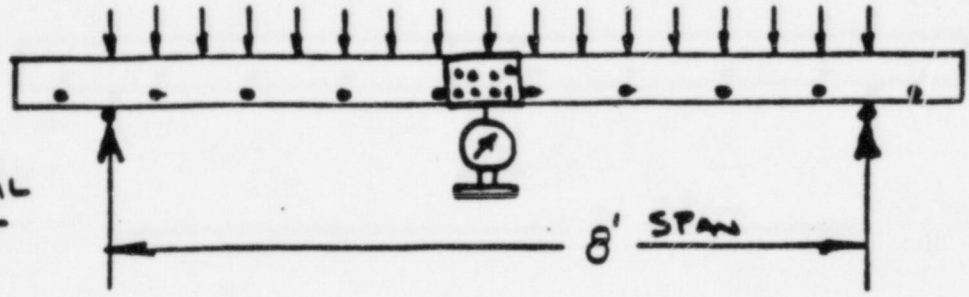
16 GA. SIDE MEMBERS 4" x 13/16" w/ SOLID CORRUGATED BOTTOM

12-CP-0
TEST NO

12-6-77
DATE

B. BROWN
BY

VERTICAL TEST



NO. OF BUNDLES	TOTAL WEIGHT LBS.	LBS/FT.	#1 INDICATOR READING	#1 INDICATOR Δ"	#2 INDICATOR READING	#2 INDICATOR Δ"	MAX AVERAGE Δ"	COMMENTS
0	0	0	4.825	0	4.800	0	0	
12	120	15	4.766	.059	4.755	.045	.052	
24	240	30	4.692	.133	4.691	.109	.121	
36	360	45	4.629	.196	4.628	.172	.184	
64	640	80	4.498	.327	4.496	.304	.316	
98	980	122.5	4.318	.507	4.336	.464	.486	
136	1360	170	—	—	—	—	—	COLLAPSE

CERTIFIED CYPRUS

TRAY DESCRIPTION: GF-06SL-12

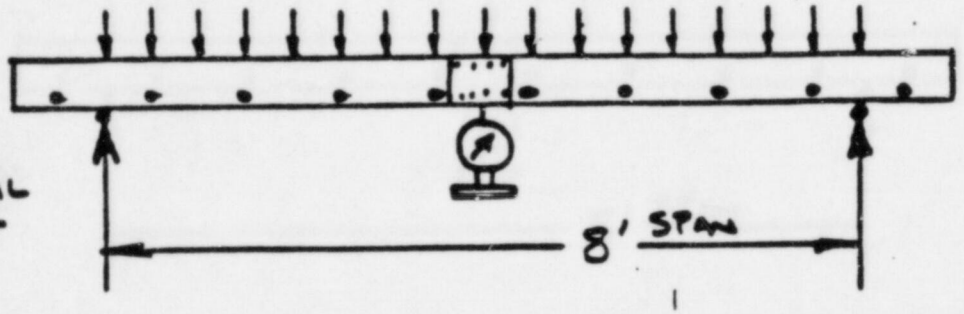
6" W. Solid Trof W/CONN. AT MID-SPAN (8-HOLES)

166A. SIDE MEMBERS 4x1 7/16 W/ SOLID CORE BOTTOM

12-CP-0[illegible]
TEST NO.

12-6-7[illegible]
DATE

DB.
BY



NO. OF BUNDLES	TOTAL WEIGHT LBS.	LBS/FT.	#1 INDICATOR READING	#1 INDICATOR Δ"	#2 INDICATOR READING	#2 INDICATOR Δ"	MAX AVERAGE Δ"	COMMENTS
0	0	0	4.420	0	4.385	0	0	
12	120	15	4.367	.053	4.341	.044	.049	
24	240	30	4.310	.110	4.285	.100	.105	
36	360	45	4.252	.168	4.233	.152	.160	
64	640	80	4.124	.296	4.107	.278	.287	
98	980	122.5	3.955	.465	3.943	.442	.454	
136	1360	170	3.734	.686	3729	.656	.671	
154	1540	192	—	—	—	—	—	COLLAPSE

CYPRUS

TRAY DESCRIPTION: GF-06SL-12

12" W SOLID TROF W/CONN. AT MID-SPAN — 8-HOLES

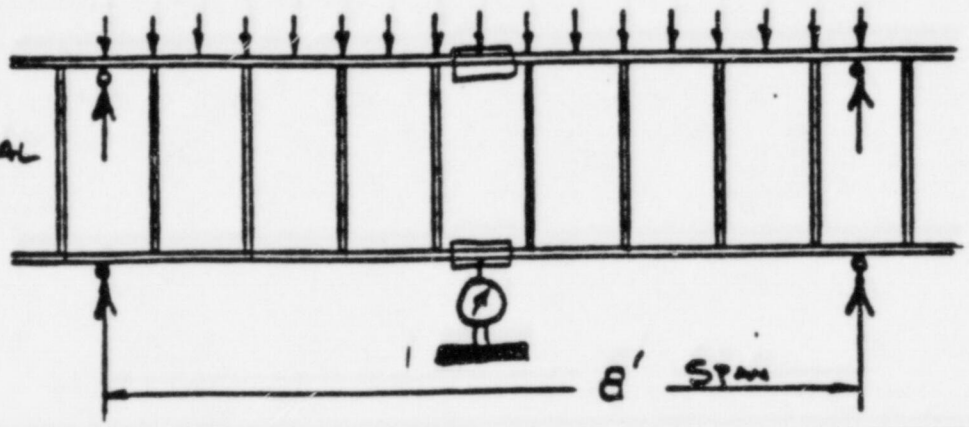
6GA. SIDE MEMBERS (4 x 13/16) W/SOLID CORR BOTTOM

12-CP-(illegible)
TEST NO.

12-14-(illegible)
DATE

B.B.
BY

HORIZONTAL TEST



No. of Bundles	Total Weight Lbs.	Total Weight Lbs/2	Lbs/Ft	Indicator Reading	Δ"	Comments
0	0	0	0	4.428	0	
—	—	36	*4.5	4.411	.018	*BRASS BAR ON SIDE-RAIL
32	320	160	24.5	4.310	.119	LBS/FT INCL. 4.5 LBS/FT
64	640	320	44.5	4.220	.209	
96	960	480	64.5	4.136	.293	
128	1280	640	84.5	4.048	.381	
158	1580	790	103	3.947	.482	
188	1880	940	122	3.839	.590	
203	2030	1015	131	3.784	.645	
218	2180	1090	141	3.715	.714	
233	2330	1165	150	3.631	.798	
248	2480	1240	159.5	3.548	.881	
263	2630	1315	169	3.491	.938	
278	2780	1390	178	5.373	1.056	
293	2930	1465	188	3.238	1.191	
308	3080	1540	197	3.136	1.293	
323	3230	1615	206.5	3.055	1.374	
338	3380	1690	216	2.940	1.489	
353	3530	1765	225	2.846	1.583	
368	3680	1840	234.5	2.737	1.692	
383	3830	1915	244	2.630	1.799	
386	3860	1930	246	—	—	COLLAPSE
—	—	—	—	—	—	

CYPRUS

TRAY DESCRIPTION: GF-06SL-12

6"W. SOLID TROF W/ CONN. AT MID-SPAN (8-HOLES)

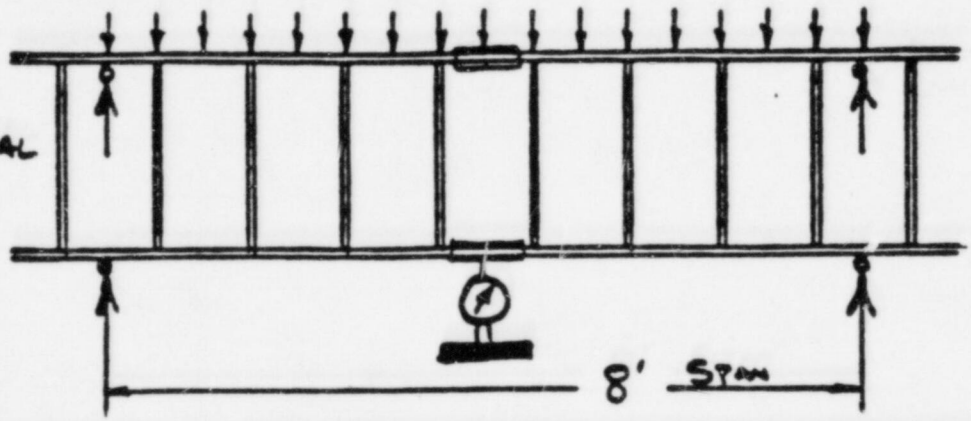
6 GA. SIDE MEMBERS 4"x 13/16" W. SOLID CORRUGATED BOTTOM

12-CP-1
TEST NO.

12-14-91
DATE

B.B.
BY

HORIZONTAL TEST

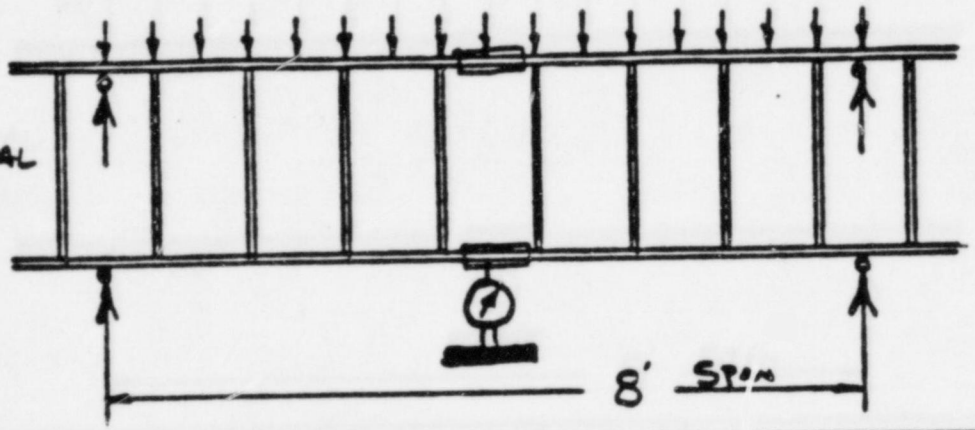


No. of Bundles	Total Weight Lbs.	Total Weight Lbs/2	Lbs/Ft	Indicator Reading	Δ"	Comments
0	0	0	0	4.150	0	
—	—	36	*4.5	4.133	.017	*BRASS BAR ON SIDE-RAIL
32	320	160	24.5	4.033	.117	LB/FT INCLUDES 4.5 LBS/FT
64	640	320	44.5	3.942	.208	TO ACCOUNT FOR BRASS BAR.
96	960	480	64.5	3.856	.294	
128	1280	640	84.5	3.772	.378	
158	1580	790	103	3.670	.480	
188	1880	940	122	3.561	.589	
203	2030	1015	131	3.508	.642	
218	2180	1090	141	3.433	.717	
233	2330	1165	150	3.362	.788	
248	2480	1240	159.5	3.277	.873	
263	2630	1315	169	3.215	.935	
278	2780	1390	178	3.129	1.021	
293	2930	1465	188	2.972	1.178	
308	3080	1540	197	2.901	1.249	
323	3230	1615	2065	2.805	1.345	
338	3380	1690	216	2.674	1.476	
353	3530	1765	225	2.640	1.510	
368	3680	1840	234.5	2.518	1.632	
383	3830	1915	244	2.420	1.730	
394	3940	1970	251	—	—	Collapse

CERTIFIED CYPRUS

TRAY DESCRIPTION: GF-06SL-12
6"W SOLID TROF W/CONN. AT MID-SPAN (8-HOLES)
66A. SIDE MEMBERS 4X13/16 W/ SOLID CORR BOTTOM

12-5
TEST NO
12-16-77
DATE
B. BROWN
BY



HORIZONTAL TEST

No. of Bundles	Total Weight Lbs.	Total Weight Lbs/2	Lbs/Ft	Indicator Reading	Δ"	Comments
0	0	0	0	4.568	0	
—	—	36	*4.5	4.551	.017	*BRASS BAR ON SIDE-RAIL
32	320	160	24.5	4.453	.115	LBS/FT INCLUDES 4.5 LBS/FT
64	640	320	44.5	4.364	.204	TO ACCOUNT FOR BRASS BAR
96	960	480	64.5	4.279	.289	
128	1280	640	84.5	4.192	.376	
158	1580	790	103	4.095	.473	
188	1880	940	122	3.986	.582	
203	2030	1015	131	3.934	.634	
218	2180	1090	141	3.861	.707	
233	2330	1165	150	3.791	.777	
248	2480	1240	159.5	3.709	.859	
263	2630	1315	169	3.646	.922	
278	2780	1390	178	3.580	.988	
290	2900	1450	186	3.453	1.115	
293	2930	1465	188	3.430	1.138	
308	3080	1540	197	3.347	1.221	
323	3230	1615	206.5	3.270	1.298	
338	3380	1690	216	3.132	1.436	
353	3530	1765	225	3.107	1.461	HIT SAFETY STOP - STOP LOWERED
368	3680	1840	234.5		1.580	
383	3830	1915	244	—	1.698	
398	3980	1990	253	—	—	COLLAPSE

CYPRUS

CALCULATION/PROBLEM COVER SHEET

Calculation/Problem No: M-04

Title: BASE ANGLE STIFFNESS

Client: TUGCO/CPSES Project: CABLE TRAY EVAL.

Job No: 0210-040

Design Input/References:

SEE SECTION 9.0 REFERENCES

Assumptions:

SEE SECTION 1.0 DESCRIPTION OF ANALYSIS

Method:

SEE SECTION 1.0 DESCRIPTION OF ANALYSIS

UNCONTROLLED COPY

Remarks:

1. SEE SECTION 8.0 DISCUSSION OF RESULTS/RECOMMENDATIONS
2. RESULTS OF THIS CALCULATION AFFECT IMPELL REPORT # 01-0210-1470, IMPELL CALCULATIONS B-01 AND B-02 (JOB NO. 0210-041).

REV. NO.	REVISION	APPROVED	DATE
[illegible]	ORIGINAL ISSUE	Calvin Yee Wong	1/31/86
[illegible]	REVISED TO INCORPORATE BASEPLATE II REANALYSIS.	Calvin Yee Wong	3/21/86

EDS 0008 5/78

TABLE OF CONTENTS:

		PAGES
1.0	DESCRIPTION OF ANALYSIS	10
2.0	BACK-UP CALCULATIONS	23
3.0	SUMMARY OF RESULTS/RECOMMENDATIONS	3
4.0	REFERENCES	2
	TOTAL:	38

APPENDIX A : SUPERCEDED PAGES

NOTE: THE FOLLOWING PAGES HAVE BEEN ADDED IN REV 1:

SEC. 1.0	PG'S 8-10 OF 10
SEC. 2.0	PG'S 16-23 OF 23
SEC 3.0	PG. 3 OF 3
SEC 4.0	PG 2 OF 2

THE FOLLOWING PAGES HAVE BEEN REVISED OR REPLACED IN REV 1.

SEC. 3.0	PG. 1 → OF 3	(REPLACED)
SEC 4.0	PG 1 OF 2	(REVISED)
SEC 2.0	PG. 4-7, 12, 13, OF 23	(REPLACED)
	PG 11 OF 23	(REVISED)
SEC. 3.0	P. 1 OF 3	(REVISED)

REV	BY	DATE	CHECKED	DATE	IMPELL CORPORATION	JOB NO 0210-040	PAGE 1
1	BD	3-10-86	JRB	5-10-86		CALC NO M-04	OF 1
0	ELC	12-6-85	JRB	12-19-85			

1.0 DESCRIPTION OF ANALYSIS:

THE PURPOSE OF THIS CALCULATION IS TO DETERMINE THE RANGE OF STIFFNESS VALUES ASSOCIATED WITH BASE ANGLES. FOUR COMMONLY USED BASE ANGLE SIZES WERE EVALUATED:

1. L 6x6x 3/4
2. L 6x6x 3/8
3. L 6x4x 3/4
5. L 5x5x 3/4

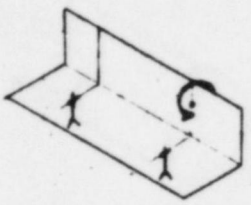
BOTH 2-BOLT AND 1-BOLT BASE ANGLE CONFIGURATIONS WERE EVALUATED. SINCE THE SUPPORT IS MODELED TO THE FACE OF THE FIXED LEG OF THE ANGLE, IT WAS DETERMINED THAT ONLY TWO OF THE SIX STIFFNESSES REQUIRED EVALUATION: (1) THE BENDING STIFFNESS ABOUT THE LONGITUDINAL AXIS OF THE ANGLE, AND (2) THE DIRECT PULLOUT STIFFNESS OF THE ANGLE. THE OTHER TWO TRANSLATIONAL STIFFNESSES AND TWO ROTATIONAL STIFFNESS ARE JUDGED TO BE RIGID, WITH EXCEPTION TO THE ROTATION ABOUT THE BOLT AXIS FOR THE 1-BOLT WHICH IS FREE.

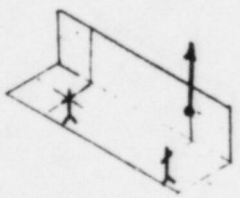
THE PROGRAM BASEPLATE-II WAS USED TO EVALUATE THE BASE ANGLES. NINE LOAD CASES WERE RUN FOR EACH 2-BOLT ANGLE, AND FIVE CASES FOR EACH 1-BOLT. A DESCRIPTION OF EACH OF THE CASES ANALYZED FOLLOWS.

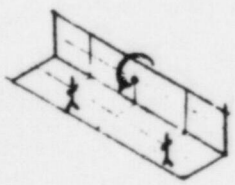
2-BOLT MODEL:

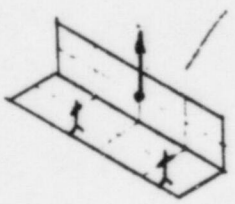
THE 2-BOLT MODELS FOR THE BASE ANGLES ARE ALL SIMILAR. THE BASE ANGLE LENGTH AND BOLT SPACINGS WERE CONSIDERED WITH VARYING DIMENSIONS THAT WERE COMMONLY USED. THE MOST COMMONLY USED ANCHOR BOLT IS THE 1¼" HILTI SUPER KWIK. THESE CONFIGURATIONS ARE REPRESENTATIVE OF THE BASE ANGLES REVIEWED TO DATE.

NINE LOAD CASES WERE EVALUATED FOR EACH OF THE FOUR BASE ANGLE SIZES:

1.  MOMENT LOADING ON FREE LEG AT BOLT LOCATION.

2.  PULL-OUT LOAD ON FREE LEG AT BOLT LOCATION.

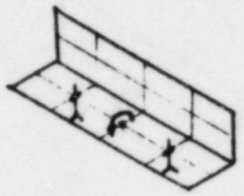
3.  MOMENT LOADING ON FREE LEG MIDWAY BETWEEN BOLTS

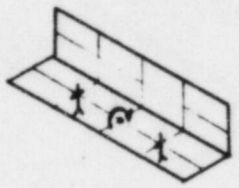
4.  PULL-OUT LOAD ON FREE LEG MIDWAY BETWEEN BOLTS

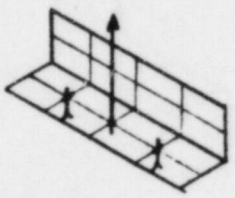
REV	BY	DATE	CHECKED	DATE
1	[illegible]	9-10-86	[illegible]	3-11-86
0	ELC	12-4-85	[illegible]	12-4-85

IMPELL CORPORATION

JOB NO 0210-040
CALC NO M-04
PAGE 2 OF 10

5.  MOMENT LOADING ON FIXED LEG MIDWAY BETWEEN BOLTS

6.  MOMENT LOADING ON FIXED LEG MIDWAY BETWEEN BOLTS

7.  PULL-OUT LOAD ON FIXED LEG MIDWAY BETWEEN BOLTS

THE ROTATION STIFFNESS CAN BE DETERMINED BY REVIEWING THE DISPLACEMENTS OF THE NODES ADJACENT TO LOADING: ie

L 6"x6"x 3/4" CASE #1 LOADING = 1000 IN·LB

$$\left.\begin{aligned}\Delta Z_{21} &= .002049" \\ \Delta Z_{22} &= .001328"\end{aligned}\right\} \Delta = .000721"$$

$$\angle = \sin^{-1}\frac{.000721"}{1.5"} = .02754^\circ \text{ OR } .000481 \text{ RAD}$$

$$K = \frac{1000 \text{ IN·LB}}{.000481 \text{ RAD}} = 2080 \frac{\text{KP·IN}}{\text{RAD}}$$

REV	BY	DATE	CHECKED	DATE
0	ELC	12-4-85	RR	12-4-85

IMPELL CORPORATION

JOB NO 0210-040 | PAGE 3 OF 10

CALC NO M-04

THE TRANSLATIONAL STIFFNESS FOR PULL-OUT LOADING CAN BE DETERMINED DIRECTLY BY REVIEWING THE DISPLACEMENT AT THE LOAD POINT: IE

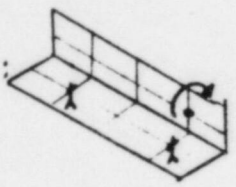
L 6"x6"x 3/4" CASE #2 LOADING = 1000 LB

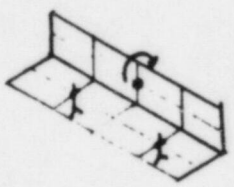
$$\Delta z_{21} = .011590"$$

$$K = \frac{1000\ LB}{.011590\ IN} = 86\ \frac{KIP}{IN}$$

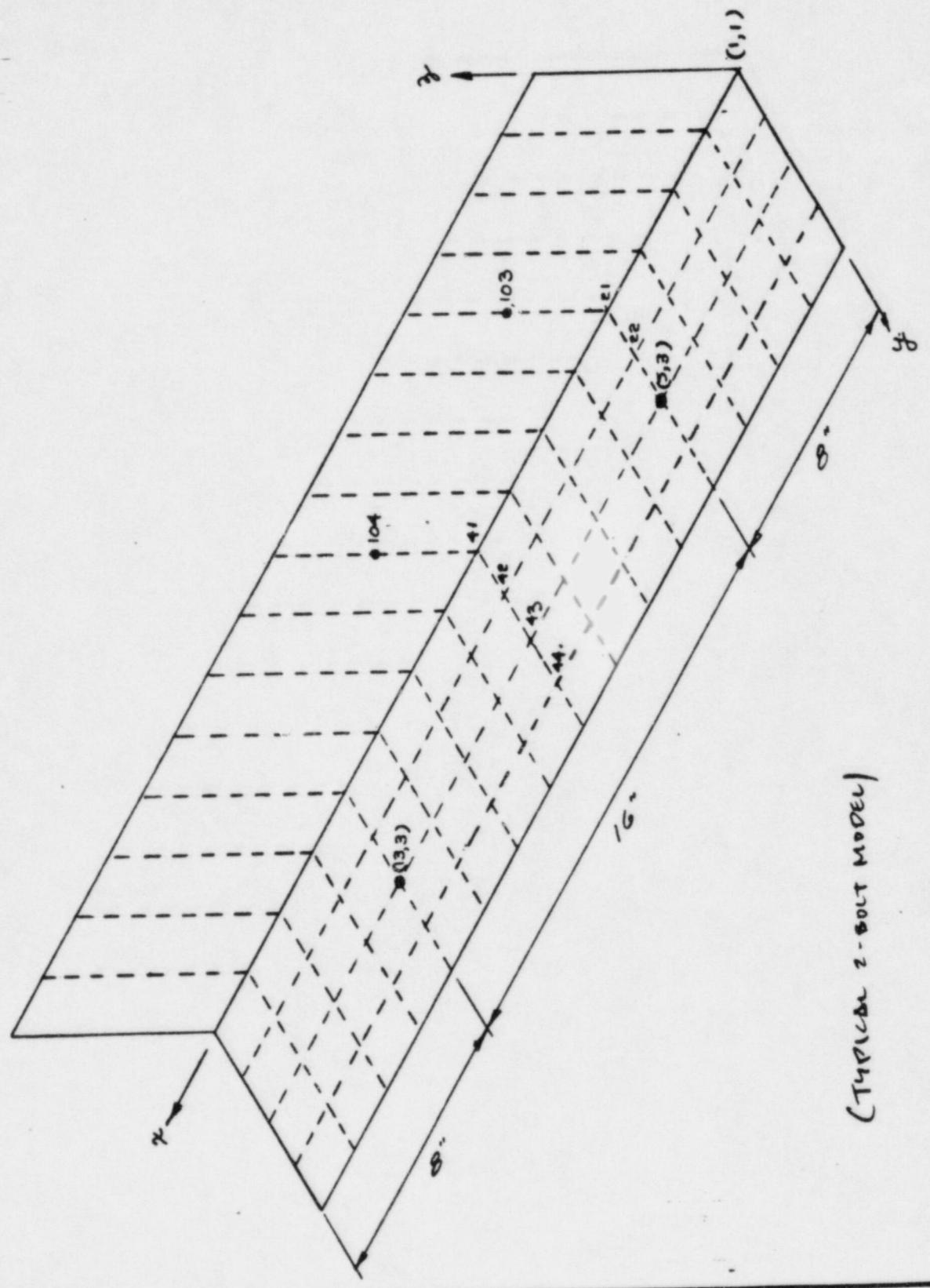
A DETAILED SKETCH OF THE BASEPLATE II MODEL FOLLOWS ON THE NEXT PAGE.

TWO ADDITIONAL LOAD CASES WERE CONSIDERED TO EVALUATE A POSITIVE X-X MOMENT FOR THE 2-BOLT BASE ANGLES AS FOLLOWS:

CASE 8:  MOMENT LOADING ON FREE LEG AT BOLT LOCATION

CASE 9:  MOMENT LOADING ON FREE LEG MIDWAY BETWEEN BOLTS.

						JOB NO 0210-040	PAGE 4
0	BL	10-1-85	VJB	12-4-85	IMPELL CORPORATION	CALC NO M-04	OF 10
REV	BY	DATE	CHECKED	DATE			



(TYPICAL 2-BOLT MODEL)

REV	BY	DATE	CHECKED	DATE
1	BP	3-10-86	VJS	3-11-86
0	ELC	12-4-85	VJS	12-4-85

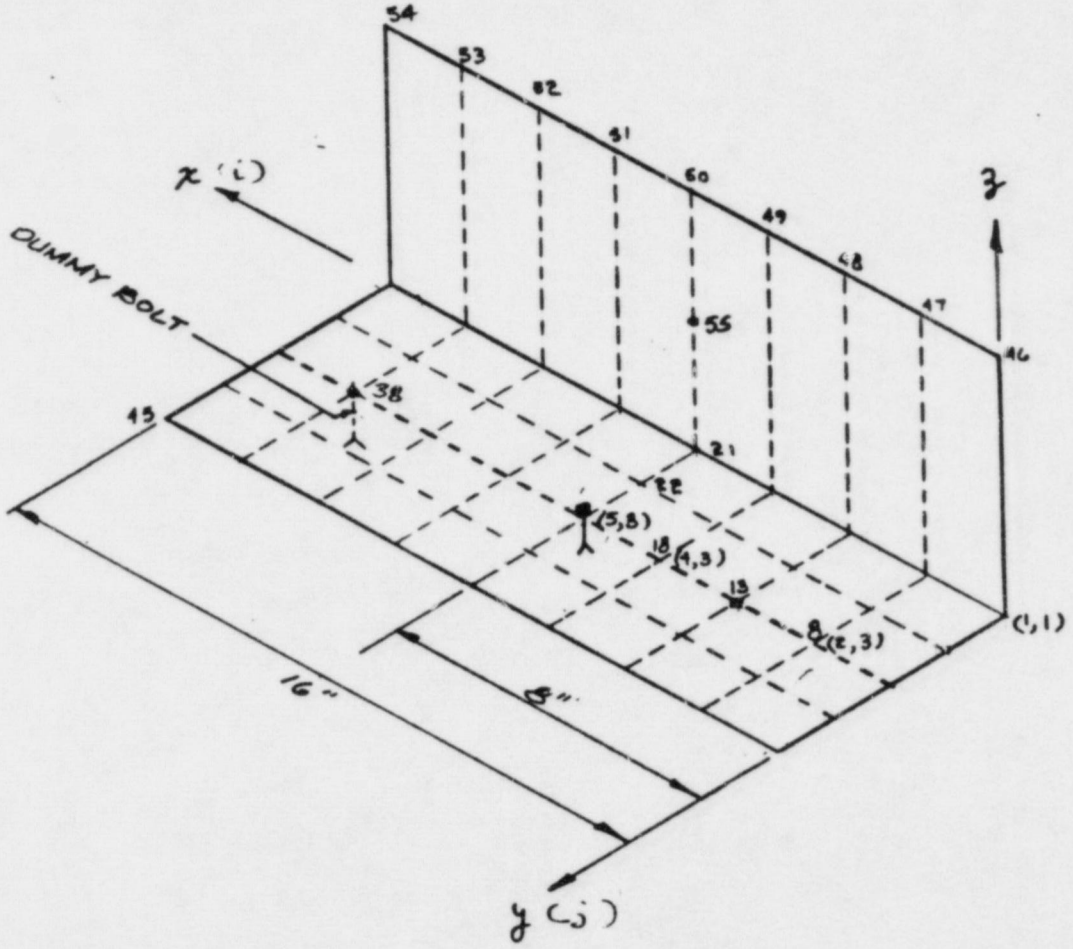
IMPELL CORPORATION

JOB NO 0210-040

CALC NO M-04

1-BOLT MODEL (WITH 1-DUMMY BOLT @ NODE 38)

THE 1-BOLT MODELS FOR THE BASE ANGLES ARE ALL SIMILAR. A 16-IN. LONG ANGLE WAS CONSIDERED WITH THE BOLT LOCATED IN THE MIDDLE (AS SHOWN BELOW). LIKE THE TWO-BOLT MODELS, THE MOST COMMONLY USED ANCHOR BOLT IS THE 1 1/4" SUPER HILTI KWIK.



REV	BY	DATE	CHECKED	DATE
0	GLC	12-4-85	VRB	12-4-85

IMPELL CORPORATION

JOB NO 0210-040

CALC NO M-04

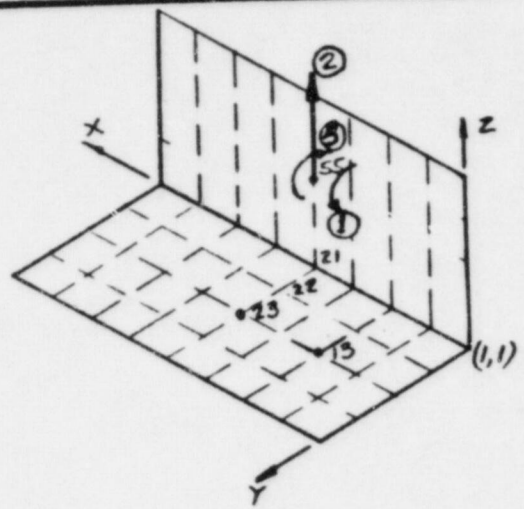
REV	BY	DATE	CHECKED	DATE
0	RB	11-24-88	ELL	12-4-88

IMPELL CORPORATION

JOB NO 0210-040

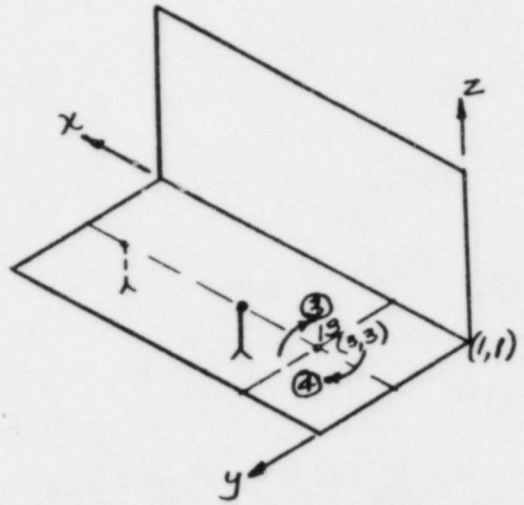
CALC NO M-04

PAGE 7 OF 10



LOADCASES AT NODE 55

* LOAD CASE (1) - APPLY NEGATIVE MOMENT X EQUAL TO -1000 IN-#
* LOAD CASE (2) - APPLY POSITIVE FORCE Z=1000#
* LOAD CASE (5) - APPLY (+) MOMENT X=1000 IN-#.



LOADCASES AT NODE 13

* LOAD CASE (3) - APPLY (+) MOMENT X=1000 IN-#
* LOAD CASE (4) - APPLY (-) MOMENT X = 1000 IN-#

FIVE LOAD CASES ARE EVALUATED FOR EACH OF THE FOUR BASE ANGLE SIZES AS SHOWN ABOVE.

1.0

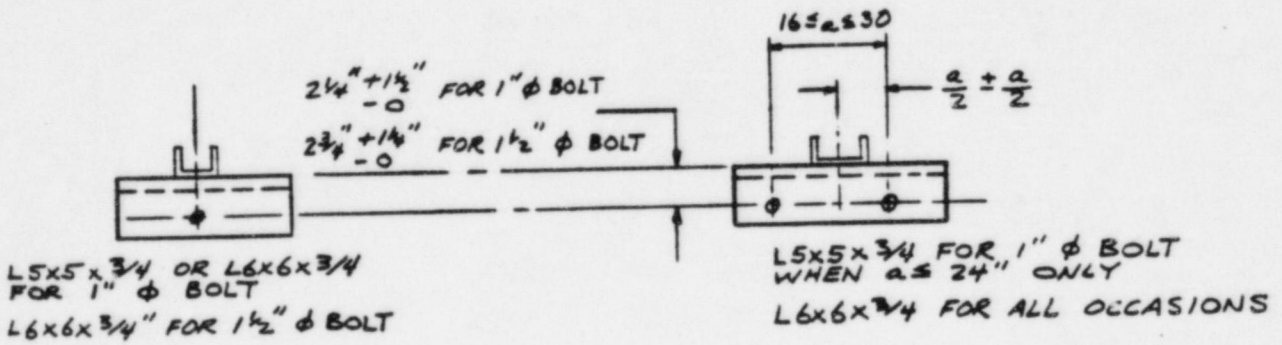
DESCRIPTION OF REV 1 ANALYSIS

THE PURPOSE OF REVISION 1 WAS TO INCORPORATE THE FOLLOWING:

- BASE ANGLE CONFIGURATIONS NOT COVERED IN REV. 0
- UPDATE f'_c = 4000 PSI
- UPDATE STIFFNESS OF HILTI SUPER KWIK BOLT, K_T = 461 K/IN (REF. F)
  K_V = 111 K/IN (REF. F)

THE BASE ANGLES REPRESENTING LOWER BOUND STIFFNESSES WERE ANALYZED. IN ADDITION, THE UPPER BOUND STIFFNESSES FOR TYPICAL BASE ANGLE CONFIGURATIONS WERE ALSO ANALYZED. FROM THE RANGE OF STIFFNESSES DETERMINED, A REPRESENTATIVE OR "AVERAGE" STIFFNESS IS CALCULATED. ALL ANALYSES WERE PERFORMED USING THE CONCRETE AND BOLT STIFFNESS PROPERTIES SPECIFIED ABOVE.

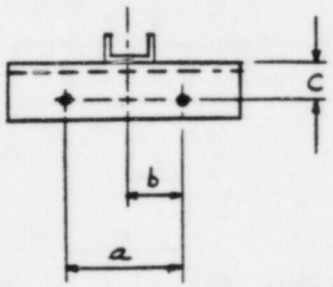
REVIEW OF ACCEPTABLE BASE ANGLE CONFIGURATIONS GIVEN IN REFERENCE A SHOWED THE FOLLOWING DIMENSIONAL TOLERANCES FOR BASE ANGLE CONFIGURATIONS.



REV	BY	DATE	CHECKED	DATE		JOB NO	PAGE
1	DC	3-4-86	AD	3-7-86	IMPELL CORPORATION	0210-040	8 OF 10
						CALC NO M-04	

1.0 DESCRIPTION OF REV 1 ANALYSIS cont'd

FROM THE PRECEDING SKETCHES IT WAS DETERMINED THAT THE FOLLOWING CASES SHOULD ALSO BE CONSIDERED.



BASE ANGLE CONFIGURATION CASE	DIMENSIONS (INCH) a	b	c	ANGLE SIZE	COMMENTS
A	24	12	2 1/4	L5x5x3/4	STIFFNESS > CASE B *
/ B	24	12	3 3/4	L5x5x3/4	
C	24	0	2 1/4	L5x5x3/4	STIFFNESS > CASE D *
/ D	24	0	3 3/4	L5x5x3/4	
E	30	15	2 3/4	L6x6x3/4	STIFFNESS > CASE F *
/ F	30	15	4	L6x6x3/4	
G	30	0	2 3/4	L6x6x3/4	STIFFNESS > CASE H *
/ H	30	0	4	L6x6x3/4	

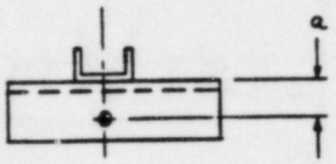
* WHEN 1 CASE HAD STIFFNESS > ANOTHER CASE, IT WAS UNNECESSARY TO RUN THE BASEPLATE II PROGRAM FOR THE STIFFER CASE.

EACH BASE ANGLE RUN ON BASEPLATE II WAS ANALYZED FOR THE (9) HINE LOAD CASES DESCRIBED ON PG 2,3&4.

REV	BY	DATE	CHECKED	DATE		JOB NO 0210-040	PAGE 9 OF 10
1	DC	3-5-86	[illegible]	3-5-86	IMPELL CORPORATION	CALC NO M-04	

1.0

DESCRIPTION OF REV 1 ANALYSIS CONT'D



BASE ANGLE CONFIGURATION CASE	DIMENSION "a" (INCH)	ANGLE SIZE	COMMENTS
I	2 1/4	L5x5x3/4	STIFFNESS > CASE J *
J	3 3/4	L5x5x3/4	
K	2 3/4	L6x6x3/4	STIFFNESS > CASE H *
L	4	L6x6x3/4	

* WHEN 1 CASE HAD STIFFNESS > ANOTHER CASE, IT WAS UNNECESSAR TO RUN THE BASEPLATE II PROGRAM FOR THE STIFFER CASE.

EACH BASE ANGLE RUN ON BASEPLATE II WAS ANALYZED FOR THE (9) LINE LOAD CASES DESCRIBED ON PG 2, 3 & 4.

REV	BY	DATE	CHECKED	DATE
1	DL	8-5-86	BD	8-5-86

IMPELL CORPORATION

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2.0 BACK-UP CALCULATIONS

REV	BY	DATE	CHECKED	DATE		JOB NO	PAGE
0	ELL	12-6-85	RS	12-9-85	IMPELL CORPORATION	0210-040	1
						CALC NO H-04	OF 23

L 6"x6"x 3/4" 2-BOLT MODEL

CASE #1 : $\Delta Z_{21} = .002049"$, $\Delta Z_{22} = .001328"$ } $\Delta = .000721"$

(K_{xx} - FREE LEG) $\angle = \sin^{-1} \frac{.000721}{1.5} = .02754°$ OR $.000481$ RAD

$K = \frac{1000}{.000481} = 2080 \frac{KP.IN}{RAD}$

CASE #2 : $\Delta Z_{21} = .011590$

$K = \frac{1000}{.01159} = 86 \frac{KIP}{IN}$

(K_z - FREE LEG)

CASE #3 : $\Delta Z_{41} = .001674"$, $\Delta Z_{42} = .001087"$ } $\Delta = .000587"$

(K_{xx} - FREE LEG) $\angle = \sin^{-1} \frac{.000587"}{1.5} = .022422°$ OR $.000391°$

$K = \frac{1000}{.000391} = 2555 \frac{KP.IN}{RAD}$

CASE #4 $\Delta Z_{41} = .009356"$

$K = \frac{1000}{.009356} = 107 \frac{KIP}{IN}$

(K_z - FREE LEG)

CASE #5 $\Delta Z_{42} = .001168$, $\Delta Z_{44} = .000197$ } $.000971$

(K_{xx} - BOLTED LEG) $\angle = \sin^{-1} \frac{.000971}{3.0} = .018545°$ OR $.000324$ RAD

$K = \frac{1000}{.000324} = 3090 \frac{KP.IN}{RAD}$

REV	BY	DATE	CHECKED	DATE		JOB NO	PAGE
0	KLL	12-4-85	[illegible]	12-4-85	IMPELL CORPORATION	0210-040	2 OF 23
						CALC NO M-04	

L 6"x6"x 3/4" 2-BOLT MODEL (CONT)

CASE #6: $\Delta Z_{42} = .000425$, $\Delta Z_{44} = .001815$ } .001390

(K_{xx} - BOLTED LEG) $\angle = \sin^{-1} \frac{.001390}{3.0} = .026547°$ OR .000463 RAD

$K = \frac{1000}{.000463} = 2158 \frac{KP \cdot IN}{RAD}$

CASE #7: $\Delta Z_{43} = .003360$

$K = \frac{1000}{.003360} = 298 \frac{KIP}{IN}$

(K_z - BOLTED LEG)

CASE #8 $\Delta Z_{21} = -.000073$, $\Delta Z_{22} = .000487$ } .000510

(K_{xx} - FREE LEG) $\angle = \sin^{-1} \frac{.000510}{1.5} = .019481°$ OR .000340 RAD

$K = \frac{1000}{.000340} = 2941 \frac{KP \cdot IN}{RAD}$

CASE #9 $\Delta Z_{41} = -.000056$, $\Delta Z_{42} = .000502$ } .000558

(K_{xx} - FREE LEG) $\angle = \sin^{-1} \frac{.000558}{1.5} = .021314°$ OR .000372 RAD

$K = \frac{1000}{.000372} = 2688 \frac{KP \cdot IN}{RAD}$

REV	BY	DATE	CHECKED	DATE	IMPELL CORPORATION	JOB NO 0210-040	PAGE 3 OF 23
0	ELC	12-9-85	VJB	12-11-85		CALC NO M-04	

6 x 6 x 3/8 (24 BOLTS) (OUTPUT 86/03/18. 17.41.54)

CASE #1:

$\Delta z_{21} = .004998$
$\Delta z_{22} = .002134$ $\Big\rangle$ .002664

$\angle = \sin^{-1} \frac{.002664}{1.5} = .1093968°$ OR .00190933 RAD

$K = 1000/.00190933 = \underline{524}$ KIP-IN/RAD

CASE #2:

$\Delta z_{11} = .019203$

$K = 1000/.019203 = \underline{52}$ K/IN

CASE #3:

$\Delta z_{41} = .002967$
$\Delta z_{42} = .001021$ $\Big\rangle$ .001946

$\angle = \sin^{-1} \frac{.001946}{1.5} = .07433175°$ OR .00129733 RAD

$K = 1000/.00129733 = \underline{771}$ K-IN/RAD

CASE #4:

$\Delta z_{41} = .012719$

$K = 1000/.012719 = \underline{79}$ K/IN

CASE #5:

$\Delta z_{42} = .001941$
$\Delta z_{44} = -.000086$ $\Big\rangle$ .002027

$\angle = \sin^{-1} \frac{.002027}{3} = .03871285°$ OR .00067567

$K = 1000/.00067567 = \underline{1480}$ K-IN/RAD

CASE #6:

$\Delta z_{44} = .006827$
$\Delta z_{42} = .001093$ $\Big\rangle$ .005734

$\angle = \sin^{-1} \frac{.005734}{3.0} = .1095114°$ OR .00191133 RAD

$K = 1000/.00191133 = \underline{523}$ K-IN/RAD

REV	BY	DATE	CHECKED	DATE	IMPELL CORPORATION	JOB NO 0210-040	PAGE 4 OF 23
1	[illegible]	3-14-86	VJB	3-18-86		CALC NO M-04	

GUSSET PLATE (2 BOLTS) CONT'D.

CASE 7: $\Delta z_{43} = .011637$

$k = 1000/.011637 = \underline{\underline{86\ k/in}}$

CASE 8: $\Delta z_{21} = -.000098$, $\Delta z_{22} = .000719$ $\Big\rangle$ .000817

$\angle = \sin^{-1} \frac{.000817}{1.5} = .031271°$ OR $.00054467$

$k = \frac{1000}{.00054467} = \underline{\underline{1836\ k\text{-}in/rad}}$

CASE 9: $\Delta z_{41} = -.000057$, $\Delta z_{42} = .001693$ $\Big\rangle$ .00175

$\angle = \sin^{-1} \frac{.00175}{1.5} = .066845°$ OR $.00116667$

$k = \frac{1000}{.00116667} = \underline{\underline{857\ \frac{k\text{-}in}{rad}}}$

REV	BY	DATE	CHECKED	DATE	IMPELL CORPORATION	JOB NO 0210-040	PAGE 5
1	KO	3-14-86	VRS	3-18-86		CALC NO M-04	OF 29

∠ 6x4x3/4 (2 BOLTS) (OUTPUT 86/03/13 17.56.20)

CASE #1:

$\Delta z_{21} = .001397$
$\Delta z_{22} = .000826$ } .000571

$\angle = \sin^{-1} = \frac{.000571}{1.5} = .02181059$ ° OR .00038067 RAD

$K = 1000/.00038067 = \underline{\underline{2627}}$ K-IN/RAD

CASE #2:

$\Delta z_{21} = .007413$

$K = 1000/.007413 = \underline{\underline{135}}$ K/IN

CASE #3:

$\Delta z_{41} = .001062$
$\Delta z_{42} = .000618$ } .000444

$\angle = \sin^{-1} = \frac{.000444}{1.5} = .01695955$ ° OR .000296 RAD

$K = 1000/.000296 = \underline{\underline{3378}}$ K-IN/RAD

CASE #4:

$\Delta z_{41} =$

$K = 1000/.005681 = \underline{\underline{176}}$ K/IN

CASE #5:

$\Delta z_{42} = .00071$
$\Delta z_{44} = .00005$ } .00066

$\angle = \sin^{-1} \frac{.00066}{3} = .01260507$ ° OR .00022 RAD

$K = 1000/.00022 = \underline{\underline{4545}}$ K-IN/RAD

CASE #6:

$\Delta z_{42} = .000273$
$\Delta z_{44} = .001356$ } .001083

$\angle = \sin^{-1} \frac{.001083}{3} = .02068378$ ° OR .000361 RAD

$K = 1000/.000361 = \underline{\underline{2771}}$ K-IN/RAD

REV	BY	DATE	CHECKED	DATE
1	AD	3-17-86	VJS	3-18-86

IMPELL CORPORATION

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CL#4 & #4 (DOUBLES) CONT'D:

CASE 7: $\Delta z_{43} = .002443$

$k = 1000/.002443 = \underline{\underline{409}}$ K/IN

CASE 8: $\Delta z_{41} = -.000075$

$\Delta z_{42} = .000266$ $\Big\rangle$ .000341

$\angle = \sin^{-1} \frac{.000341}{1.5} = .013078424^{\circ}$ OR .00022737 RAD

$k = 1000/.00022737 = \underline{\underline{4399}}$ K-IN/RAD

CASE #9: $\Delta z_{41} = -.000057$

$\Delta z_{42} = .00036$ $\Big\rangle$ .000417

$\angle = \sin^{-1} \frac{.000417}{1.5} = .015979823^{\circ}$ OR .000278 RAD

$k = \frac{1000}{.000278} = \underline{\underline{3597}}$ $\frac{\text{K-IN}}{\text{RAD}}$

REV	BY	DATE	CHECKED	DATE	IMPELL CORPORATION	JOB NO 0210-040	PAGE 7 OF 23
/	[illegible]	3-17-86	[illegible]	3-18-86		CALC NO M-04	

L 5"x5"x 3/4" 2-BOLT MODEL

CASE # 1

$\Delta Z_{21} = .002256$
$\Delta Z_{22} = .001510$ } .000746

$\angle = \sin^{-1} \frac{.000746}{1.25} = .034194°$ OR .000597 RAD

$K = \frac{1000}{.000597} = 1676 \frac{KP \cdot IN}{RAD}$

CASE # 2

$\Delta Z_{21} = .010816$

$K = \frac{1000}{.010816} = 92 \frac{KIP}{IN}$

CASE #3

$\Delta Z_{41} = .001896$
$\Delta Z_{42} = .001276$ } .000620

$\angle = \sin^{-1} \frac{.000620}{1.25} = .028419°$ OR .000496 RAD

$K = \frac{1000}{.000496} = 2016 \frac{KP \cdot IN}{RAD}$

CASE #4

$\Delta Z_{41} = .009073$

$K = \frac{1000}{.009073} = 110 \frac{KIP}{IN}$

CASE #5

$\Delta Z_{42} = .001353$
$\Delta Z_{44} = .000246$ } .001107

$\angle = \sin^{-1} \frac{.001107}{2.50} = .025371°$ OR .000443 RAD

$K = \frac{1000}{.000443} = 2258 \frac{KP \cdot IN}{RAD}$

REV	BY	DATE	CHECKED	DATE		JOB NO 0210-040	PAGE 8
0	ELC	10-4-95	VJF	12-4-95	IMPELL CORPORATION	CALC NO M-04	OF 23

L 5"x5"x 3/4" 2-BOLT MODEL (CONT)

CASE #6

$\Delta Z_{42} = .000501$
$\Delta Z_{44} = .002019$ } .001518

$\angle = \sin^{-1} \frac{.001518}{2.50} = .034790°$ OR .000607 RAD

$K = \frac{1000}{.000607} = 1647 \frac{KP.IN}{RAD}$

CASE #7

$\Delta Z_{43} = .003141$

$K = \frac{1000}{.003141} = 318 \frac{KIP}{IN}$

CASE #8

$\Delta Z_{21} = -.000093$
$\Delta Z_{22} = .000483$ } .000576

$\angle = \sin^{-1} \frac{.000576}{1.25} = .026402°$ OR .000461 RAD

$K = \frac{1000}{.000461} = 2170 \frac{KP.IN}{RAD}$

CASE #9

$\Delta Z_{41} = -.000074$
$\Delta Z_{42} = .000560$ } .000634

$\angle = \sin^{-1} \frac{.000634}{1.25} = .029060°$ OR .000507 RAD

$K = \frac{1000}{.000507} = 1972 \frac{KP.IN}{RAD}$

REV	BY	DATE	CHECKED	DATE	IMPELL CORPORATION	JOB NO 0210-040	PAGE 9
0	DLC	12-4-85	VJB	12-4-85		CALC NO M-04	OF 23

L6"x6"x3/4 (1-BOLT MODEL)

CASE 1: (-)Mx @ NODE 55

$\Delta z(21) = 0.003465$
$\Delta z(22) = 0.002305$
$\Delta = 0.001160$

$L = \sin^{-1}\frac{0.001160}{1.5} = 0.04431°$ OR 0.000773 RAD

$K = \frac{1000}{0.000773} = 1293 \frac{KP\text{-}IN}{RAD.}$

CASE 2: (+) F_Z @ NODE 55

$\Delta z(21) = 0.018396$

$K = \frac{1000}{0.018396} = 54$ KP/IN

CASE 3: (+)Mx @ NODE 13

$\Delta z(12) = 0.000743$
$\Delta z(14) = 0.002839$
$\Delta = 0.002096$

$L = \sin^{-1}\frac{0.002096}{3} = 0.040031$ OR 0.000699 RAD

$K = \frac{1000}{0.000699} = 1431 \frac{KIP\text{-}IN}{RAD.}$

CASE 4: (-) Mx @ NODE 13

$\Delta z(12) = 0.002607$
$\Delta z(14) = 0.000712$
$\Delta = 0.001985$

$L = \sin^{-1}\frac{0.001985}{3} = 0.037911°$ OR 0.000662 RAD

$K = \frac{1000}{0.000662} = 1511$ KIP IN/RAD.

REV	BY	DATE	CHECKED	DATE
0	[illegible]	11.26.85	ELC	12.6.85

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CASE 5: (+)Mx @ NODE 55

$\Delta z(21) = -0.000088$

$\Delta z(22) = 0.0.000689$ } $\Delta = 0.000777$

$L = \sin^{-1} \frac{0.000777}{1.5} = 0.029679°$ OR 0.000518 RAD

$K = \frac{1000}{0.000518} = 1931$ KIP-IN/RAD.

L

L6" x 6" x 3/8" (1-BOLT MODEL)

CASE 1: (-) Mx @ NODE 55

$\Delta z(21) = 0.008276$

$\Delta z(22) = 0.004037$ } $\Delta = 0.004239$"

$L = \sin^{-1} \frac{0.004239}{1.5} = 0.161918°$ OR 0.002826 RAD.

$K = \frac{1000}{0.002826} = 354$ KIP-IN/RAD.

CASE 2: (+) Fz @ NODE 55

$\Delta z(21) = 0.032456$

$K = \frac{1000}{0.032456} = 31$ KIP/IN.

CASE 3: (+) Mx @ NODE 13

$\Delta z(12) = 0.001552$

$\Delta z(14) = 0.008433$ } $\Delta = 0.006881$

$L = \sin^{-1} \frac{0.006881}{3} = 0.131418°$ OR 0.002294 RAD

$K = \frac{1000}{0.002294} = 436$ KIP-IN/RAD.

REV	BY	DATE	CHECKED	DATE
1	[illegible]	2-14-86	JPB	3-18-86
0	JRB	11-26-85	ELL	12-6-85

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$\angle$ 6x6x 3/4 (1 BOLT)

CASE 1: $\Delta_{z21} = .007065$, $\Delta_{z22} = .003138$ $\Big\rangle$ .003927

$\angle = \sin^{-1} \frac{.003927}{1.5} = .15°$ OR .002618 RAD

$k = 1000/.002618 = 382$ K-IN/RAD (354)

CASE 2: $\Delta_{z21} = .025104$

$k = 1000/.025104 = 40$ K/IN (41)

CASE 3: $\Delta_{z12} = .00125$, $\Delta_{z14} = .007516$ $\Big\rangle$ .006266

$\angle = \sin^{-1} \frac{.006266}{3} = .11967167°$ OR .0020887 RAD

$k = 1000/.0020887 = 479$ K-IN/RAD (436)

CASE 4: $\Delta_{z12} = .005407$, $\Delta_{z14} = .000747$ $\Big\rangle$ .00466

$\angle = \sin^{-1} \frac{.00466}{3} = .08899448°$ OR .0015533 RAD

$k = 1000/.0015533 = 644$ K-IN/RAD (582)

CASE 5: $\Delta_{z21} = -.000095$, $\Delta_{z22} = .000843$ $\Big\rangle$ .000938

$\angle = \sin^{-1} \frac{.000938}{1.5} = .03582896°$ OR .0006253 RAD

$k = 1000/.0006253 = 1594$ K-IN/RAD (1264)

REV	BY	DATE	CHECKED	DATE		JOB NO 0210.040	PAGE 17 OF 23
1	AO	9-14-86	[illegible]	3-18-86	IMPELL CORPORATION	CALC NO M-04	

L 6×4×3/4 (1 BOLT MODEL)

CASE 1: $\Delta z_{11} = .002262$, $\Delta z_{22} = .001389$ → $\Delta = .000873$

$\angle = \sin^{-1} \frac{.000873}{1.5} = .03394615°$ OR $.000582$ RAD

$k = 1000 / .000582 = \underline{1718}$ K-IN/RAD

CASE 2: $\Delta z_{11} = .011073$

$k = 1000 / .011073 = \underline{90}$ K/IN

CASE 3: $\Delta z_{12} = .000446$, $\Delta z_{13} = .001936$ → $.00149$

$\angle = \sin^{-1} \frac{.00149}{3} = .028495691°$ OR $.00049667$ RAD

$k = 1000 / .00049667 = \underline{2013}$ K-IN/RAD

CASE 4: $\Delta z_{12} = .001793$, $\Delta z_{13} = .000414$ → $.001379$

$\angle = \sin^{-1} \frac{.001379}{3} = .026332496°$ OR $.00045967$ RAD

$k = 1000 / .00045967 = \underline{2175}$ K-IN/RAD

CASE 5: $\Delta z_{11} = -.000085$, $\Delta z_{22} = .000392$ → $.000477$

$\angle = \sin^{-1} \frac{.000477}{1.5} = .018220006°$ OR $.00031$ RAD

$k = 1000 / .000318 = \underline{3145}$ K-IN/RAD

REV	BY	DATE	CHECKED	DATE		JOB NO 0210.040	PAGE 13 OF 23
1	[illegible]	3-17-86	[illegible]	3-18-86	IMPELL CORPORATION	CALC NO M-04	

L 5" x 5" x 3/4" (1-BOLT MODEL)

CASE 1: (-) Mx @ NODE 55

$\Delta z(21) = 0.003908$
$\Delta z(22) = 0.002680$
$\Delta = 0.001228$"

$L = \sin^{-1} \frac{0.001228}{1.25} = 0.056287°$ OR 0.000982 RAD

$K = \frac{1000}{0.000982} = 1018$ KIP-IN/RAD.

CASE 2: (+) Fz @ NODE 55

$\Delta z(21) = 0.017808$"

$K = \frac{1000}{0.017808} = 56$ KIP-IN/RAD

CASE 3: (+) Mx @ NODE 13

$\Delta z(12) = 0.000889$
$\Delta z(14) = 0.003242$
$\Delta = 0.002353$"

$L = \sin^{-1} \frac{0.002353}{2.5} = 0.053927$ OR 0.000941 RAD.

$K = \frac{1000}{0.000941} = 1062$ KIP-IN/RAD.

CASE 4: (-) Mx @ NODE 13

$\Delta z(12) = 0.003051$
$\Delta z(14) = 0.000829$
$\Delta = 0.002222$

$L = \sin^{-1} \frac{0.002222}{2.5} = 0.050924$ OR 0.000889 RAD.

$K = \frac{1000}{0.000889} = 1125$ KIP-IN/RAD.

REV	BY	DATE	CHECKED	DATE	IMPELL CORPORATION	JOB NO 0210-040	PAGE 14
0	VRB	11-27-85	ELC	12-6-85		CALC NO M-04	OF 23

CASE 5 : (+) MX @ NODE 55

$\Delta z(21) = -0.000115$

$\Delta z(22) = 0.000798$

$\Delta = 0.000913''$

$L = \sin^{-1} \frac{0.000913}{1.25} = 0.041849^\circ$ OR 0.000730 RAD

$K = \frac{1000}{0.000730} = 1369$ KIP-IN/RAD.

REV	BY	DATE	CHECKED	DATE
0	[illegible]	11-27-85	ELC	12-6-85

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L5x5x3/4: 2BOLT MODEL (CONFIG. CASE B&D)

LOAD CASE #1: $\Delta z_{21} - \Delta z_{22} = .005798 - .003277 = .002521$

$\theta = \sin^{-1} \frac{.002521}{3.75/2} = .077036°$ OR $.001345$

$K = 1000/.001345 = \underline{744\ \frac{KIP\text{-}IN}{RAD}}$

LOAD CASE #2: $\Delta z_{21} = .027555$

$K = 1000/.027555 = \underline{36.\ K/IN}$

LOAD CASE #3: $\Delta z_{51} - \Delta z_{52} = .005148 - .003017 = .002131$

$\theta = \sin^{-1} \frac{.002131}{3.75/2} = .065°$ OR $.0011365$ RAD.

$K = 1000/.0011365 = \underline{880\ KIP/IN}$

LOAD CASE #4: $\Delta z_{51} = .024988$

$K = 1000/.024988 = \underline{40\ K/IN}$

LOAD CASE #5: $\Delta z_{52} - \Delta z_{54} = .003088 - .000281 = .002807$

$\theta = \sin^{-1} \frac{.002807}{2.5} = .064°$ OR $.001123$ RAD.

$K = 1000/.001123 = \underline{891\ \frac{KIP\text{-}IN}{RAD}}$

LOAD CASE #6: $\Delta z_{54} - \Delta z_{52} = .001695 - .000487 = .001208$

$\theta = \sin^{-1} \frac{.001208}{2.5} = .028°$ OR $.0004832$ RAD

$K = 1000/.0004832 = \underline{2070\ K\text{-}IN/RAD}$

LOAD CASE #7: $\Delta z_{53} = .004606$

$K = 1000/.004606 = \underline{217\ K/IN}$

REV	BY	DATE	CHECKED	DATE
1	BD	3-5-86	VJB	3-10-86

IMPELL CORPORATION

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L5×5×3/4 : 2BOLT MODEL (CONT'D)

LOAD CASE #8 : $\Delta_{z21} = -.00005$, $\Delta_{z22} = .000271$ $\Big\rangle = .000321$

$\theta = \sin^{-1} \frac{.000321}{3.75/2} = .0098°$ OR $.00017$ RAD

$K = 1000/.00017 = 5,841$ K-IN/RAD

LOAD CASE #9 : $\Delta_{z51} = -.000031$, $\Delta_{z52} = .000495$ $\Big\rangle = .000526$

$\theta = \sin^{-1} \frac{.000526}{3.75/2} = .016°$ OR $.00028053$ RAD

$K = 1000/.00028053 = 3565$ K-IN/RAD

REV	BY	DATE	CHECKED	DATE		JOB NO 0210-040	PAGE 17
1	[illegible]	3-5-86	[illegible]	3-10-86	IMPELL CORPORATION	CALC NO M-04	OF 23

L 6×6×3/4 : 2 BOLT MODEL. (CONFIG CASE F & H)

LOAD CASE #1 : $\Delta z_{21} = .002978$, $\Delta z_{22} = .001708$ $\Big\rangle = .00127$

$\theta = \sin^{-1} .00127/2 = .036°$ OR $.00064$ RAD

$K = 1000/.00064 = \underline{1575}$ K-IN/RAD

LOAD CASE #2 : $\Delta z_{21} = .016846$

$K = 1000/.016846 = \underline{59}$ K/IN

LOAD CASE #3 : $\Delta z_{51} = .002943$, $\Delta z_{52} = .001411$ $\Big\rangle .000932$

$\theta = \sin^{-1} .000932/2 = .027°$ OR $.00047$ RAD

$K = 1000/.00047 = \underline{2146}$ K-IN/RAD

LOAD CASE #4 : $\Delta z_{51} = .01352$

$K = 1000/.01352 = \underline{74}$ K/IN

LOAD CASE #5 : $\Delta z_{52} = .001502$, $\Delta z_{54} = .000243$ $\Big\rangle = .001259$

$\theta = \sin^{-1} .001259/3 = .024°$ OR $.00042$ RAD

$K = 1000/.00042 = \underline{2383}$ K-IN/RAD

LOAD CASE #6 : $\Delta z_{52} = .000496$, $\Delta z_{54} = .00169$ $\Big\rangle .001194$

$\theta = \sin^{-1} \dfrac{.001194}{3} = .0228°$ OR $.000398$ RAD

$K = 1000/.000398 = \underline{2513}$ K-IN/RAD

REV	BY	DATE	CHECKED	DATE		JOB NO 0210-040	PAGE 18 OF 23
1	BM	3-5-86	[illegible]	3-10-86	IMPELL CORPORATION	CALC NO M-04	

L6x6x3/4 : 2BOLT MODEL (CONT'D)

LOAD CASE 7 : $\Delta z_{57} = .004684$

$k = 1000/.004684 = \underline{214\ k/in}$

LOAD CASE 8 : $\Delta z_{28} = -.000048$, $\Delta z_{22} = .000266$ $\Big\rangle$ .000314

$\theta = \sin^{-1} \frac{.000314}{2} = .009°$ OR $.000157$ RAD

$k = 1000/.000157 = \underline{6369\ k\text{-}in/RAD}$

LOAD CASE 9 : $\Delta z_{51} = -.000021$, $\Delta z_{52} = .000477$ $\Big\rangle$ .000498

$\theta = \sin^{-1} \frac{.000498}{2} = .0143°$ OR $.000249$ RAD

$k = 1000/.000249 = \underline{4016\ k\text{-}in/RAD}$

REV	BY	DATE	CHECKED	DATE
1	DD	7-5-86	VJC	8-10-86

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6

L 6×5×3/4 : 1 BOLT MODEL (CONFIG CASE 3)

LOAD CASE 1: $\Delta z_{11} = .010065$, $\Delta z_{22} = .005778$ $\rangle$ .004287

$\theta = \sin^{-1} \frac{.004287}{3.75/2} = .131°$ OR .00229 RAD

$K = 1000/.00229 =$ 437 K-IN/RAD

LOAD CASE 2: $\Delta z_{21} = .046733$

$K = 1000/.046733 =$ 21 K/IN

LOAD CASE 3: $\Delta z_{12} = .000461$, $\Delta z_{14} = .00162$ $\rangle$ .001159

$\theta = \sin^{-1} \frac{.001159}{2.5} = .0266°$ OR .0004636 RAD

$K = 1000/.0004636 =$ 2157 K-IN/RAD

LOAD CASE 4:

$\Delta z_{12} = .006152$, $\Delta z_{14} = .000848$ $\rangle$ .005304

$\theta = \sin^{-1} \frac{.005304}{2.5} = .1216°$ OR .002 RAD

$K = 1000/.002 =$ 471 K-IN/RAD

LOAD CASE 5: $\Delta z_{11} = -.000053$, $\Delta z_{22} = .000365$ $\rangle$ .000418

$\theta = \sin^{-1} \frac{.000418}{3.75/2} = .0128°$ OR .0002 RAD

$K = 1000/.0002 =$ 4486 K-IN/RAD

REV	BY	DATE	CHECKED	DATE
1	AD	3-5-86	VGS	3-10-86

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L6×6×3/4 : 1 BOLT MODEL (CONFIG. CASE 'L')

LOAD CASE 1: $\Delta z_{21} = .004849$, $\Delta z_{22} = .002820$ } .002029

$\theta = \sin^{-1} \frac{.002029}{2} = .058°$ OR .001 RAD

$K = 1000/.001 = \underline{\underline{986 \text{ K-IN/RAD}}}$

LOAD CASE 2: $\Delta z_{21} = .025833$

$K = 1000/.025833 = \underline{\underline{39 \text{ K/IN}}}$

LOAD CASE 3: $\Delta z_{12} = .000382$, $\Delta z_{14} = .001571$ } = .001189

$\theta = \sin^{-1} \frac{.001189}{3} = .023°$ OR .0004 RAD

$K = 1000/.0004 = \underline{\underline{2523 \text{ K-IN/RAD}}}$

LOAD CASE 4: $\Delta z_{12} = .003217$, $\Delta z_{14} = .000587$ } .00263

$\theta = \sin^{-1} \frac{.00263}{3} = .05°$ OR .00088 RAD

$K = 1000/.00088 = \underline{\underline{1141 \text{ K-IN/RAD}}}$

LOAD CASE 5: $\Delta z_{21} = -.000049$, $\Delta z_{22} = .00035$ } .000399

$\theta = \sin^{-1} \frac{.000399}{2} = .011°$ OR .0002 RAD

$K = 1000/.0002 = \underline{\underline{5013 \text{ K-IN/RAD}}}$

REV	BY	DATE	CHECKED	DATE
1	[illegible]	3-5-86	[illegible]	3-10-86

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CALC NO M-04

BASE ANGLE STIFFNESS TABLE (2-BOLT BASE ANGLE)

BASE ANGLE CONFIGURATION CASE	ANGLE SIZE	LOADING CASE	STIFFNESS (K)	COMMENTS / REF.
B	L 5×5×3/4	3	$K_z = 880 \frac{K\text{-}IN}{RAD}$	BASEPLATE II OUTPUT 86/03/04 19.10.59
		4	$K_z = 40\ K/IN$	
		5	$K_{xx} = 891 \frac{K\text{-}IN}{RAD}$	
		6	$K_{xx} = 2070 \frac{K\text{-}IN}{RAD}$	
D		1	$K_{xx} = 744 \frac{K\text{-}IN}{RAD}$	
D		2	$K_z = 36\ K/IN$	
B		7	$K_z = 217\ K/IN$	
D		8	$K_{xx} = 5841 \frac{K\text{-}IN}{RAD}$	
B		9	$K_{xx} = 3565 \frac{K\text{-}IN}{RAD}$	
F	L 6×6×3/4	3	$K_{xx} = 2146 \frac{K\text{-}IN}{RAD}$	BASEPLATE II OUTPUT 86/03/05 19.30.09
		4	$K_z = 74\ K/IN$	
		5	$K_{xx} = 2383 \frac{K\text{-}IN}{RAD}$	
		6	$K_{xx} = 2513 \frac{K\text{-}IN}{RAD}$	
		7	$K_z = 214\ K/IN$	
		9	$K_{xx} = 4016 \frac{K\text{-}IN}{RAD}$	
H		1	$K_{xx} = 1575 \frac{K\text{-}IN}{RAD}$	
		2	$K_z = 59\ K/IN$	
		8	$K_{xx} = 6369 \frac{K\text{-}IN}{RAD}$	

REV	BY	DATE	CHECKED	DATE
1	AD	3-5-86	[illegible]	3-10-86

IMPELL CORPORATION

JOB NO 0210-040

CALC NO M-04

BASE ANGLE STIFFNESS TABLE (1-BOLT BASE ANGLE)

BASE ANGLE CONFIGURATION CASE	ANGLE SIZE	LOADING CASE	STIFFNESS (K)	COMMENTS/REF.
J	L5x5x 3/4	1	K_{xx} = 437 K-IN/RAD	BASEPLATE II OUTPUT 86/03/04 19.14.02
		2	K_z = 21 K/IN	"
		3	K_{xx} = 2157 K-IN/RAD	BASEPLATE II OUTPUT 86/03/04 19.16.17
		4	K_{xx} = 471 K-IN/RAD	↓
		5	K_{xx} = 4486 K-IN/RAD	
L	L6x6x 3/4	1	K_{xx} = 986 K-IN/RAD	BASEPLATE II OUTPUT 86/03/05 13.36.35
		2	K_z = 39 K/IN	"
		3	K_{xx} = 2503 K-IN/RAD	BASEPLATE II OUTPUT 86/03/05 13.34.12
		4	K_{xx} = 1141 K-IN/RAD	↓
		5	K_{xx} = 5013 K-IN/RAD	

REV	BY	DATE	CHECKED	DATE
1	[illegible]	7-5-86	[illegible]	8-10-86

IMPELL CORPORATION

JOB NO 0210-040

CALC NO M-04

3.0 SUMMARY OF RESULTS / RECOMMENDATIONS

A SUMMARY OF THE RANGE OF THE STIFFNESSES DETERMINED IS PRESENTED IN TABLE 1 ON THE FOLLOWING PAGE. IT HAS BEEN SHOWN THAT BASE ANGLES WITH THE SAME NUMBER OF BOLTS, AND SAME THICKNESS, EXHIBIT APPROXIMATELY THE SAME STIFFNESS REGARDLESS WHETHER THE ATTACHMENT IS ON THE FREE OR FIXED LEG.

AS EXPECTED, THE THINNER ANGLES, AND THE 1-BOLT ANGLES EXHIBIT LOWER STIFFNESSES THAN THE THICKER AND 2-BOLT CONFIGURATIONS RESPECTIVELY.

IT IS RECOMMENDED THAT THE AVERAGE VALUES OF ROTATIONAL STIFFNESS BE USED FOR PRODUCTION WORK BASED ON ANGLE THICKNESS AND NUMBER OF BOLTS AS FOLLOWS:

BASE L THICKNESS	2-BOLT	1-BOLT
0.375"	1200 $\frac{KP.IN}{RAD}$	1000 $\frac{KP.IN}{RAD}$
0.75"	3600 $\frac{KP.IN}{RAD}$	2600 $\frac{KP.IN}{RAD}$

FOR THE PULL-OUT STIFFNESS, IT IS RECOMMENDED THAT THE CONNECTION BE MODELED AS RIGID SINCE THE STIFFNESS VARIES BETWEEN A MODERATELY STIFF CONNECTION AND A RIGID CONNECTION DEPENDING ON DIRECTION OF LOAD.

REV	BY	DATE	CHECKED	DATE
1	[illegible]	9-14-86	[illegible]	9-18-86
0	ELL	12-6-85	[illegible]	12-9-85

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JOB NO 0210-040
CALC NO M-04
PAGE 1 OF 3

TABLE 1
STIFFNESS SUMMARY
(UNITS: K_{xx} (KIP-IN/RAD), K_z (KIP/IN))

	2-BOLT				1-BOLT				COMMENTS
	BOLTED LEG		FREE LEG		BOLTED LEG		FREE LEG		
	K_{xx}	K_z	K_{xx}	K_z	K_{xx}	K_z	K_{xx}	K_z	
L6x6x3/8	523-1480	86-∞	524-1836	52-∞	479-644	N/A	382-1599	40-∞	
L6x6x3/4	2287-3615*	214-∞	1575-6369	59-∞	1141-2523		986-5013	39-∞	
L6x4x3/4	2771-4545	409-∞	2627-4399	135-∞	2013-2175		1712-3145	90-∞	
L5x5x3/4	991-2697	217-∞	796-5041	36-∞	471-2157		437-4436	21-∞	

* 2158 (1.06) = 2287
3090 (1.17) = 3615

NOTE: ABOVE VALUES ARE ENVELOPED FROM THE RESULTS SHOWN ON PAGES 2-21 OF THIS CALC. THE ENVELOPING IS DONE SUCH THAT, THE STIFFNESS VALUES FOR L5x5x3/4 FROM REV. 0 OF THIS CALC. WERE INCREASED BY THE SAME FACTOR AS THE STIFFNESS FOR L6x6x3/8 INCREASED THEN ENVELOPED W/ THE VALUES SHOWN ON PAGE 16, 17 AND 20 OF THIS CALC.

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CALC NO M-04
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REV	BY	DATE	CHECKED	DATE
1	[illegible]	3-10-86	[illegible]	3-11-86

REV 1 SUMMARY
OF RESULTS AND RECOMMENDATIONS

PRELIMINARY REVIEW OF THE CPSES CABLE TRAY SYSTEMS CURRENTLY BEING ANALYZED BY IMPELL SHOWS THAT THE ANCHORAGES ARE COVERED BY THE RANGE OF CONFIGURATIONS CONSIDERED IN THIS CALC. THAT IS, NO ANCHORAGES HAVE BEEN FOUND WHICH ARE MORE FLEXIBLE THAN ALREADY ANALYZED IN THIS CALC.

IN FUTURE CABLE TRAY SYSTEM ANALYSES, ANY ANCHORAGES IDENTIFIED AS BEING MORE FLEXIBLE THAN THOSE COVERED IN THIS CALC SHOULD BE BROUGHT TO THE ATTENTION OF THE LEAD ENGINEER AND ANALYZED ON A CASE BY CASE BASIS. ANCHORAGE STIFFNESS VALUES ARE SUMMARIZED IN THE TABLE SHOWN ON Pg 2 OF 3. RECOMMENDED VALUES FOR PRODUCTION ARE AS SHOWN ON P. 1 OF 3.

REV. 1 OF THIS CALC. ALSO INCORPORATED THE FOLLOWING CHANGES:

1.) MADE ANGLE DIMENSION CHANGES

2) CONCRETE COMPRESSIVE STRENGTH VALUE TO 4000 PSI.

3) BOLT STIFFNESS VALUE TO; K_T = 461,000 #/IN
K_V = 111,000 #/IN

REV	BY	DATE	CHECKED	DATE		JOB NO 0210-040	PAGE 3
1	DC	3-6-86	AD	3-10-86	IMPELL CORPORATION	CALC NO M-04	OF 3

4.0 REFERENCES:

A) BASEPLATE-II COMPUTER RUNS:

CALC. NO.	ANGLE	BOLTS	DATE/TIME ID		CASES	
M-04-CR01	L 6×6× 3/4	2	85/11/04.	18.20.50.	1-7	
M-04-CR02	L 6×6× 3/4	2	85/11/12.	12.30.28.	8,9	
M-04-CR03	L 6×6× 3/8	2	85/11/04.	18.56.44.	1-7	*
M-04-CR04	L 6×6× 3/8	2	85/11/18.	12.41.20.	8,9	*
M-04-CR05	L 6×4× 3/4	2	85/11/04.	19.06.34.	1-7	*
M-04-CR06	L 6×4× 3/4	2	85/11/12.	13.08.30.	8,9	*
M-04-CR07	L 5× 5× 3/4	2	85/11/11.	11.46.23.	1-7	
M-04-CR08	L 5× 5× 3/4	2	85/11/12.	13.09.18.	8,9	
M-04-CR09	L 6×6× 3/4	1	85/11/11.	12.40.56.	1,2	
M-04-CR10	L 6×6× 3/4	1	85/11/25.	14.59.25.	3-5	
M-04-CR11	L 6×6× 3/8	1	85/11/11.	13.37.38.	1,2	*
M-04-CR12	L 6×6× 3/8	1	85/11/26.	18.22.39.	3-5	*
M-04-CR13	L 6×4× 3/4	1	85/11/11.	13.42.43.	1,2	*
M-04-CR14	L 6×4× 3/4	1	85/11/26.	18.29.36.	3-5	*
M-04-CR15	L 5×5× 3/4	1	85/11/11.	13.41.02.	1,2	
M-04-CR16	L 5×5× 3/4	1	85/11/26.	18.35.54.	3-6	

(SEE PAGE 2 OF 2 FOR ADDITIONAL REFERENCES)

B) COMPUTER PROGRAM: BASEPLATE-II VERSION 2.5

C) EDS BASEPLATE DESIGN PROCEDURE 2.6.1 REV. 0

D) REF. DWG. 2323-S-903 REV. 5
2323-S-908 REV. 1 & 909 REV. 1

E) CABLE TRAY SYSTEM PARAMETRIC SENSITIVITY STUDY
CALC 0210-040-M11 REV 0.

F) EBASCO GENERAL INSTRUCTIONS FOR CABLE TRAY HANGER ANALYSIS FOR CASES UNIT 1 & 2, ATTACHMENT G6 REV. 2

REV	BY	DATE	CHECKED	DATE
1	AD	3-10-86	VRB	3-10-86
0	ELC	10-9-85	VSR	12-9-85

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CALC NO M-04
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4.0 REFERENCES (CONT'D)

A) BASEPLATE II COMPUTER RUNS

CALC. NO.	ANGLE	BOLTS	DATE / TIME	LOAD CASES
M-04-CR17	5x5x3/4	2-BOLTS	86/03/04. 19.10.59	1-9
M-04-CR18	6x6x3/4	2-BOLTS	86/03/05 19.30.09	1-9
M-04-CR19	5x5x3/4	1-BOLT	86/03/04 19.14.02	1-2
M-04-CR20	5x5x3/4	1-BOLT	86/03/04 19.16.17	3-5
M-04-CR21	6x6x3/4	1-BOLT	86/03/05 13.36.33	1-2
M-04-CR22	6x6x3/4	1-BOLT	86/03/05 13.24.12	3-5
23	6x4x3/4	1-BOLT	86/03/13 18.28.58	1-2
24	6x4x3/4	1-BOLT	86/03/13 18.37.33	3-5
25	6x4x3/4	2-BOLTS	86/03/13 17.56.20	1-9
26	6x6x3/4	2-BOLTS	86/03/13 17.41.54	1-9
27	6x6x3/8	1-BOLT	86/03/13 18.14.21	1-2
28	6x6x3/8	1-BOLT	86/03/13 18.17.52	3-5

* NOTE: SUPERSEDES BASEPLATE II RUNS

REV	BY	DATE	CHECKED	DATE
1	AD	3-10-86	[illegible]	3-10-86

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CALC NO M-04

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APPENDIX A

SUPERCEDED PAGES (REPLACED)

REV	BY	DATE	CHECKED	DATE
1	DC	3-10-86	DO	3-11-86

IMPELL CORPORATION

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CALC NO M-04

TABLE 1
STIFFNESS SUMMARY
(UNITS: K_{xx} (KP-IN/RAD), K_z (KIP/IN))

ANGLE SIZE	2-BOLT				1-BOLT				COMMENTS
	BOLTED LEG		FREE LEG		BOLTED LEG		FREE LEG		
	K_{xx}	K_z	K_{xx}	K_z	K_{xx}	K_z	K_{xx}	K_z	
L 6" x 6" x 3/8"	496-1261	80-∞	492-1426	46-∞	436-548	N/A	354-1204	31-∞	
L 6" x 6" x 3/4"	2158-3090	298-∞	2080-2941	86-∞	1431-1511		1293-1931	54-∞	
L 6" x 4" x 3/4"	2155-3090	298-∞	2063-2935	87-∞	1425-1503		1270-1911	54-∞	
L 5" x 5" x 3/4"	1647-2258	318-∞	1676-2170	92-∞	1062-1125		1018-1369	56-∞	

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CALC NO M-04
PAGE 2 OF 2

REV	BY	DATE	CHECKED	DATE
0	ELC	12-9-85	[illegible]	12-9-85

L 6"x6"x 3/8" 2-BOLT MODEL

CASE #1

$\Delta Z_{21} = .005740$
$\Delta Z_{22} = .002693$ } .003047

(K_{xx} - FREE LEG)

$\angle = \sin^{-1} \frac{.003047}{1.5} = .116387°$ OR .002031 RAD

$K = \frac{1000}{.002031} = 492 \frac{KP \cdot IN}{RAD}$

CASE #2

$\Delta Z_{21} = .024524$

(K_z - FREE LEG)

$K = \frac{1000}{.024524} = 41 \frac{KIP}{IN}$

CASE #3

$\Delta Z_{41} = .003542$
$\Delta Z_{42} = .001426$ } .002116

(K_{xx} - FREE LEG)

$\angle = \sin^{-1} \frac{.002116}{1.5} = .080825°$ OR .001411 RAD

$K = \frac{1000}{.001411} = 709 \frac{KP-IN}{RAD}$

CASE #4

$\Delta Z_{41} = .016492$

(K_z - FREE LEG)

$K = \frac{1000}{.016492} = 61 \frac{KIP}{IN}$

CASE #5

$\Delta Z_{42} = .002332$
$\Delta Z_{44} = -.000047$ } .002379

(K_{xx} - BOLTED LEG)

$\angle = \sin^{-1} \frac{.002379}{3.0} = .045486°$ OR .000793 RAD

$K = \frac{1000}{.000793} = 1261 \frac{KP \cdot IN}{RAD}$

REV	BY	DATE	CHECKED	DATE		JOB NO 0210-040	PAGE 4
0	ALC	12-4-85	VJS	12-4-85	IMPELL CORPORATION	CALC NO M-04	OF 23

L 6"x6"x 3/8" 2-BOLT MODEL (CONT)

CASE #6: $\Delta Z_{44} = .007298$, $\Delta Z_{42} = .001248$ } .006050

(K_{xx} - BOLTED LEG) $\angle = \sin^{-1} \frac{.006050}{3.0} = .115547°$ OR .002017 RAD

$K = \frac{1000}{.002017} = 496 \frac{KP.IN}{RAD}$

CASE #7: $\Delta Z_{43} = .012575$

(K_z - BOLTED LEG) $K = \frac{1000}{.012575} = 80 \frac{KIP}{IN}$

CASE #8: $\Delta Z_{21} = -.000100$, $\Delta Z_{22} = .000952$ } .001052

(K_{xx} - FREE LEG) $\angle = \sin^{-1} \frac{.001052}{1.5} = .040183°$ OR .000701 RAD

$K = \frac{1000}{.000701} = 1426 \frac{KIP}{IN}$

CASE #9: $\Delta Z_{41} = -.000060$, $\Delta Z_{42} = .001848$ } .001908

(K_{xy} - FREE LEG) $\angle = \sin^{-1} \frac{.001908}{1.5} = .072880°$ OR .001272 RAD

$K = \frac{1000}{.001272} = 786 \frac{KIP}{IN}$

REV	BY	DATE	CHECKED	DATE	IMPELL CORPORATION	JOB NO 0210-040	PAGE 5
0	ELC	12-4-85	[illegible]	12-4-85		CALC NO M-04	OF 23

L 6"x 4"x 3/4" 2-BOLT MODEL

CASE #1

K_{xx} - FREE LEG

$\Delta Z_{21} = .002035$
$\Delta Z_{22} = .001308$ } .000727

$\angle = \sin^{-1} \frac{.000727}{1.5} = .027769°$ OR .000485 RAD

$K = \frac{1000}{.000485} = 2063 \frac{KP.IN}{RAD}$

CASE #2

K_z - FREE LEG

$\Delta Z_{21} = .011499$

$K = \frac{1000}{.011499} = 87 \frac{KIP}{IN}$

CASE #3

K_{xx} - FREE LEG

$\Delta Z_{41} = .001676$
$\Delta Z_{42} = .001078$ } .000598

$\angle = \sin^{-1} \frac{.000598}{1.5} = .022842°$ OR .000399 RAD

$K = \frac{.000399}{1000} = 2508 \frac{KP.IN}{RAD}$

CASE #4

K_z - FREE LEG

$\Delta Z_{41} = .009358$

$K = \frac{1000}{.009358} = 107 \frac{KIP}{IN}$

CASE #5

K_{xx} - BOLTED LEG

$\Delta Z_{42} = .001169$
$\Delta Z_{44} = .000198$ } .000971

$\angle = \sin^{-1} \frac{.000971}{3.0} = .018545°$ OR .000324 RA

$K = \frac{1000}{.000324} = 3090 \frac{KP.IN}{RAD}$

L 6"x 4"x 3/4" 2-BOLT MODEL (CONT)

CASE #6

$\Delta Z_{42} = .000424$
$\Delta Z_{44} = .001815$ } .001391

K_{xx} - BOLTED LEG

$\angle = \sin^{-1} \frac{.001391}{3.0} = .026566°$ OR .000464 RAD

$K = \frac{1000}{.000464} = 2155 \frac{KP.IN}{RAD}$

CASE #7

$\Delta Z_{43} = .003358$

K_z - BOLTED LEG

$K = \frac{1000}{.003358} = 298 \frac{KIP}{IN}$

CASE #8

$\Delta Z_{21} = -.000075$
$\Delta Z_{22} = .000436$ } .000511

K_{xx} - FREE LEG

$\angle = \sin^{-1} \frac{.000511}{1.5} = .019519°$ OR .000341 RAD

$K = \frac{1000}{.000341} = 2935 \frac{KP.IN}{RAD}$

CASE #9

$\Delta Z_{41} = -.000060$
$\Delta Z_{42} = .000511$ } .000571

K_{xx} - FREE LEG

$\angle = \sin^{-1} \frac{.000571}{1.5} = .021811°$ OR .000381 RAD

$K = \frac{1000}{.000381} = 2627 \frac{KP.IN}{RAD}$

REV	BY	DATE	CHECKED	DATE	IMPELL CORPORATION	JOB NO 0210-040	PAGE 7
0	ELC	11-1-85	VJE	12-4-85		CALC NO M-04	OF 23

CASE 4 : (-) M_x @ NODE 13

$\Delta z(12) = 0.006695$
$\Delta z(14) = 0.001220$
$\Delta = 0.005475$

$L = \sin^{-1} \frac{0.005475}{3} = 0.104565$ OR 0.001825 RAD

$K = \frac{1000}{0.001825} = 548$ KIP-IN / RAD

CASE 5 : (+) M_x @ NODE 55

$\Delta z(21) = -0.000100$
$\Delta z(22) = 0.001146$
$\Delta = 0.001246"$

$L = \sin^{-1} \frac{0.001246}{1.5} = 0.047594"$ OR 0.0008308

$K = \frac{1000}{0.0008308} = 1204$ KIP-IN / RAD.

L6" x 4 x 3/4" (1-BOLT MODEL)

CASE 1 : (-) M_x @ NODE 55

$\Delta z(21) = 0.003489$
$\Delta z(22) = 0.002308$
$\Delta = 0.001181"$

$L = \sin^{-1} \frac{0.001181}{1.5} = 0.045111°$ OR 0.000787 RAD.

$K = \frac{1000}{0.000787} = 1270$ KIP-IN / RAD

CASE 2 : (+) F_z @ NODE 55

$\Delta z(21) = 0.018418"$

$K = \frac{1000}{0.018418} = 54$ KIP/IN.

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0	VR	11-27-85	ELC	12-6-85

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CASE 3 : (+) M_x @ NODE 13

$\Delta z(12) = 0.000749$
$\Delta z(14) = 0.002854$
} $\Delta = 0.002105$

$L = \sin^{-1} \frac{0.002105}{3} = 0.040203$ OR 0.000702 RAD

$K = \frac{1000}{0.000702} = 1425$ KIP-IN/RAD.

CASE 4 : (-) M_x @ NODE 13

$\Delta z(12) = 0.002712$
$\Delta z(14) = 0.000716$
} $\Delta = 0.001996$

$L = \sin^{-1} \frac{0.001996}{3} = 0.038121°$ OR 0.000665 RAD

$K = \frac{1000}{0.000665} = 1503$ KIP-IN/RAD.

CASE 5 : (+) M_x @ NODE 55

$\Delta z(21) = -0.000090$
$\Delta z(22) = 0.000695$
} $\Delta = 0.000785"$

$L = \sin^{-1} \frac{0.000785}{1.5} = 0.029985°$ OR 0.000523 RAD

$K = \frac{1000}{0.000523} = 1911$ KIP-IN/RAD

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0	VRS	11-27-85	ELL	12-6-85

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CALCULATION/PROBLEM COVER SHEET

Calculation/Problem No: M-10

Title: CABLE TRAY-CLIP ANGLE STIFFNESS

Client: TUGCO **Project:** CPSES-TRAYS

Job No: 0210-040

Design Input/References:

SECTION 5.0

Assumptions:

SECTION 2.0

Method:

SECTION 2.0

Remarks:

OPEN ITEM:

ROC's USED AS REFERENCE 7 REQUIRES VERIFICATION PER QP 2.1 OF QA MANUAL

REV. NO.	REVISION	APPROVED	DATE
RECORD CENTER 0	INITIAL ISSUE	Calvin Yee Wong	12/12/85
RECORD CENTER 1	REFINED CALCULATED STIFFNESSES	Calvin Yee Wong	1/31/86

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ATTACHMENT 1 : COMPUTER RUN FOR TYPE "A"-4" HIGH — 54 PAGES

ATTACHMENT 2 : COMPUTER RUN FOR TYPE "A"-6" HIGH — 31 PAGES

ATTACHMENT 3 : COMPUTER RUN FOR TYPE "B" — 63 PAGES

ATTACHMENT 4 : COMPUTER RUN FOR TYPE "C" — 69 PAGES

ATTACHMENT 5 : COMPUTER RUN FOR TYPE "D" OR "M" — 69 PAGES

ATTACHMENT 6 : COMPUTER RUN FOR TYPE "DA" OR "L" OR "LA" — 69 PAGES

ATTACHMENT 7 : COMPUTER RUN FOR TYPE "G"-4" HIGH — 14 PAGES

ATTACHMENT 8 : COMPUTER RUN FOR TYPE "G"-6" HIGH — 14 PAGES

ATTACHMENT 9 : COMPUTER RUN FOR TYPE "T" — 37 PAGES

ATTACHMENT 10 : . ROC - T. WILK & R. F. OSBORNE 11-13-85
. CLAMP DETAIL - TYPE 'C' CLAMP", TAKEN FROM EBASCO "SPECIFICATION FOR DYNAMIC TESTING"
. ROC - J. VORDERBRUEGGEN & D. CORPORANDY — 4 PAGES

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CABLE TRAY - CLIP ANGLE STIFFNESS

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1.0 PURPOSE :

The purpose of this calculation is to calculate the axial, bending & rotational stiffnesses of clip angles which connect cable trays to structural support members.

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2.0 DESCRIPTION OF ANALYSIS

STIFFNESSES OF REPRESENTATIVE CLIP TYPES HAVE BEEN CALCULATED IN THIS CALC. AND TABULATED BELOW.

REPRESENTATIVE CLIP TYPES	REPRESENTED CLIP TYPES
A	-
B	J
C	-
D OR M	F, N, P, Q, R
DA OR L OR LA	-
G	-
T	E

STIFFNESSES FOR REPRESENTED CLIPS ARE TAKEN TO HAVE THE SAME STIFFNESS VALUES AS REPRESENTATIVE CLIPS.

CLIPS MAY BE -

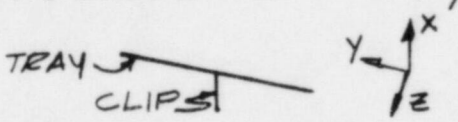
1) NON-LONGITUDINAL RESTRAINED CLIPS
   - WHICH CAN BE CLASSIFIED AS
   a) NON-RELEASED OR FRICTION CLIPS
   b) RELEASED OR FRICTIONLESS CLIPS
2) LONGITUDINAL RESTRAINED CLIPS

NON-LONGITUDINAL RESTRAINED CLIPS :-

A, C AND G CLIPS ARE NON-LONGITUDINAL RESTRAINED CLIPS.

CLIP TYPE A IS DESIGNED TO CLAMP OVER THE CABLE TRAY SIDE RAIL (SEE DET. SEC. 3.10 P1) HOWEVER IN SOME CASES WHEN THE BOLT AT THE BASE OF THE CLIP IS TIGHTENED, A FRICTION FORCE BETWEEN CABLE TRAY AND CLIP IS DEVELOPED. THE STIFFNESSES FOR FRICTION CLIPS WERE CALCULATED BY CONSIDERING ONLY THE FRICTION BETWEEN THE TRAY AND CLIPS AND NEGLECTING THE FRICTION BETWEEN THE TRAY AND SUPPORTING BEAM. THIS CONDITION CAN EXIST WHEN THE TRAY HAS LIFTED OFF THE SUPPORTING BEAM DURING AN EARTHQUAKE. THE CALCULATED STIFFNESSES WILL RESULT IN A FLEXIBLE TRAY RESTRAINT WHICH IS CONSIDERED CONSERVATIVE.

CABLE TRAY AS-BUILT WALKDOWNS HAVE NOTED SOME TYPE A CLIPS WITH GAPS BETWEEN TRAY AND THE CLIPS. FOR THIS REASON CLIP TYPE A IS ALSO ANALYSED AS FRICTIONLESS CLIP i.e. $K_Y = 0$, $K_{XX} = 0$ & $K_{ZZ} = 0$



CLIP TYPE C IS SIMILAR IN FUNCTION TO CLIP TYPE A EXCEPT THAT CLIP TYPE C FITS ONLY OVER THE BOTTOM FLANGE OF THE CABLE TRAY SIDE RAIL (SEE DET. SEC. 3.3 P1). CASES OF FRICTION AND FRICTIONLESS ARE ALSO APPLICABLE.

CLIP TYPE G IS ALSO SIMILAR IN DESIGN TO THE CLIP TYPE A EXCEPT THAT IT IS WELDED AT ITS BASE AND CAN BE ONLY A FRICTIONLESS CLIP (SEE DET. SEC 3.6G R1). CLIP TYPE G DO NOT EXERT A DOWNWARD FORCE AGAINST THE CABLE TRAY WHEN INSTALLED.

IT IS RECOGNIZED THAT FRICTION WILL NOT COMPLETELY

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RESTRAINED THE TRAY. HOWEVER DUE TO VARIOUS AS-BUILT CONDITIONS, THE PRECISE FRICTIONAL RESISTANCE COULD NOT BE DETERMINE. TO PROVIDE UPPERBOUND VALUES, IT IS ASSUMED THAT FRICTION WILL RESTRAIN THE TRAY AT ALL CONDITION. LOWERBOUND VALUES ARE PROVIDED BY THE FRICTIONLESS CLIP CASE.

LONGITUDINAL RESTRAINED CLIPS:-

B, D, M, DA, L, LA AND T ARE LONGITUDINAL RESTRAINED CLIPS. DEPENDING ON CONNECTION, FOR NON-LONGITUDINAL (FRICTION) RESTRAINED AND LONGITUDINAL RESTRAINED CLIPS, K_{xx} AND K_{yy} ARE CALCULATED FROM ADDING K_θ AND K_Δ VALUES CONSIDERING FIXED CONNECTION BETWEEN CLIP & CABLE TRAY AND APPLYING UNIT ROTATIONAL & DISPLACEMENT VALUE DUE TO UNIT ROTATIONAL OF CABLE TRAY.

STIFFNESSES:-

DEPENDING ON CLIP TYPES AND ATTACHMENT TO THE SUPPORTING MEMBER AND TRAY, THE FOLLOWING METHODS ARE USED TO CALCULATE STIFFNESSES.

1. TRANSLATIONAL STIFFNESS

$K = F/\Delta$

K = STIFFNESS

F = APPLIED FORCE

Δ = DISPLACEMENT

2. ROTATIONAL AND BENDING STIFFNESS

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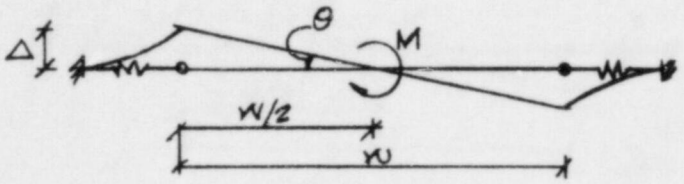
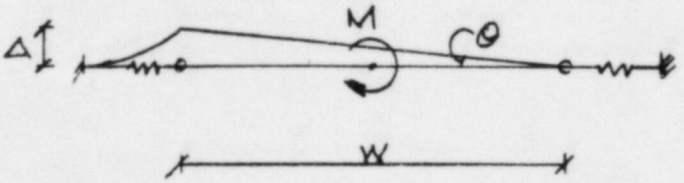
1a. PINNED END CONDITION

CASE I -



CASE II -



$K = \frac{M}{\theta}$

K = ROTATIONAL STIFFNESS

M = APPLIED MOMENT

θ = ROTATIONAL VALUE

$= \tan^{-1} 2\Delta/W$ FOR CASE I

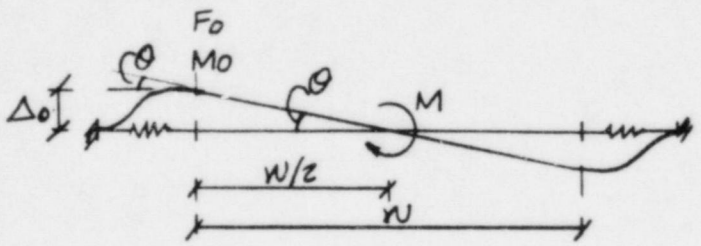
$= \tan^{-1} \Delta/W$ FOR CASE II

Δ = DISPLACEMENT

W = WIDTH OF CABLE TRAY

2b. FIXED END CONDITION

CASE I



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CABLE TRAY - CLIP ANGLE STIFFNESS

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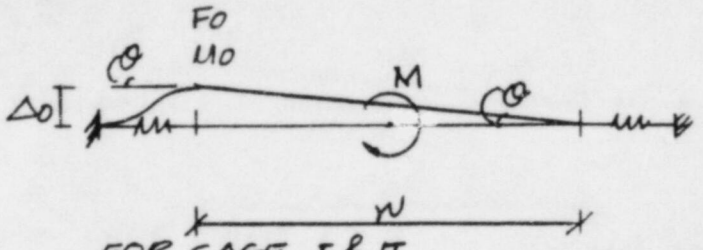
CASE II



FOR CASE I & II

$K = K\theta + K\Delta$ $\quad K\theta = \frac{4M_o}{\theta} = \frac{M}{\theta}$ $\quad K\Delta = \frac{M}{\theta} = \frac{F_o W}{\theta}$

K = ROTATIONAL STIFFNESS (TOTAL)

Kθ = STIFFNESS DUE TO ROTATION

KΔ = STIFFNESS DUE TO DISPLACEMENT

M = MOMENT FOR UNIT ROTATION

θ = UNIT ROTATION

Δ_o = DISPLACEMENT

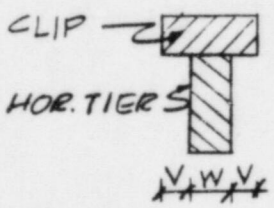
F_o = FORCE DUE TO M

M_o = MOMENT DUE TO M

W = WIDTH OF CABLE TRAY

ASSUMPTIONS :-

1. DEPENDING ON THE COMBINATION OF THE HORIZONTAL SUPPORTING TIER AND CLIP SIZE, CLIP CAN OVERLAP "V" AMOUNT TO THE SUPPORTING MEMBER AS SHOWN



SINCE V < W, IT IS ASSUMED THAT THE RATIO OF V/W IS SMALL AND THE EFFECT OF THE OVERLAP IS NEGLIGIBLE IN THE DETERMINATION OF STIFFNESS.

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CABLE TRAY-CLIP ANGLE STIFFNESS

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2. FOR CALCULATION OF STIFFNESSES, IT IS ASSUMED THAT CABLE TRAY IS RIGID.
3. IT IS ASSUMED THAT THERE IS NO SLIP FOR BOLTED CLIP CONNECTIONS.

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3.0 DETAILED CALCULATIONS

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0	[illegible]	12-4-85	[illegible]	12-4-85

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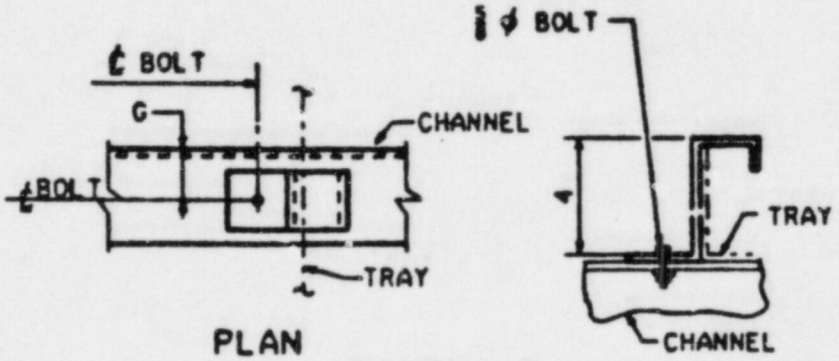
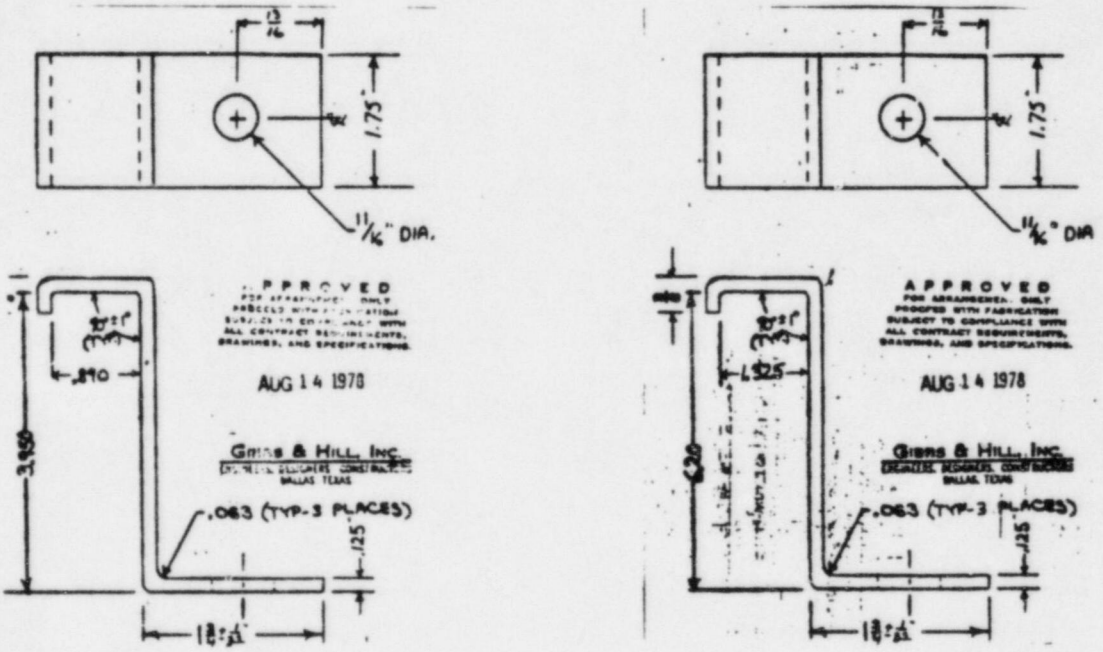


TYPE A



REF. 5a FOR TUGCO DRAWINGS NUMBER

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					CABLE TRAY-CLIP ANGE STIFFNESS
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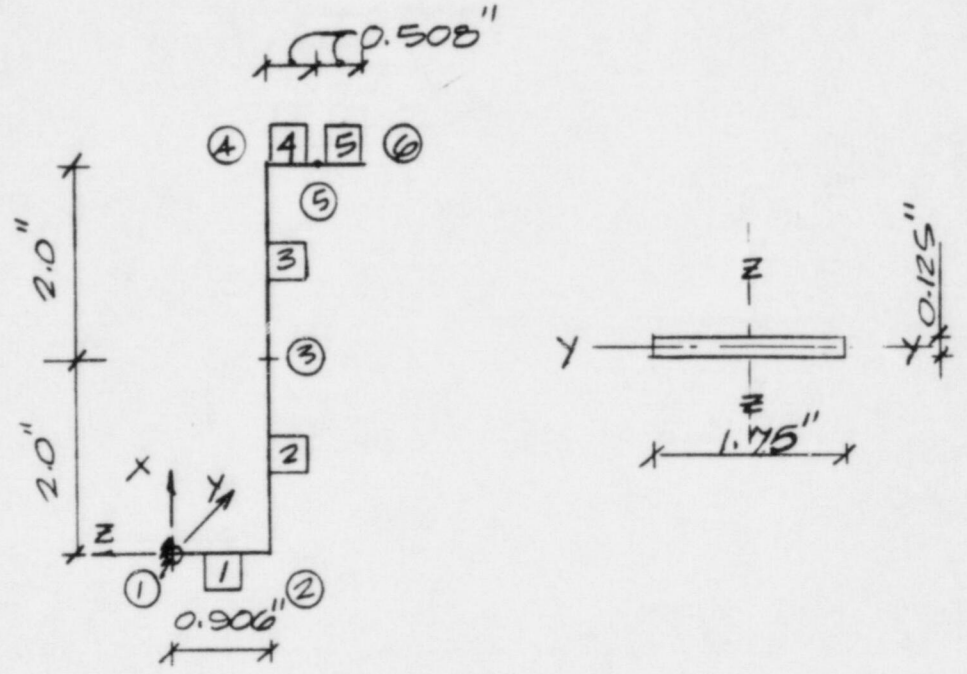
STIFFNESSES FOR CLIP TYPE "A" - 4" HIGH

GTSTRUDL MODEL - W/O 1/4" OFFSET



THICKNESS $t = \frac{1}{8}" = 0.125"$ $b/d = 1.75/0.125 = 14.0$ $\beta = 0.333$

$A_X = A_Y = A_Z = 1.75 \times 0.125 = 0.219\ IN^2$

$I_{XY} = 0.333 \times 1.75 \times 0.125^2 = 1.138 \times 10^{-3}\ IN^4$

$I_{YY} = 1.75 \times 0.125^3/12 = 2.848 \times 10^{-4}\ IN^4$

$I_{ZZ} = 0.125 \times 1.75^3/12 = 0.0558\ IN^4$

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CABLE TRAY - CLIP ANGLE STIFFNESS

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APPLIED LOADS:

1) W/O 1/4" OFFSET

a) FRICTION CLIP

K_x & K_{yy}: F_x = 1000 LBS NODE 5

K_{xy}:

FOR ROTATIONAL $\alpha = 1°$ $\Delta y = \frac{w}{2} \tan 1°$ W = CABLE TRAY WIDTH

AT FIXED END NODE 5

CABLE TRAY SIZE	Δy
36 x 4	0.3141912"
30 x 4	0.2618260"
24 x 4	0.2094608"
18 x 4	0.1570956"
12 x 4	0.1047304"
6 x 4	0.0523652"

K_y : F_y = 1000 LBS NODE 5

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Kz : Fz = 1000 LBS NODE 2
(ACTS ON 1-CLIP. NEGLECT FRICTION FOR OTHER CLIP)

Kzz : Mz = 1000 LBS-IN NODE 4
(Mz MOMENT WILL HAVE TENSION FORCE ON ONE EDGE OF CLIP AND ULTIMATELY WILL HAVE TENSION AND COMPRESION @ NODE4 ON VERTICAL SIDE OF CLIP RESULTING INTO M @ NODE4)

b) FRICTIONLESS CLIP

Kx : FX = 1000 LBS NODE 5

Kxx : NO RESISTING CAPACITY

Ky : NO RESISTING CAPACITY

Kyy : SAME AS FRICTION CLIP

Kz : Fz = 1000 LBS (ACTS ON 1-CLIP) NODE 2

Kzz : NO RESISTING CAPACITY

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CABLE TRAY - CLIP ANGLE STIFFNESS
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STIFFNESSES FOR CLIP TYPE "A" - W/O 1/4" OFFSET

ATTACHMENT-1 (1&2) 4" HIGH

FRICTION CLIP -

Kx : $\Delta xs = 0.23600326''$

Kx FOR 2-CLIPS $= 2/\Delta xs$ $= 8.473$ K/IN ($\simeq 8.50$)

Kxx : $Kxx = KxxO + KxxD$ $KxxO = \frac{4M}{\theta} = \frac{4 \times 57.296}{1^\circ} M = 229.184M$ x

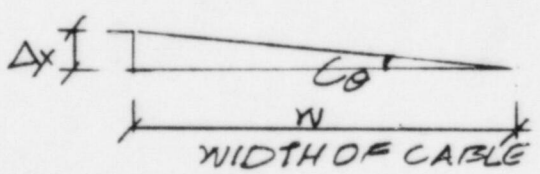
$KxxD = \frac{WEy}{\theta} = 57.296WFy$

SIZE OF CABLE TRAY WxH	Mθs K-IN	KxxO K-IN/RAD	Fys K	KxxD K-IN/RAD	≃ Kxx K-IN/RAD
36 x 4	0.336	77	0.776	1,600	1.6E3
30 x 4	0.268	61	0.643	1,105	1.2E3
24 x 4	0.200	46	0.510	701	750
18 x 4	0.133	30	0.377	389	420
12 x 4	0.065	15	0.244	168	180
6 x 4	0.002	1	0.111	39	40

Ky : $\Delta ys = 1.169973''$

Ky FOR 2-CLIPS $= 2/\Delta ys$ $= 1.709$ K/IN ($\simeq 1.70$)

Kyy :



FOR $Fx = 1000\#$ $\Delta xs = 0.23600326''$ $M = 1000 \times W\ \#''$

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CABLE TRAY - CLIP ANGLE STIFFNESS

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SIZE OF CABLE TRAY	$\theta = \tan^{-1}\frac{\Delta x 5}{W}$	$K_{YY} = \frac{57.296 \times M}{\theta}$ K-IN/RAD
36×4	0.375621617	5,490.867 ≙ 5.5E3
30×4	0.450779759	3,813.126 ≙ 3.8E3
24×4	0.563468159	2,440.429 ≙ 2.4E3
18×4	0.751272042	1,372.776 ≙ 1.4E3
12×4	1.126827348	610.1666 ≙ 610.0
6×4	2.252783689	152.600 ≙ 150.0

K_Z: $\Delta z_2 = 0.0001402''$ (NEGLECT FRICTION ON OTHER CLIP)

K_Z FOR 1-CLIP $= 1/\Delta z_3$ $= 7,132.668$ K/IN (≙ 7.1E3)

K_{ZZ}: $\theta z_3 = 3.9981319°$

K_{ZZ} FOR 2-CLIPS $= \frac{2 \times 57.296}{\theta z_3} = 28.6661$ K-IN/RAD (≙ 29.0)

FRICTIONLESS CLIP -

K_X:	SAME AS FRICTION CLIP	= 8.50	K/IN
K_{XX}:	NO RESISTING CAPACITY	= 0.0	K-IN/RAD
K_Y:	NO RESISTING CAPACITY	= 0.0	K/IN
K_{YY}:	SAME AS FRICTION CLIP - 36×4	= 5.5E3	K-IN/RAD
	30×4	= 3.8E3	"
	24×4	= 2.4E3	"
	18×4	= 1.4E3	"
	12×4	= 610.0	"
	6×4	= 150.0	"

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KZ : $\Delta z_2 = 0.0001402''$ FORCE ACTING ON ONE CLIP

KZ = $1/\Delta zz$ $= 7,132.668$ K/IN
$(\simeq 7.1E3)$

KZZ : NO RESISTING CAPACITY $= 0.0$ K-IN/RAD

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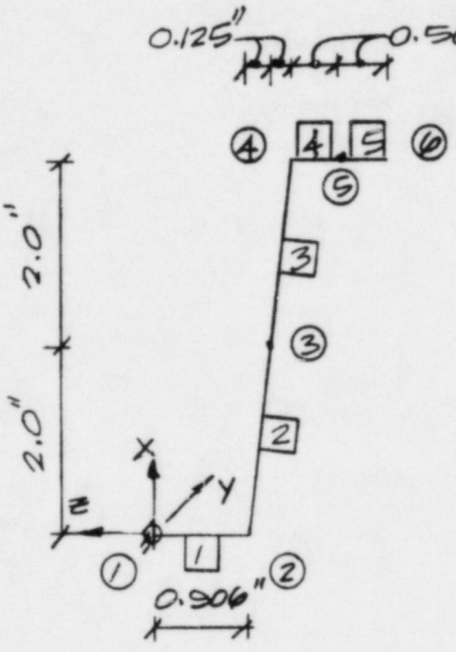
GTSTRUDL MODEL - W/ 4" OFFSET



PROPERTIES ARE SAME AS W/O 4" OFFSET

APPLIED LOADS:

Kx & Kyy:	FX = 1000 LBS	NODE 5
Ky :	FY = 1000 LBS	NODE 5
Kz :	FZ = 1000 LBS	NODE 4
Kzz :	MZ = 1000 LBS-IN	NODE 4

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1	[illegible]	12-26	DMR	1-28-86		

STIFFNESSES FOR CLIP TYPE "A" - W/ 1/4" OFFSET - 4" HIGH (FRICTION TYPE)

ATTACHMENT-1 (3)

Kx : $\Delta X_5 = 0.36552624$"

Kx FOR 2-CLIPS = $2/\Delta X_5$ = 10.952 K/IN

Ky : $\Delta Y_5 = 1.1705131$"

Ky FOR 2-CLIPS = $2/\Delta Y_5$ = 1.708 K/IN

Kyy : $F_x = 1000$# $M = 1000 \times W$ #" $\Delta X_5 = 0.36552624$"

CABLE TRAY SIZE	$\theta = \tan^{-1} \frac{\Delta X_5}{W}$	$K_{yy} = \frac{57.296 \times M}{\theta}$ K-IN/RAD
36 X 4	0.5813313217	3,548.270
30 X 4	0.69756533	2,404.113
24 X 4	0.87193243	1,577.076
18 X 4	1.1625506782	887.159
12 X 4	1.7434601186	394.360
6 X 4	3.4836099684	98.681

Kzz : $\theta_{Z4} = 4.0642824°$

Kzz FOR 2 CLIPS = $2/\Delta Z_4$ = 28.191 K-IN/RAD

Kz : $\Delta Z_4 = 4.2682304$"

Kz FOR 1-CLIP = $1/\Delta Z_4$ = 0.240 K/IN

Kz VALUE OF THIS CLIP REMAINS SAME TILL CLIP HAS FRICTION AND DEFLECTION IS UPTO 1/4". AFTER THAT CABLE TRAY Fz FORCE ACTS AT NODE POINT 2, SIMILAR WAY

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1	[illegible]	1-20-86	JMR	1/28/86

TUGCO

CABLE TRAY - CLIP ANGLE STIFFNESS

IMPELL CORPORATION

JOB NO 0210-040

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AS IN THE CASE OF CLIP TYPE "A" W/O ¼" OFFSET. AND K_z VALUE CHANGES TO 7.1E3 K/IN. OTHER CLIP HAS STIFFNESS VALUE VERY HIGH UP TO FRICTION CAPACITY, AS Δz IS VERY LOW BECAUSE OF CABLE TRAY SIDE RAIL RESISTS THE Δz DISPLACEMENT AT NODE 4. NEGLECTING FRICTION ON OTHER CLIP K_z VALUE CAN BE TAKEN THE SAME AS IN THE CASE OF CLIP TYPE "A" W/O ¼" OFFSET.

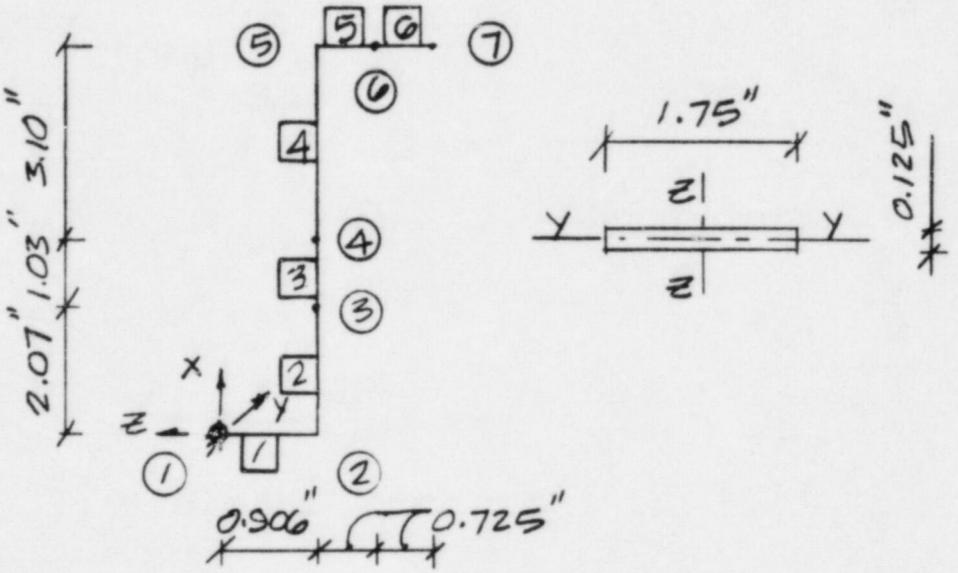
$K_z = 7.1E3$ K/IN

CONCLUSION: AS K_{yy} IS DERIVED FROM ΔxS AND K_x, K_y, K_z AND K_{zz} VALUES ARE PRACTICALLY SAME AS IN CASE OF CLIP TYPE "A" W/O ¼" OFFSET, USE ALL K_x, K_y, K_z, K_{xx}, K_{yy} & K_{zz} VALUES OF CLIP TYPE "A" W/O ¼" OFFSET FOR THIS CLIP OR ½" OFFSET DOES NOT MAKE ANY DIFFERENCE IN CLIP TYPE "A".

REV	BY	DATE	CHECKED	DATE	TUGCO CABLE TRAY-CLIP ANGLE STIFFNESS IMPELL CORPORATION JOB NO 0210-040 CALC NO M-10	PAGE 10 OF 10
1	[signature]	1-20-88	[signature]	1-28-88		

STIFFNESSES FOR CLIP TYPE "A" – 6" HIGH

GTSTRUDL MODEL -



THICKNESS $t = \frac{1}{8}" = 0.125$ $b/d = 1.75/0.125 = 14.0$ $\beta = 0.333$

$A_x = A_y = A_z = 0.125 \times 1.75 = 0.219\ in^2$

$I_{xx} = 0.333 \times 1.75 \times 0.125^3 = 1.138 \times 10^{-3}\ in^4$

$I_{yy} = 1.75 \times 0.125^3/12 = 2.848 \times 10^{-4}\ in^4$

$I_{zz} = 0.125 \times 1.75^3/12 = 0.0558\ in^4$

REV	BY	DATE	CHECKED	DATE
1	[illegible]	1.20.88	[illegible]	1.28.88

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APPLIED LOADS:

K_{XX} & K_{YY}: $F_X = 1000$ LBS — NODE 6

K_{XX}: FOR ROTATIONAL $\theta = 1°$ $\Delta Y = \frac{W}{2} \tan 1°$ — NODE 5 - FIXED END

W = CABLE TRAY WIDTH

CABLE TRAY SIZE	ΔY
36X6	0.3141912"
30X6	0.2618226"
24X6	0.2094608"
18X6	0.1570956"

K_Y: $F_Y = 1000$ LBS — NODE 6

K_Z: $F_Z = 1000$ LBS — FRICTIONLESS CLIP - NODE 2; FRICTION CLIP - NODE 2

K_{ZZ}: $M_Z = 1000$ LBS — NODE 5

REV	BY	DATE	CHECKED	DATE
1	[illegible]	1-20-86	[illegible]	1-28-86

TUGCO

CABLE TRAY - CLIP ANGLE STIFFNESS

IMPELL CORPORATION

JOB NO 0210-040

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STIFFNESSES FOR CLIP TYPE "A" - 6" HIGH

ATTACHMENT-2 (1&2)

FRICTION CLIP -

K_X : $\Delta X6 = 0.561224"$

K_X FOR 2-CLIPS $= 2/\Delta X6$ $= 3.564$ K/IN ($\simeq 3.60$)

K_{XX} : $K_{XX} = K_{XX\theta} + K_{XX\Delta}$

$K_{XX\theta} = \frac{4M}{\theta} = 4 \times \frac{57.296M}{1^\circ} = 229.184M$

$K_{XX\Delta} = \frac{WF_Y}{\theta} = 57.296\,WF_Y$

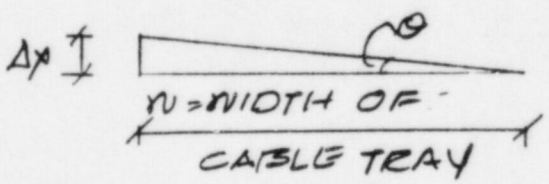
SIZE OF CABLE TRAY W x H	M@6 K-IN	$K_{XX\theta}$ K-IN/RAD	F_{Y5} K	$K_{XX\Delta}$ K-IN/RAD	$\simeq K_{XX}$ K-IN/RAD
36x6	0.150	34	0.258	532	570
30x6	0.117	27	0.214	367	390
24x6	0.085	20	0.169	232	250
18x6	0.052	12	0.124	128	140

NEGLECT $K_{XX\theta}$ SMALL VALUE FOR K_{XX}

K_Y : $\Delta Y6 = 2.8833859"$

K_Y FOR 2-CLIPS $= 2/\Delta Y6$ $= 0.694$ K/IN ($\simeq 0.70$)

K_{YY} :



FOR $F_X = 1000^\#$ $\Delta X6 = 0.561224"$ $M = 1000 \times W^{\#"}$

REV	BY	DATE	CHECKED	DATE
1	[illegible]	1.20.86	[illegible]	1.21.86

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CABLE TRAY - CLIP ANGLE STIFFNESS

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SIZE OF CABLE TRAY	$\theta = \tan^{-1}\frac{\Delta x_6}{W}$	$K_{yy} = \frac{57.296 \times M}{\theta}$ K-IN/RAD
36×6	0.8931433387	2,309.434 ≃ 2.3E3
30×6	1.071733872	1,603.840 ≃ 1.6E3
24×6	1.339577947	1,026.519 ≃ 1.0E3
18×6	1.785582928	577.499 ≃ 580.0

KZ : $\Delta z_2 = 0.0001402$" (NEGLECT FRICTION ON OTHER CLIP)

KZ FOR 1-CLIPS = $1/\Delta z_6$ = 7,132.668 K/IN (≃ 7.1E3)

KZZ: $\theta z_6 = 4.074707°$

KZZ FOR 2-CLIPS = $\frac{2 \times 57.296}{\theta z_6}$ = 28.123 K-IN/RAD (≃ 29.0)

FRICTIONLESS CLIP –

KX : SAME AS FRICTION CLIP = 3.60 K/IN

KXX : NO RESISTING CAPACITY = 0.0 K-IN/RAD

KY : NO RESISTING CAPACITY = 0.0 K/IN

KYY : SAME AS FRICTION CLIP- 36×6 = 2.3E3 K-IN/RAD
30×6 = 1.6E3 "
24×6 = 1.0E3 "
18×6 = 580.0 "

KZ : $\Delta z_2 = 0.0001402$" FORCE ACTING ON ONE CLIP

REV	BY	DATE	CHECKED	DATE
1	[illegible]	1.20.86	GMR	1.28.86

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CABLE TRAY - CLIP ANGLE STIFFNESS

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$K_z = 1/\Delta_{z2}$ = 7,132.668K/IN (≙ 7.1E3)

K_{zz}: NO RESISTING CAPACITY = 0.0 K-IN/RAD

REV	BY	DATE	CHECKED	DATE
1	[illegible]	1.20.86	[illegible]	1.28.86

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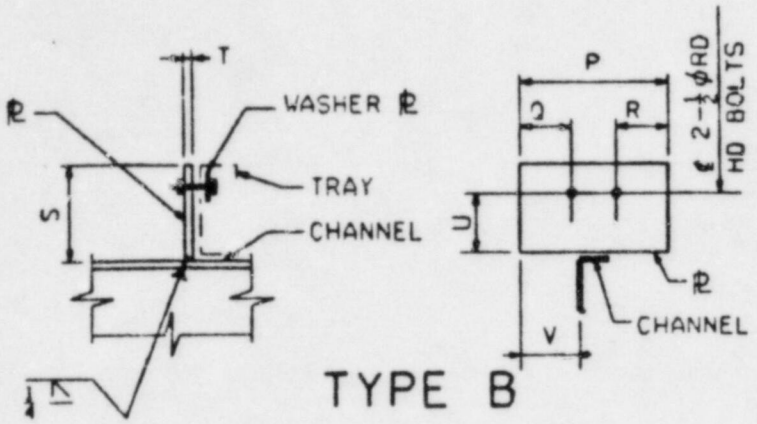
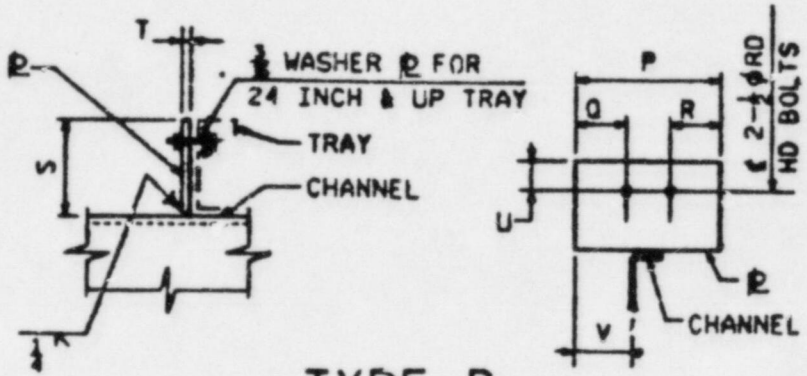


TYPE B

○	4	½	1	4	½	2	
CLAMP NO.	P	Q	R	S	T	U	V
CLAMP TYPE B							



TYPE B

PLAN

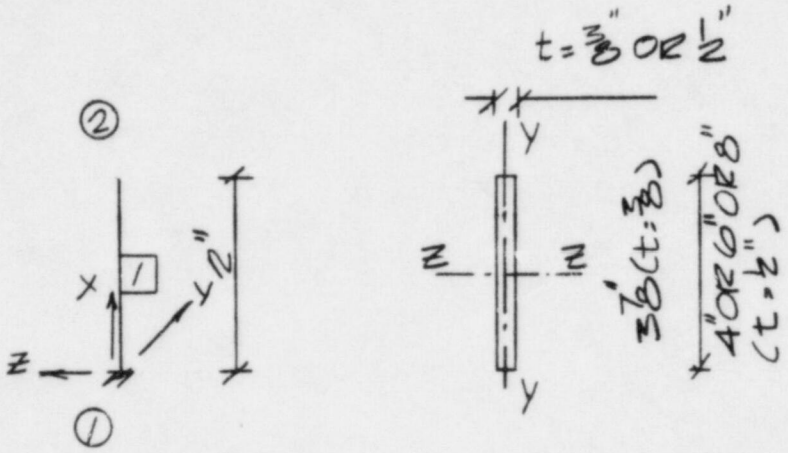
②	4	1	1	4	½	2	-
①	4	1	1	4	½	2	-
CLAMP NO.	P	Q	R	S	T	U	V
CLAMP TYPE B							

REF. 56 - FOR TUSCO DRAWINGS NUMBER

REV	BY	DATE	CHECKED	DATE	TUSCO CABLE TRAY - CLIP ANGLE STIFFNESS IMPELL CORPORATION JOB NO 0210-0AD CALC NO M-10	PAGE 1 OF 7
0	[illegible]	12-3-85	DC	12-4-85		

STIFFNESSES FOR CLIP TYPE "B"

GT STRUDL MODEL -



FOR $t = \frac{3}{8}" = 0.375"$

$A_X = A_Y = A_Z = 0.375 \times 3.875 = 1.45 \ in^2$

$b/d = 3.875/0.375 = 10.33 \quad \beta = 0.313$

$I_{XX} = 0.313 \times 0.375^3 \times 3.875 = 0.064 \ in^4$

$I_{YY} = 3.875 \times 0.375^3/12 = 0.0170 \ in^4$

$I_{ZZ} = 0.375 \times 3.875^3/12 = 1.8183 \ in^4$

FOR $t = \frac{1}{2}" = 0.5"$ (4" STIFFNGSSES WILL BE CONSERVATIVE FOR 6" OR 8"

$A_X = A_Y = A_Z = 0.5 \times 4 = 2.0 \ in^2$

$b/d = 4/0.5 = 8. \quad \beta = 0.307$

$I_{XX} = 0.307 \times 0.5^3 \times 4 = 0.1535 \ in^4$

$I_{YY} = 4 \times 0.5^3/12 = 0.04167 \ in^4$

$I_{ZZ} = 0.5 \times 4^3/12 = 2.0667 \ in^4$

REV	BY	DATE	CHECKED	DATE
1	[illegible]	1-14-86	GMK	1-28-86

TUGCO
CABLE TRAY - CLIP ANGLE STIFFNGSS
IMPELL CORPORATION
JOB NO. 0210-040
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APPLIED LOADS:

Kx : $F_X = 1000$ LBS — NODE 2

Kxx : FOR ROTATIONAL $\theta = 1°$ $\Delta y = \frac{W}{2} \tan 1°$

W = WIDTH OF CABLE TRAY

AT FIXED END NODE 2

CABLE TRAY SIZE	Δy
36 x 6 OR 4	0.3141912"
30 x 6 OR 4	0.2618216"
24 x 6 OR 4	0.2094608"
18 x 6 OR 4	0.1570956"
12 x 4	0.1047304"
6 x 4	0.0523652"

Ky : $F_Y = 1000$ LBS — NODE 2

Kz : $F_Z = 1000$ LBS — NODE 1

Kzz : $M_Z = 1000$ LBS-IN — NODE 2

REV	BY	DATE	CHECKED	DATE
1	[illegible]	1.20.88	GMR	1.21.88

TUGCO

CABLE TRAY - CLIP ANGLE STIFFNESS

IMPELL CORPORATION

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STIFFNESSES FOR CLIP TYPE "B" - $\frac{3}{8}$"

ATTACHMENT-3 (1&2)

Kx : $\Delta x2 = 0.0000468$"

Kx FOR 2-CLIPS = $2/\Delta x2$ = 42,735.043 K/IN
≙ 4.3E4 K/IN

Kxy : $Kxx = Kxx\theta + Kxy\Delta$ $\quad Kxx\theta = \frac{4M}{\theta} = 4 \times \frac{57.296M}{1^\circ} = 229.184M$

$Kxy\Delta = \frac{WFy}{\theta} = 57.296WFy$

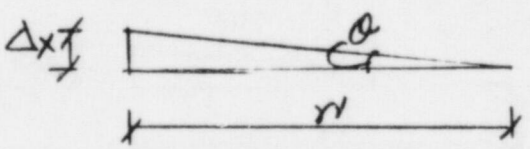
SIZE OF CABLE TRAY W x H	MθZ K-IN	KxxΘ K-IN/RAD	Fy2 K	KxxΔ K-IN/RAD	Kxx K-IN/RAD
36 x 6 OR 4	6,590.3633	1,510	2,430	5,012,254	5.012E6
30 x 6 OR 4	↓	↓	2,025	3,480,732	3.481E6
24 x 6 OR 4			1,620	2,227,668	2.229E6
18 x 6 OR 4			1,215	1,253,063	1.255E6
12 x 4			810	556,917	5.584E5
6 x 4			405	139,229	1.407E5

Kxy = 1.40E5 K-IN/RAD (MIN.)

Ky : $\Delta y2 = 0.000160666$"

Ky FOR 2-CLIPS = $2/\Delta y2$ = 12,004.802 K/IN
≙ 1.2E4 K/IN

Kyy :



FOR Fx = 1000# $\Delta x2 = 0.0000468$" $M = 1000 \times W$ #"

REV	BY	DATE	CHECKED	DATE
1	[signature]	1.20.86	jmr	1.28.86

TUGCO

CABLE TRAY - CLIP ANGLE STIFFNESS

IMPELL CORPORATION

JOB NO 0210-040

CALC NO M-10

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SIZE OF CABLE TRAY	$\theta = \tan^{-1}\frac{\Delta x z}{N}$	$K_{YY} = \frac{57.296 \times M}{\theta}$ K-IN/RAD
36 X 6 OR 4	0.000074484	27,692,605
30 X 6 OR 4	0.000089381	19,230,843
24 X 6 OR 4	0.000111726	12,307,739
18 X 6 OR 4	0.000148969	6,923,103
12 X 4	0.000223453	3,076,934
6 X 4	0.000446907	769,233

$\therefore K_{YY} = 7.7E5$ K-IN/RAD (MIN)

K_Z : F_Z FORCE IS ACTING AT THE BASE OF THE CLIP SIMILAR TO THE CLIP TYPE A AND CAN BE ASSUMED CONSERVATIVELY $K_Z = 7.1E3$ K/IN

K_{ZZ} : $\theta_{ZZ} = 0.0021363°$

K_{ZZ} FOR 2-CLIPS $= \frac{2 \times 57.296}{\theta_{ZZ}} = 53,640.406$ K-IN/RAD $\simeq 5.4E4$ K-IN/RAD

REV	BY	DATE	CHECKED	DATE
1	[signature]	1-20-86	[illegible]	1-28-86

TUGCO

CABLE TRAY-CLIP ANGLE STIFFNESS

IMPELL CORPORATION

JOB NO 0210-040

CALC NO M-10

STIFFNESSES FOR CLIP TYPE "B" - #1/2"

ATTACHMENT-3 (3 & 4)

K_X : $\Delta X2 = 0.0000339"$

K_X FOR 2-CLIPS $= {}^{2}/_{\Delta X2}$ $= 58,997.050$ K/IN

$\div 5.9$ EA K/IN

K_{XY} : $K_{XX} = K_{XX\theta} + K_{XX\Delta}$ $K_{XX\theta} = \frac{4M}{\theta} = 4 \times \frac{57.296M}{1^\circ} = 229.184M$

$K_{XX\Delta} = \frac{NF_Y}{\theta} = 57.296NF_Y$

SIZE OF CABLE TRAY WXH	$M\theta_2$ K-IN	$K_{XX\theta}$ K-IN/RAD	F_{Y2} K	$K_{XX\Delta}$ K-IN/RAD	K_{XY} K-IN/RAD
36X60X4	15,806.574	3,622	3,370	6,951,150	6.955E6
30X60X4			2,809	4,828,333	4.832 E6
24X60X4			2,247	3,089,859	3.093E6
18X60X4			1,682	1,734,693	1.738E6
12X4			1,123	772,120	7.757E5
6X4			562	193,202	1.938E5

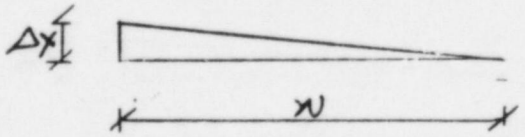
$K_{XY} = 2.0E5$ K-IN/RAD (MIN)

K_Y : $\Delta Y2 = 0.0001186"$

K_Y FOR 2-CLIPS $= {}^{2}/_{\Delta Y2}$ $= 16,863.406$ K/IN

$\div 1.7$ EA K/IN

K_{YY} :



FOR $F_X = 1000\#$ $\Delta Y2 = 0.0000468"$ $M = 1000 \times N\ \#"$

REV	BY	DATE	CHECKED	DATE	
1	[illegible]	1.20.86	MR	1.28.86	

TUGCO

CABLE TRAY - CLIP ANGLE STIFFNESS

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SIZE OF CABLE TRAY	$\theta = \tan^{-1}\frac{\Delta x_2}{w}$	$K_{yy} = \frac{57.296 \times M}{\theta}$ K-IN/RAD
36 X 6 OR 4	0.000053953	38,230,235
30 X 6 OR 4	0.000064744	26,548,774
24 X 6 OR 4	0.00008093	16,991,215
18 X 6 OR 4	0.000107907	9,557,558
12 X 4	0.000161860	4,247,803
6 X 4	0.000323721	1,061,950

K_{yy} = 1.166 K-IN/RAD (MIN)

K_z : F_z FORCE IS ACTING AT THE BASE OF THE CLIP SIMILAR TO THE CLIP TYPE A AND CAN BE ASSUMG CONSERVATIVELY K_z = 7.163 K/IN

K_{zz} : $\theta_{z2} = 0.001456^{0}$

K_{zz} FOR 2-CLIPS $= \frac{2 \times 57.296}{\theta_{z2}}$ = 78,665.477 K-IN/RAD
≅ 7.9 E4 K-IN/RAD

REV	BY	DATE	CHECKED	DATE
1	[illegible]	1-20-86	bmk	1-28-86

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CABLE TRAY-CLIP ANGLE STIFFNESS

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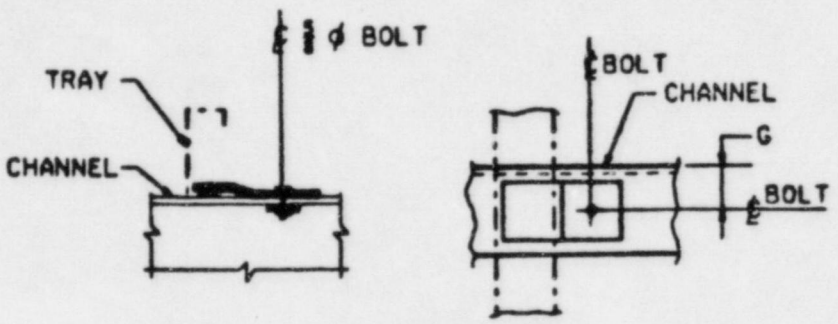


TYPE C

FOR DIMENSIONS SEE ATTACHMENT 10 (REF. 7) ⚠
REF. 5C FOR TUGCO DRAWINGS NUMBER

REV	BY	DATE	CHECKED	DATE
1	[signature]	1-20-86	[illegible]	1-28-86
0	[signature]	12-3-85	MJC	12-4-85

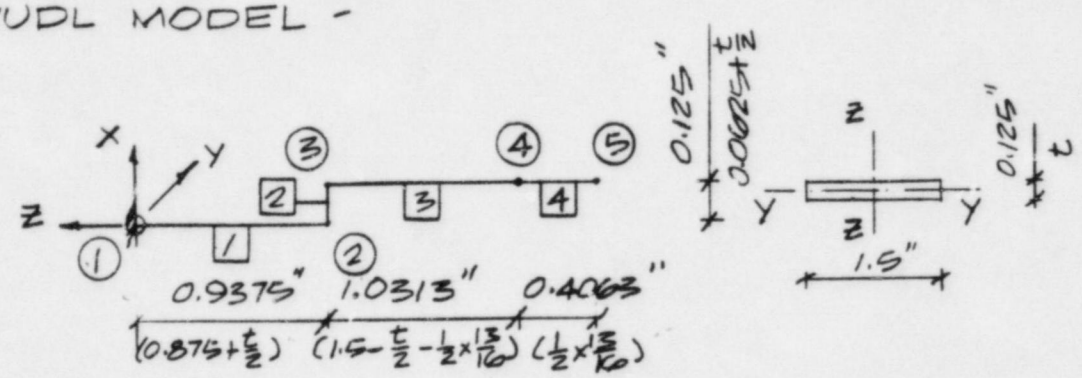
TUGCO
CABLE TRAY-CLIP ANGLE STIFFNESS

IMPELL CORPORATION

JOB NO 0210-040-
CALC NO M-10

STIFFNESSES FOR CLIP TYPE "C" - 1/8" THICK

GTSTRUDL MODEL -



THICKNESS $t = 1/8" = 0.125"$ $b/t = 20$ $\beta = 0.333$

$A_x = A_y = A_z = 1.5 \times 0.125 = 0.1875\ in^2$

$I_{xx} = 1.5 \times 0.125^3 \times 0.333 = 9.7559 \times 10^{-4}\ in^4$

$I_{yy} = \frac{1.5 \times 0.125^3}{12} = 2.4414 \times 10^{-4}\ in^4$

$I_{zz} = \frac{0.125 \times 1.5^3}{12} = 0.03516\ in^4$

REV	BY	DATE	CHECKED	DATE
1	[signature]	12-31-85	DC	12-31-85

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APPLIED LOADS:

K_x & K_{yy}: $F_x = 1000/2 = 500$ LBS — NODE 4

K_{xx}: FOR ROTATIONAL $\theta = 1°$ $\Delta y = (W - 13/16)\frac{\tan 1°}{2}$

W = WIDTH OF CABLE TRAY

AT FIXED END NODE 4 —

CABLE TRAY SIZE	Δy
36 x 6 OR 4	0.3071"
30 x 6 OR 4	0.254735"
24 x 6 OR 4	0.202370"
18 x 6 OR 4	0.150005"
12 x 4	0.097639"
6 x 4	0.045274"

K_y: $F_y = 1000/2 = 500$ LBS — NODE 4

K_z: $F_z = 1000$ LBS — NODE 5

K_{zz}: $M_z = 1000/2 = 500$ LBS-IN — NODE 4

STIFFNESSES FOR CLIP TYPE "C" - 1/8" THICK
ATTACHMENT-4 (1&2)

FRICTION CLIP -

KX : $\Delta X4 = 0.1863934"$

KX FOR 2-CLIPS = $1/\Delta X$ = 5.365 K/IN
≅ 5.40 K/IN

KXX : $KXX = KXX\theta + KXX\Delta$ $KXX\theta = \frac{4M}{\theta} = 4 \times \frac{57.296}{1^\circ} M = 229.184\,M$

$KXX\Delta = \frac{WFY}{\theta} = 57.296\,WFy$

SIZE OF CABLE TRAY WXH	M@4 K-IN	KXXθ K-IN/RAD	FY4 K	KXXΔ K-IN/RAD	KXX K-IN/RAD
36 X 6 OR 4	131.514	30,140	130	168,145	2.983E5
30 X 6 OR 4	107.455	24,627	106	182,201	2.068E5
24 X 6 OR 4	83.396	19,113	83	114,133	1.332E5
18 X 6 OR 4	59.337	13,599	59	60,848	7.448E4
12 X 4	35.277	8,085	36	24,751	3.284E4
6 X 4	20.801	4,767	20	6,875	1.164E4

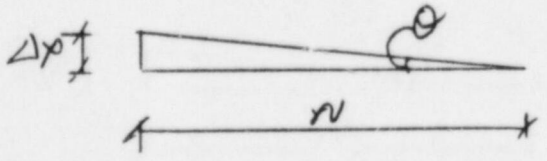
KXX = 1.2E4 K-IN/RAD (MIN.)

KY : $\Delta Y4 = 0.0081076"$

KY FOR 2-CLIPS = $1/\Delta Y4$ = 123.341 K/IN
≅ 120.0 K/IN

KYY : $FX = 2 \times 0.5 = 1.0K$ $\Delta X4 = 2 \times 0.1863934" = 0.3727868"$

$M = 1.0 \times N$ K"



REV	BY	DATE	CHECKED	DATE
1	[illegible]	1-20-86	GMK	1-28-86

TUGCO
CABLE TRAY - CLIP ANGLE STIFFNESS

IMPELL CORPORATION

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SIZE OF CABLE TRAY	$\theta = \tan^{-1}\frac{\Delta X2}{(W-\frac{3}{16})}$	$K_{YY} = \frac{57.296 \times M}{\theta}$ K-IN/RAD
36 X 6 OR 4	0.6069857 54	3,398.195 ≙ 3.4E3
30 X 6 OR 4	0.731749858	2,348.999 ≙ 2.3E3
24 X 6 OR 4	0.9210682602	1,492.945 ≙ 1.5E3
18 X 6 OR 4	1.242517057	830.031 ≙ 830.0
12 X 4	1.908488807	360.260 ≙ 360.0
6 X 4	4.111035298	83.637 ≙ 80.0

K_Z : $\Delta Z4 = 0.002538$"

K_{ZZ} FOR 2-CLIPS $= 2/\Delta Z4$ = 788.022 K/IN ≙ 790.0 K/IN

K_{ZZ} : $\theta Z4 = 4.3008168°$

K_{ZZ} FOR 2-CLIPS $= \frac{57.296}{\theta Z4}$ = 11.691 K-IN/RAD ≙ 12.0 K-IN/RAD

FRICTIONLESS CLIP -

K_X : SAME AS FRICTION CLIP = 5.4 K/IN

K_{XX} : NO RESISTING CAPACITY = 0.0 K-IN/RAD

K_Y : NO RESISTING CAPACITY = 0.0 K/IN

K_{YY} : SAME AS FRICTION CLIP- 36X60R4 = 3.4E3 K-IN/RAD
30X60R4 = 2.3E3 "
24X60R4 = 1.5E3 "
18X60R4 = 830.0 "
12X4 = 360.0 "
6X4 = 80.0 "

REV	BY	DATE	CHECKED	DATE
1	[illegible]	1-20-86	GMR	1-28-86

TUGCO
CABLE TRAY - CLIP ANGLE STIFFNESS
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Kz : $\Delta z_5 = 0.002614''$ (ONE CLIP RESISTS THE FORCE)

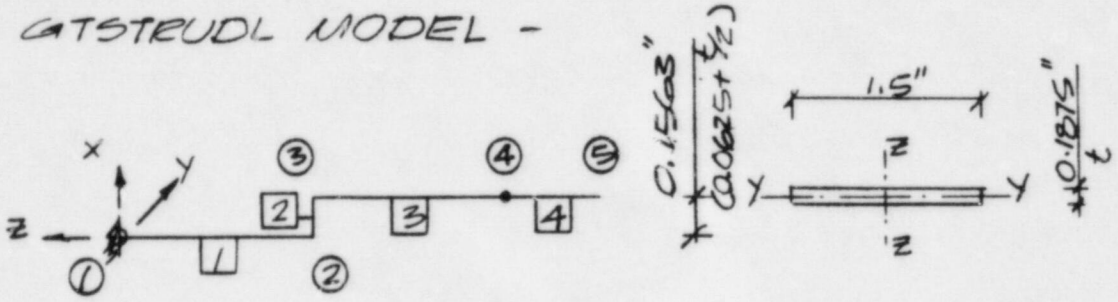
$K_z = 1/\Delta z_5$ $= 382.930$ K/IN
$\rightarrow 380.0$ K/IN

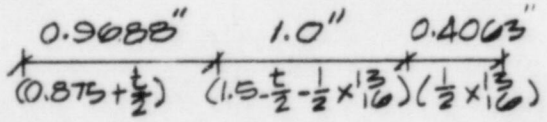
Kzz : NO RESISTING CAPACITY $= 0.0$ K-IN/RAD

					TUGCO		
					CABLE TRAY - CLIP ANGLE STIFFNESS		
					JOB NO 0210-040		PAGE 6
1	[signature]	1.20.86	GMR	1.28.86	IMPELL CORPORATION	CALC NO M-10	OF 10
REV	BY	DATE	CHECKED	DATE			

STIFFNESSES FOR CLIP TYPE "C" - 3/16" THICK

GTSTRUDL MODEL -





THICKNESS $t = 3/16" = 0.1875"$ $b/t = 8$ $\beta = 0.307$

$A_x = A_y = A_z = 1.5 \times 0.1875 = 0.2812\ IN^2$

$I_{xx} = 1.5 \times 0.1875^3 \times 0.307 = 0.00304\ IN^4$

$I_{yy} = 1.5 \times 0.1875^3/12 = 0.000824\ IN^4$

$I_{zz} = 0.1875 \times 1.5^3/12 = 0.0527\ IN^4$

FORCES - SAME AS CLIP TYPE "C" - 1/8" THICK

REV	BY	DATE	CHECKED	DATE
1	[illegible]	1-20-86	GMK	1-28-86

TUGCO

CABLE TRAY-CLIP ANGLE STIFFNESS

IMPELL CORPORATION

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STIFFNGSSES FOR CLIP TYPE "C" - 3/16" THICK

ATTACHMENT-4 (3&4)

FRICTION CLIP -

KX : $\Delta X4 = 0.0558455"$

KX FOR 2-CLIPS = $1/\Delta X4$ $= 17.907$ K/IN
$\simeq 18.0$ K/IN

KXX : $KXX = KXX\theta + KXX\Delta$ $KXX\theta = \frac{4M}{\theta} = \frac{4 \times 57.296M}{1^\circ} = 229.184M$

$KXX\Delta = \frac{WFy}{\theta} = 57.296\,WFy$

SIZE OF CABLE TRAY WXH	MθA K-IN	KxxΘ K-IN/RAD	FyA K	KxxΔ K-IN/RAD	KxY K-IN/RAD
36X60R4	205.367	47,066	209	431,095	4.791E5
30X60R4	167.686	38,430	171	293,928	3.324E5
24X60R4	130.004	29,794	133	184,263	2.141 E5
18X60R4	92.322	21,158	96	99,007	1.201 E5
12X4	54.640	12,522	58	39,878	5.240 E4
6X4	16.959	3,886	20	6,875	1.076E4

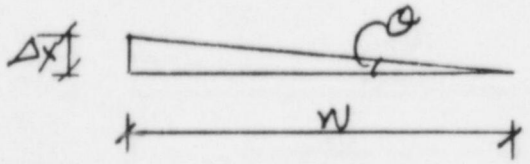
IKxy = 1.1E4 K-IN/RAD (MIN)

KY : $\Delta Y4 = 0.00360472"$

KY FOR 2-CLIPS = $1/\Delta Y4$ $= 247.183$ K/IN
$\simeq 250.0$ K/IN

KYY : $Fx = 2 \times 0.5K = 1.0K$ $\Delta X4 = 0.0558455 \times 2 = 0.111691"$

$M = 1.0W$ K"



REV	BY	DATE	CHECKED	DATE
1	[illegible]	1.20.86	[illegible]	1.28.86

TUGCO

CABLE TRAY-CLIP ANGLE STIFFNESS

IMPELL CORPORATION

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SIZE OF CABLE TRAY	$\theta = \tan^{-1}\frac{\Delta_{X4}}{(W-0.8125)}$	$K_{YY} = 57.296M/\theta$ K-IN/RAD
36 x 6 OR 4	0.181865759	11,341.640 ≃ 1.1E4
30 x 6 OR 4	0.219251106	7,839.778 ≃ 7.8E3
24 x 6 OR 4	0.275983759	4,982.554 ≃ 5.0E3
18 x 6 OR 4	0.372324819	2,769.968 ≃ 2.8E3
12 x 4	0.57996452	1,202.021 ≃ 1.2E3
6 x 4	1.233433121	278.715 ≃ 280.0

K_Z : $\Delta_{Z4} = 0.0013105''$

K_Z FOR 2-CLIPS $= 2/\Delta_{Z4}$ = 1,526.135 K/IN
≃ 1.5E3 K/IN

K_{ZZ} : $\theta_{Z4} = 1.575119^{\circ}$

K_{ZZ} FOR 2-CLIPS $= 57.296/\theta_{Z4}$ = 36.376
≃ 36.0 K-IN/RAD

FRICTIONLESS CLIP –

K_X : SAME AS FRICTION CLIP = 18.0 K/IN

K_{XY} : NO RESISTING CAPACITY = 0.0 K-IN/RAD

K_Y : NO RESISTING CAPACITY = 0.0 K/IN

K_{YY} : SAME AS FRICTION CLIP – 36 x 6 OR 4 = 1.1E4 K-IN/RAD
30 x 6 OR 4 = 7.8E3 "
24 x 6 OR 4 = 5.0E3 "
18 x 6 OR 4 = 2.8E3 "
12 x 4 = 1.2E2 "
6 x 4 = 280.0 "

REV	BY	DATE	CHECKED	DATE
1	[illegible]	1-20-86	[illegible]	1-28-86

TUGCO
CABLE TRAY – CLIP ANGLE STIFFNESS

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Kz : $\Delta z_5 = 0.0013554$" (ONE CLIP RESISTS THE FORCE)

$K_z = 1/\Delta z_5$ $= 735.619$ K/IN

$\hat{=} 740.0$ K/IN

Kzz : NO RESISTING CAPACITY $= 0.0$ K-IN/RAD

REV	BY	DATE	CHECKED	DATE
1	[illegible]	1-20-86	GMR	1-28-86

TUSCO
CABLE TRAY - CLIP ANGLE ST

IMPELL CORPORATION

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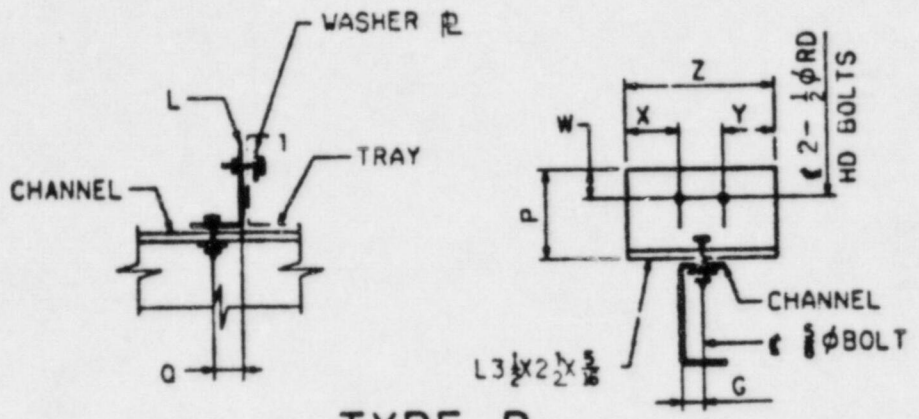
M-10

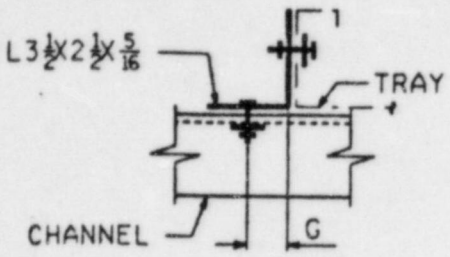


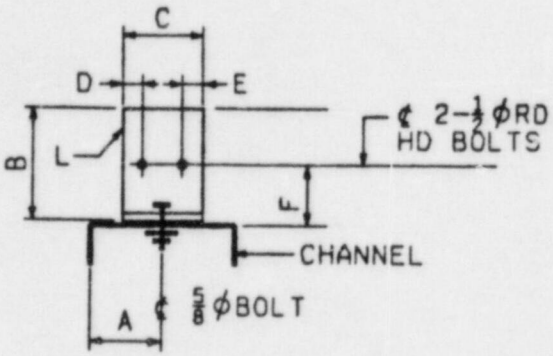
TYPE D

G2	1 13/16	-	3½	1 5/16	-	1½	⅞	1	4	HS	B
G1	13/16	-	3½	1 5/16	-	1½	1	1	4	HS	B
CLAMP NO.	G	H	P	Q	R	W	X	Y	Z	BOLT MKG	WASH TYPE
CLAMP TYPE D											

REF. 5d FOR TUGCO DRAWINGS NUMBER

REV	BY	DATE	CHECKED	DATE	TUGCO CABLE TRAY-CLIP ANGLE STIFFNESS JOB NO 0210.040 CALC NO M-10	PAGE 7 OF 8
0	[illegible]	12.2.85	DC	12-4-85	IMPELL CORPORATION	





TYPE M

G2	4	3½	4	1	1	2 1/16	1 7/16	H. S.	NONE
G1	4¼	3½	4	1 1/16	1	2 1/16	1 3/8	H. S.	NONE
CLAMP NO.	A	B	C	D	E	F	G	BOLT MKG	WASH TYPE
CLAMP TYPE M									

REF. 5E FOR TUGCO DRAWINGS NUMBER

REV	BY	DATE	CHECKED	DATE
2	[illegible]	12.2.85	DC	12-4-85

TUGCO
CABLE TRAY - CLIP ANGLE STIFFNER

IMPELL CORPORATION

JOB NO 0210-040-
CALC NO M-10

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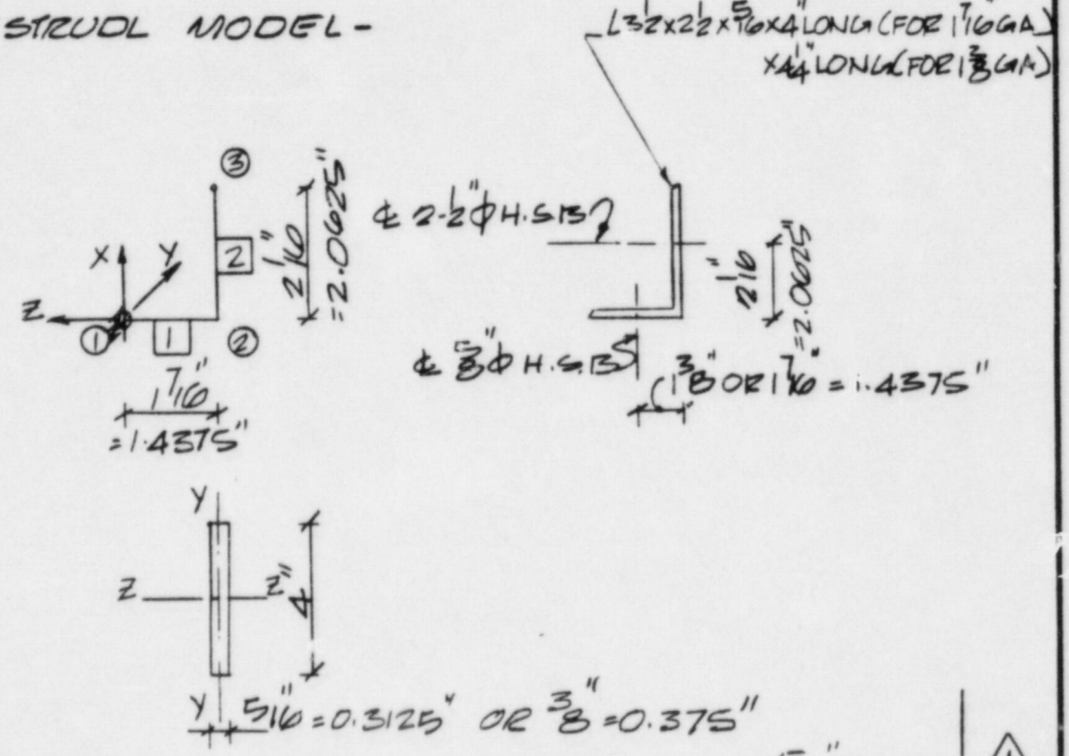
STIFFNESSES FOR CLIP TYPE "D" OR "M"

GT STRUDL MODEL -



MEMBER PROPERTIES : FOR CLIP ANGLE 5/16" THICK ⚠1

$A_X = A_Y = A_Z = 4 \times 0.3125 = 1.25\ IN^2$ $\quad b/d = 4/0.3125 = 12.8$

$\beta = 0.32$

$I_{XX} = 0.32 \times 4 \times 0.3125^3 = 0.039906\ IN^4$

$I_{YY} = 4 \times 0.3125^3/12 = 0.01017\ IN^4$

$I_{ZZ} = 0.3125 \times 4^3/12 = 1.06667\ IN^4$

⚠1

REV	BY	DATE	CHECKED	DATE
1	[illegible]	1.20.86	GMA	1.28.86
0	[illegible]	12.2.85	A.S.	12.3.85

TUGCO

Cable Tray - Clip Angle Stiffness

IMPELL CORPORATION

JOB NO 0210-040

CALC NO M-10

MEMBER PROPERTIES: FOR CLIP ANGLE $\frac{3}{8}$" THICK

$A_x = A_y = A_z = 4 \times 0.375 = 1.50 IN^2$

$b/d = 4/0.375 = 10.67 \quad \beta = 0.32$

$I_{xx} = 0.32 \times 4 \times 0.375^3 = 0.0675 IN^4$

$I_{yy} = 4 \times 0.375^3/12 = 0.01758 IN^4$

$I_{zz} = 0.375 \times 4^3/12 = 2.0 IN^4$

APPLIED LOADS:

K_x : $F_x = 1000/2 = 500$ LBS NODE 3

K_{xx} : FOR ROTATIONAL $\theta = 1°$ $\Delta y = \frac{W}{2} \tan 1°$ W = WIDTH OF CABLE TRAY

AT FIXED END NODE 3

CABLE TRAY SIZE	Δy
36 x 6 OR 4	0.3141912"
30 x 6 OR 4	0.2618826"
24 x 6 OR 4	0.2094608"
18 x 6 OR 4	0.1570956"
12 x 4	0.1047304"
6 x 4	0.05236052"

K_y : $F_y = 1000/2 = 500$ LBS NODE 3

K_z : $F_z = 1000/2 = 500$ LBS NODE 3

K_{zz} : $M_z = 1000/2 = 500$ LBS-IN NODE 3

REV	BY	DATE	CHECKED	DATE
1	[illegible]	1-28-86	[illegible]	1-8-86

TUSCO
CABLE TRAY - CLIP ANGLE STIFFENER

IMPELL CORPORATION

JOB NO 0210-040
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STIFFNESSES FOR CLIP TYPE "D" OR "M" - 5/16 ANGLE

ATTACHMENT-5 (1&2)

Kx : $\Delta x_3 = 0.0017269$"

Kx FOR 2-CLIPS = $1/\Delta x_3$ = 579.072 K/IN
≃ 580.0 K/IN

Kxx : $K_{xx} = K_{xx\theta} + K_{xx\Delta}$

$K_{xx\theta} = \frac{4M}{\theta} = \frac{4 \times 57.296 M}{1^0} = 229.184 M$

$K_{xx\Delta} = \frac{WF_y}{\theta} = 57.296\, WF_y$

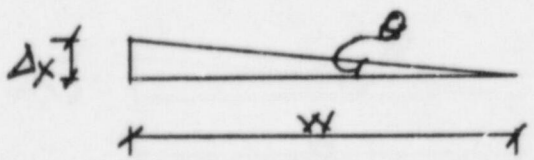
SIZE OF CABLE TRAY WXH	$M\theta_3$ K-IN	$K_{xx\theta}$ K-IN/RAD	Fy_3 K	$K_{xx\Delta}$ K-IN/RAD	K_{xx} K-IN/RAD
36X60R4	0.759	174	993	2,048,217	2.048E6
30X60R4	0.013	3	828	1,423,232	1.423E6
24X60R4	0.785	180	662	910,318	9.105E5
18X60R4	1.558	357	497	512,570	5.129E5
12 X4	2.330	534	331	227,579	2.281E5
6 X4	3.103	711	165	56,723	5.743E4

K_{xx} = 5.7E4 K-IN/RAD (MIN)

Ky : $\Delta y_3 = 0.0067921$"

Ky FOR 2-CLIPS = $1/\Delta y_3$ = 147.230 K/IN
≃ 150.0 K/IN

Kyy :



FOR FX = 2 × 500 = 1000 #

$\Delta x_3 = 2 \times 0.0017269 = 0.0034538$" $M = 1000 \times W$ #"

REV	BY	DATE	CHECKED	DATE
1	[illegible]	1.20.86	[illegible]	[illegible]

TUGCO

CABLE TRAY-CLIP ANGLE STIFFNESS

IMPELL CORPORATION

JOB NO 0210-040

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SIZE OF CABLE TRAY	$\theta = \tan^{-1}\frac{\Delta x_3}{n}$	$K_{yy} = \frac{57.296}{\theta} \times M$ K-IN/RAD
36 X 6 OR 4	0.005496893	375,240.313
30 X 6 OR 4	0.006596272	260,583.551
24 X 6 OR 4	0.00824534	166,773.473
18 X 6 OR 4	0.010993786	93,810.079
12 X 4	0.016490679	41,693.369
6 X 4	0.032981356	10,423.343

K_{yy} = 1.0E4 K-IN/RAD (MIN

K_z : F_z FORCE IS ACTING AT THE BASE OF THE CLIP SIMILAR TO THE CLIP TYPE A AND CAN BE ASSUME CONSERVATIVELY K_z = 7.1E3 K/IN

K_{zz} : $\theta_{z_3} = 0.09055°$

K_{zz} FOR 2-CLIPS $= \frac{57.296}{\theta_{z_3}}$ = 632.755 K-IN/RAD ≈ 630.0 K-IN/RAD

REV	BY	DATE	CHECKED	DATE
1	[signature]	1.20.86	[signature]	1.28.86

TUGCO
CABLE TRAY-CLIP ANGLE STIFFNESS
IMPELL CORPORATION
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CALC NO M-10
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STIFFNESSES FOR CLIP TYPE "D" OR "M" - 3" ANGLE

ATTACHMENT-5 (3&4)

Kx : $\Delta x_3 = 0.0010185"$

Kx FOR 2-CLIPS $= 1/\Delta x_3$ $= 981.836$ K/IN
$\simeq 980.0$ K/IN

Kxx : $Kxx = Kxx\theta + Kx x\Delta$ $Kxx\theta = \frac{4M}{\theta} = \frac{4 \times 57.296M}{1^\circ} = 229.184M$

$Kx x\Delta = \frac{WFy}{\theta} = 57.296 WFy$

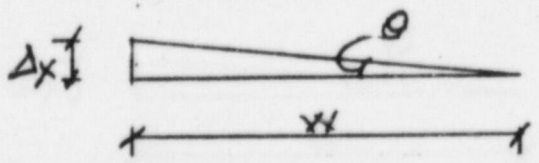
SIZE OF CABLE TRAY WXH	$M\theta_3$ K-IN	Kxxθ K-IN/RAD	Fy_3 K	Kxxδ K-IN/RAD	Kxx K-IN/RAD
36X60R4	1.315	301	1193	2,460,748	2.461 E6
30X60R4	0.017	4	994	1,708,566	1.709 E6
24X60R4	1.349	309	795	1,093,207	1.093 E6
18X60R4	2.682	615	596	614,671	6.153 E5
12 X4	4.015	920	397	272,958	2.739 E5
6 X4	5.347	1,225	198	68,067	6.929 E4

$Kxy = 6.9E4$ K-IN/RAD
(MIN)

Ky : $\Delta y_3 = 0.0039707"$

Ky FOR 2-CLIPS $= 1/\Delta y$ $= 251.845$ K/IN
$\simeq 250.0$ K/IN

Kyy :



FOR FX = 2 X 500 = 1000#

$\Delta x_2 = 2 \times 0.0010185 = 0.002037"$ $M = 1000 \times W$ #"

REV	BY	DATE	CHECKED	DATE
1	[illegible]	1.20.86	[illegible]	1.24.86

TUGCO
CABLE TRAY-CLIP ANGLE STIFFNESS

IMPELL CORPORATION

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SIZE OF CABLE TRAY	$\theta = \tan^{-1}\frac{\Delta x_3}{r_0}$	$K_{yy} = \frac{57.296}{\theta} \times M$ K-IN/RAD
36 X 6 OR 4	0.003241986	6036,232.199
30 X 6 OR 4	0.003890383	441,827.916
24 X 6 OR 4	0.004862979	282,769.866
18 X 6 OR 4	0.006483972	159,058.050
12 X 4	0.009759568	70,692.467
6 X 4	0.019451916	17,673.117

$K_{yy} = 1.8E4$ K-IN/RAD (MIN)

K_z : F_z FORCE IS ACTING AT THE BASE OF THE CLIP SIMILAR TO THE CLIP TYPE A AND CAN BE ASSUME CONSERVATIVELY $K_z = 7.1E3$ K/IN

K_{zz} : $\theta_{z_3} = 0.0527043°$

K_{zz} FOR 2-CLIPS $= \frac{57.296}{\theta_z} = 1087.122$ K-IN/RAD $\cong 1.1E3$ K-IN/RAD

REV	BY	DATE	CHECKED	DATE
1	[signature]	1.20.86	[signature]	1.28.86

TUGCO

CABLE TRAY-CLIP ANGLE STIFFNESS

IMPELL CORPORATION

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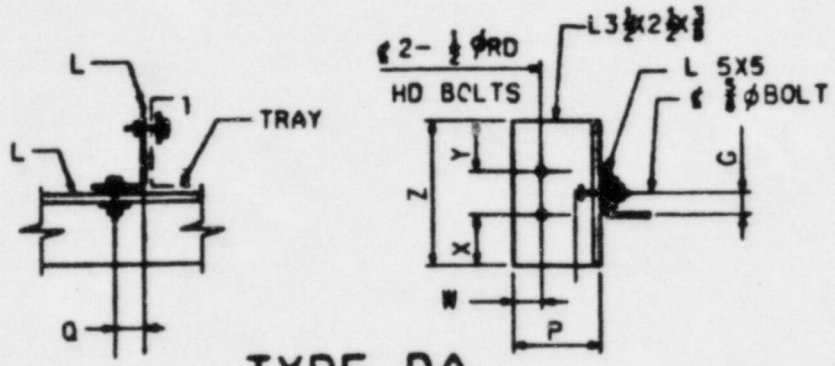
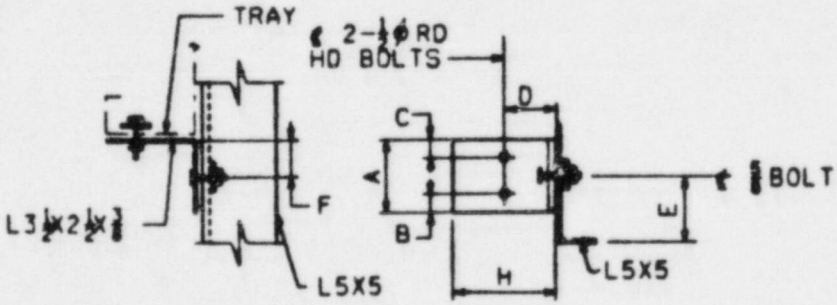


TYPE DA

G2	2⅝	3½	1⅜	-	1⅝	1 1/16	1	4	H. S.	B
G1	2⅝	3½	1⅜	-	1⅝	1	1 1/16	4	H. S.	B
CLAMP NO.	G	P	Q	R	W	X	Y	Z	BOLT MKG	WASH TYPE
CLAMP TYPE DA										



TYPE L

G2	4	[illegible]	1	2	2 7/16	[illegible]	-	3½	HS	F
G1	4	1	1	2	2½	½	-	3½	HS	F
CLAMP NO.	A	B	C	D	E	F	G	H	BOLT MKG	WASH TYPE
CLAMP TYPE L										

REF. 5f, 5g AND 5h FOR TUGCO DRAWINGS NUMBER

					TUGCO
					CABLE TRAY - CLIP ANGLE STIFFNESS
0	[illegible]	12-2-85	DC	12-4-85	IMPELL CORPORATION
REV	BY	DATE	CHECKED	DATE	

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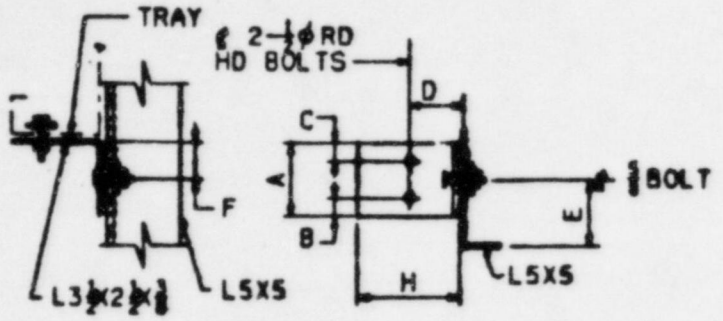
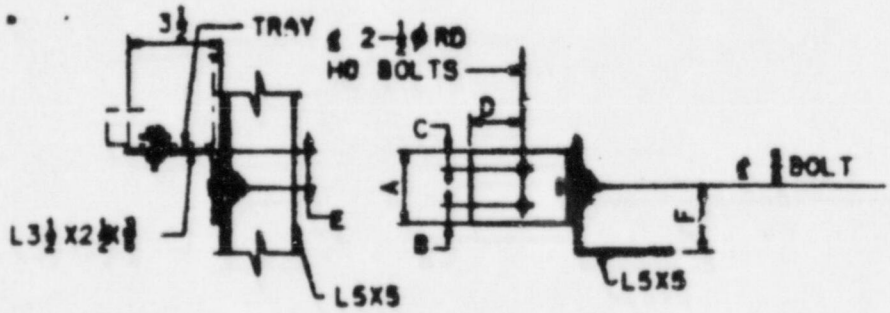


TYPE L

G2	4	1	1	2	2½	1 13/16	-	3½	HS	F
G1	4	1	1	2	2½	1 13/16	-	3½	HS	F
CLAMP NO.	A	B	C	D	E	F	G	H	BOLT MKG	WASH TYPE
CLAMP TYPE L										



TYPE LA

G2	4 3/16	1½	1⅛	2	1⅛	2 3/16	-	-	H. S.	F
G1	4	1	1	1⅛	1⅛	2 3/16	-	-	H. S.	F
CLAMP NO.	A	B	C	D	E	F	G	H	BOLT MKG	WASH TYPE
CLAMP TYPE LA										

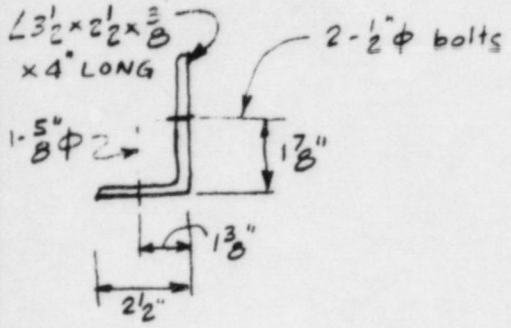
REV	BY	DATE	CHECKED	DATE
0	[illegible]	12-2-85	NC	12-4-85

TUSCO
CABLE TRAY - CLIP ANGLE STIFFNESS

IMPELL CORPORATION

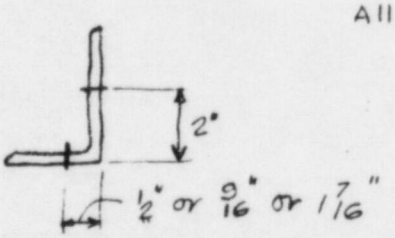
JOB NO 020-040
CALC NO M-10

STIFFNESS FOR CLIP TYPE "DA"

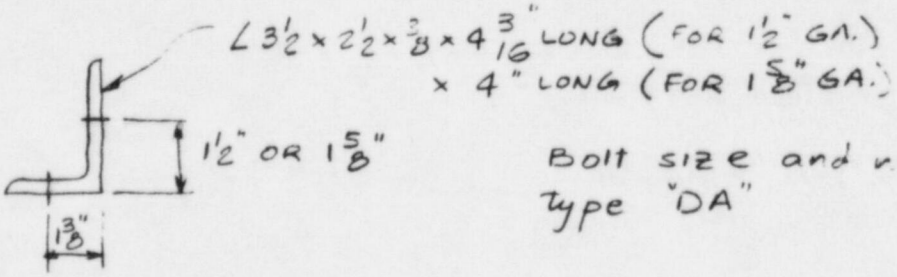


STIFFNESS FOR CLIP TYPE "L"

All other dimensions same as type "DA"



STIFFNESS FOR CLIP TYPE "LA"

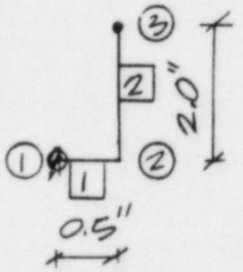


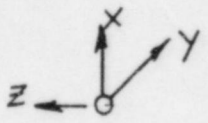
Bolt size and no. same as type "DA"

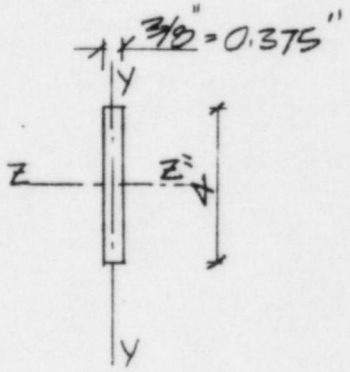
					TULACO	
					CABLE TRAY - CLIP ANGLE STIFFNESS	
					JOB NO 0210-040	PAGE 3
0	A. Lahr	11.27.05	[illegible]	12.2.05	CALC NO M-10	OF 9
REV	BY	DATE	CHECKED	DATE	IMPELL CORPORATION	

STIFFNGSSES FOR CLIP TYPE "DA" OR "L" OR "LA"

GTSTRUDL MODEL – ½" LEG







MEMBER PROPERTIES

$AX = AY = AZ = 4 \times 0.375 = 1.5\ IN^2$

$b/d = 4/0.375 = 10.67 \simeq 10 \qquad \beta = 0.313$

$IXX = 0.313 \times 4 \times (0.375)^3 = 0.0660\ IN^4$

$IYY = 4 \times (0.375)^3/12 = 0.01758\ IN^4$

$IZZ = 0.375 \times (4)^3/12 = 2.0\ IN^4$

REV	BY	DATE	CHECKED	DATE
1	[illegible]	1.20.86	GMR	1.28.86
0	[illegible]	12.2.85	A. Saha	12.2.85

TUCSCO
CABLE TRAY-CLIP ANGLE STIFFNESS

IMPELL CORPORATION

JOB NO 0210-000
CALC NO M-10

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STIFFNESSES FOR CLIP TYPE "DA" OR "L" OR "LA" - ½" LEG

ATTACHMENT-6 (1&2)

Kx : $\Delta x_3 = 0.0001538"$

Kx FOR 2-CLIPS $= 2/\Delta x_3$ $= 6,501.951$ K/IN
$\simeq 6.5E3$ K/IN

Kxx : $Kxx = Kxx\theta + Kxx\Delta$ $Kxx\theta = \frac{4M}{\theta} = \frac{4 \times 57.296M}{1^\circ} = 229.184M$

$Kxx\Delta = \frac{WFy}{\theta} = 57.296WFy$

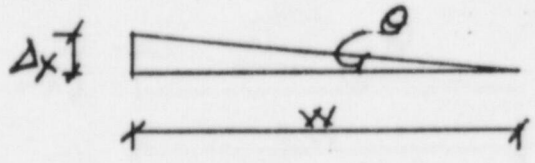
SIZE OF CABLE TRAY WXH	$M\theta_3$ K-IN	Kxxθ K-IN/RAD	Fy_3 K	KxxΔ K-IN/RAD	Kxx K-IN/RAD
36X60R4	5.380	1,233	1,694	3,494,139	3.495E6
30X60R4	5.613	1,286	1,412	2,427,058	2.428E6
24X60R4	5.845	1,340	1,129	1,552,492	1.553E6
18X60R4	6.077	1,393	847	873,534	8.749E5
12 X4	6.309	1,446	565	388,406	3.899E5
6 X4	6.541	1,499	282	96,945	9.844E4

Kxy = 9.8E4 K-IN/RAD (MIN)

Ky : $\Delta y_3 = 0.0027552"$

Ky FOR 2-CLIPS $= 2/\Delta y_3$ $= 725.900$ K/IN
$\simeq 730.0$ K/IN

Kyy :



FOR FX = 1000#

$\Delta x_3 = 0.0001538"$ $M = 1000 \times W$ #"

REV	BY	DATE	CHECKED	DATE
1	[illegible]	1-20-86	[illegible]	1-28-86

TUGCO
CABLE TRAY-CLIP ANGLE STIFFNESS

IMPELL CORPORATION

JOB NO 0210-040
CALC NO M-10

PAGE 5 OF 9

SIZE OF CABLE TRAY	$\theta = \tan^{-1}\frac{\Delta x}{n}$	$K_{yy} = \frac{57.296 \times M}{\theta}$ K-IN/RAD
36 x 6 ORA	0.000024478	8,426,560.386
30 x 6 ORA	0.000295736	5,851,778.046
24 x 6 ORA	0.000367117	3,745,137.949
18 x 6 ORA	0.000489556	2,106,640.096
12 x 4	0.000734334	936,284.487
6 x 4	0.001468668l	234,071.122

$K_{yy} = 2.3E5$ K-IN/RAD

K_z : F_z FORCE IS ACTING AT THE BASE OF THE CLIP SIMILAR TO THE CLIP TYPE A AND CAN BE ASSUME CONSERVATIVELY $K_z = 7.1E3$ K/IN

K_{zz} : $\theta_{z3} = 0.0387269°$

K_{zz} FOR 2-CLIPS $= \frac{2 \times 57.296}{\theta_{z3}}$ $= 2,958.976$ K-IN/RAD $\approx 3.0E3$ K-IN/RAD

REV	BY	DATE	CHECKED	DATE
1	[illegible]	1.20.86	[illegible]	1.28.86

IMPELL CORPORATION

TUGCO

CABLE TRAY-CLIP ANGLE STIFFNESS

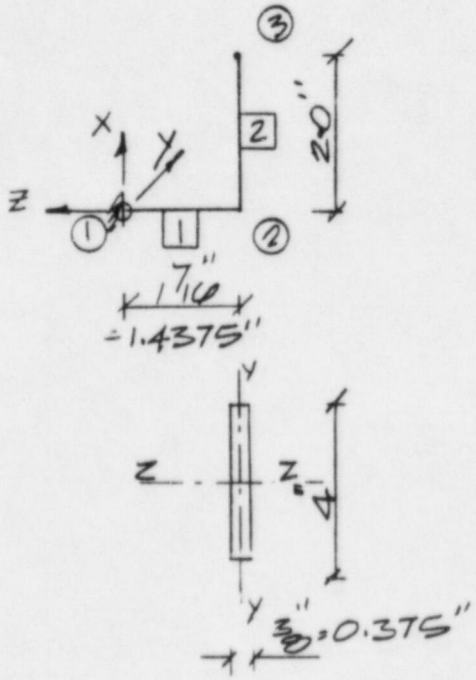
JOB NO 0210-040

CALC NO M-10

PAGE 6 OF 9

STIFFNESSES FOR CLIP TYPE "DA" OR "L" OR "LA"

GTSTRUDL MODEL - 1.4375" LEG



MEMBER PROPERTIES:

$A_X = A_Y = A_Z = 4 \times 0.375 = 1.5\ IN^2$ $\quad b/d = 4/0.375 = 10.67 \simeq 10$

$\beta = 0.313$

$I_{XX} = 0.313 \times 4 \times 0.375^3 = 0.0066\ IN^4$

$I_{YY} = 4 \times 0.375^3/12 = 0.01758\ IN^4$

$I_{ZZ} = 0.375 \times 4^3/12 = 2.0\ IN^4$

1

REV	BY	DATE	CHECKED	DATE
1	[signature]	1.20.86	GMA	1.28.86
0	[signature]	12.2.85	A. Saha	12.3.85.

TURCO

CABLE TRAY - CLIP ANGLE STIFFNESS

IMPELL CORPORATION

JOB NO 2210-046

CALC NO M-10

STIFFNESSES FOR CLIP TYPE "DA" OR "L" OR "LA" - 1 7/16" LEG

ATTACHMENT-6 (3&4)

Kx : $\Delta x_3 = 0.0020357$"

Kx FOR 2-CLIPS $= 2/\Delta x$ $= 982.464$ K/IN
$\Rightarrow 980.0$ K/IN

Kxx : $Kxx = Kxx\theta + Kxx\Delta$

$Kxx\theta = \frac{4M}{\theta} = \frac{4 \times 57.296M}{1^\circ} = 229.184M$

$Kxx\Delta = \frac{WFy}{\theta} = 57.296WFy$

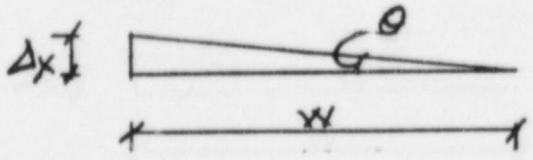
SIZE OF CABLE TRAY WXH	$M\theta_3$ K-IN	Kxxθ K-IN/RAD	Fy_3 K	KxxΔ K-IN/RAD	Kxx K-IN/RAD
36X60R4	1.572	360	1,229	2,535,004	2.535E6
30X60R4	0.187	43	1,024	1,760,113	1.760E6
24X60R4	1.196	274	819	1,126,210	1.126E6
18X60R4	2.581	592	614	633,235	6.338E5
12 X4	3.966	909	410	281,896	2.828E5
6 X4	5.350	1,226	205	70,474	7.170E4

Kxx = 7.2E4 K-IN/RAD (MIN)

Ky : $\Delta y_3 = 0.0076593$"

Ky FOR 2-CLIPS $= 2/\Delta y_3$ $= 261.304$ K/IN
$\Rightarrow 260.0$ K/IN

Kyy :



FOR FX = 1000 #

$\Delta x_3 = 0.0020357$ $M = 1000 \times W$ #"

REV	BY	DATE	CHECKED	DATE
1	[illegible]	1.20.88	[illegible]	1.28.88

TUGCO
CABLE TRAY-CLIP ANGLE STIFFNESS

IMPELL CORPORATION

JOB NO 0210-040
CALC NO M-10
PAGE 8 OF 9

SIZE OF CABLE TRAY	$\theta = \tan^{-1}\frac{\Delta x_3}{n}$	$K_{yy} = \frac{57.296 \times M}{\theta}$ K-IN/RAD
36 X 6 OR 4	0.003239917	636,638.497
30 X 6 OR 4	0.0038879	442,110.067
24 X 6 OR 4	0.004859875	282,950.443
18 X 6 OR 4	0.006479834	159,159.625
12 X 4	0.009719751	70,737.611
6 X 4	0.019439503	17,684.403

$K_{yy} = 1.8E4$ K-IN/RAD (MIN)

K_z : F_z FORCE IS ACTING AT THE BASE OF THE CLIP SIMILAR TO THE CLIP TYPE A AND CAN BE ASSUMED CONSERVATIVELY

$K_z = 7.1E3$ K/IN

K_{zz} : $\theta_{z_3} = 0.10769881°$

K_{zz} FOR 2-CLIPS $= \frac{2 \times 57.296}{\theta_{z_3}}$ $= 1{,}064.92$ K-IN/RAD $\approx 1.1E3$ K-IN/RAD

REV	BY	DATE	CHECKED	DATE
1	[illegible]	1-20-86	[illegible]	1-28-86

TUGCO
CABLE TRAY-CLIP ANGLE STIFFNESS

IMPELL CORPORATION

JOB NO 0210-040
CALC NO M-10

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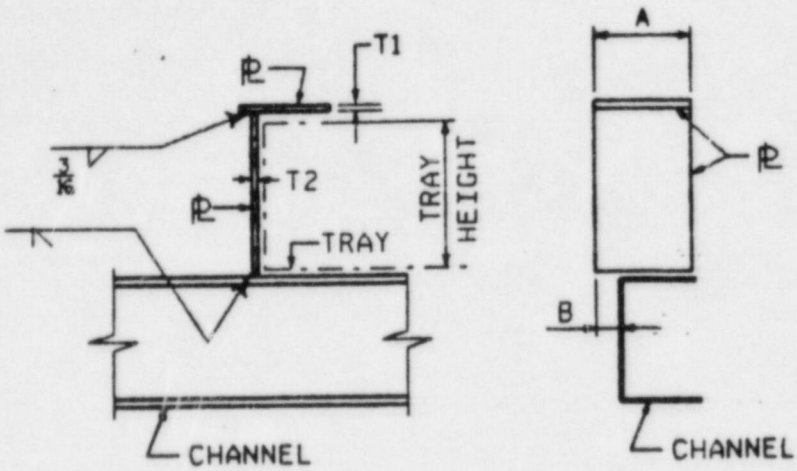


TYPE G

(2)	2	0	1/2	1/2
(1)	2	0	1/2	1/2
CLAMP NO.	A	B	T1	T2

CLAMP TYPE G

REF. 5j FOR TUGCO DRAWING NUMBER

REV	BY	DATE	CHECKED	DATE
0	[illegible]	12-4-85	MC	12-4-85

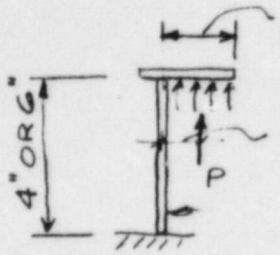
TUGCO
CABLE TRAY - CLIP ANGLE STIFFENER

IMPELL CORPORATION

JOB NO 0210-040
CALC NO M-10

PAGE 1 OF 3

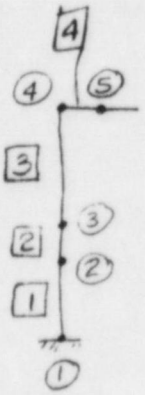
STIFFNESS FOR CLIP TYPE "G"

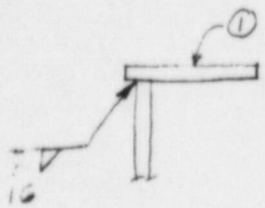


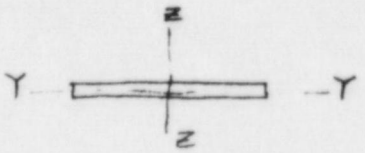
Not given

assume 1/4" for 4" high tray
3/8" for 6" high tray.

∠ PL 1/4 × 2, TYP.







$$\frac{b}{d} = \frac{2}{0.25} = 8 \qquad \beta = 0.307$$

$$I_{xx} = 0.307 \times (0.25)^3 \times (2) = 0.009594 \text{ in}^4.$$

$$I_{YY} = (0.25)^3 \times 2/12 = 0.002604 \text{ in}^4.$$

$$I_{ZZ} = 0.25 \times (2)^3/12 = 0.16667 \text{ in}^4$$

$$A_x = A_Y = A_Z = 0.25 \times 2 = 0.5 \text{ in}^2.$$

Although weld capacity is not high but plate ① is assumed to have full fixity against rotation.

NOTE: IT IS ASSUMED THAT CLIP TYPE "G" HAS A RESISTING CAPACITY FOR UPWARD X-DIRECTION FORCE.

					TUGCO
					Cable Tray - Clip Angle stiffness
0	A. [illegible]	11.27.85	[illegible]	12.5.85	IMPELL CORPORATION — JOB NO 0210-040- — CALC NO M-10 — PAGE 2 OF 3
REV	BY	DATE	CHECKED	DATE	

STIFFNESSES FOR CLIP TYPE "G" - 4" HIGH
ATTACHMENT-7

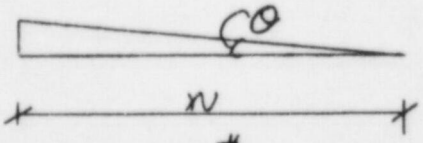
K_X : $\Delta_{XS} = 0.0036358$"

K_X FOR 2-CLIPS = $2/\Delta_{XS}$ = 550.085 K/IN (≃ 550.0)

K_{XX} : NO RESISTING CAPACITY = 0.0 K-IN/RAD

K_Y : NO RESISTING CAPACITY = 0.0 K/IN

K_{YY} : Δ_X



FOR $F_X = 1000$ #

$\Delta_{XS} = 0.0036358$" $M = 1000 \times W$ #"

SIZE OF CABLE TRAY	$\theta = \tan^{-1}\frac{\Delta_{XS}}{W}$	$K_{YY} = \frac{57.296 \times M}{\theta}$ K-IN/RAD
36 x4	0.005786555	356,456.624 ≃ 3.565E5
30 x4	0.006943866	247,539.322 ≃ 2.475E5
24 x4	0.008679833	158,425.167 ≃ 1.584E5
18 x4	0.011573111	89,114.157 ≃ 8.911E4
12 x4	0.017359665	39,606.293 ≃ 3.961E4
6 x4	0.034719328	9,901.574 ≃ 9.901E3

K_{YY}: 9.9E3 (MIN.)

K_Z : AS CLIP IS ATTACHED TO HORI. MEM. WITH A WELD AND FZ FORCE IS ASSUME TO ACT AT NODE-1, KZ=0
- FOR CALC. TAKE VALUE 10^4 ∴ $K_Z = 1.0E^4$ K/IN

K_{ZZ} : NO RESISTING CAPACITY = 0.0 K-IN/RAD

REV	BY	DATE	CHECKED	DATE
1	[illegible]	1-20-86	[illegible]	1-21-86

TUGCO
CABLE TRAY-CLIP ANGLE STIFFNESS
IMPELL CORPORATION
JOB NO 0210-040
CALC NO M-10
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STIFFNESSES FOR CLIP TYPE "G" - 6" HIGH

ATTACHMENT-8

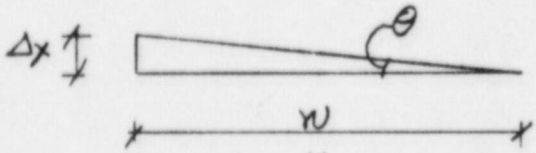
Kx : $\Delta x5 = 0.011683''$

Kx FOR 2-CLIPS $= 2/\Delta x5$ = 171.189 K/IN

Kxx : NO RESISTING CAPACITY = 0.0 K-IN/RAD

Ky : NO RESISTING CAPACITY = 0.0 K/IN

Kyy :



FOR $F_x = 1000^{\#}$ $\Delta x5 = 0.011683''$ $M = 1000 \times W^{\#''}$

SIZE OF CABLE TRAY	$\theta = \tan^{-1} \frac{\Delta x5}{W}$	$K_{yy} = \frac{57.296 \times M}{\theta}$ K-IN/RAD
36x6	0.018594071	110,930.843 ≃ 1.109E5
30x6	0.022312885	77,035.309 ≃ 7.704E4
24x6	0.027891105	49,302.600 ≃ 4.930E4
18x6	0.037188138	27,732.714 ≃ 2.773E4

Kyy: 2.8E4 (MIN)

Kz : AS CLIP IS ATTACHED TO HORI. MEM.
WITH A WELD AND Fz FORCE IS ASSUE
TO ACT ACT AT NODE - 1, Kz = ∞
- FOR CALC. TAKE VALUE 10^4 ∴ Kz = 1.0E4 K/IN

Kzz : NO RESISTING CAPACITY = 0.0 K-IN/RAD

REV	BY	DATE	CHECKED	DATE
1	[signature]	1-20-86	[signature]	1-?-86

TUGCO
CABLE TRAY - CLIP ANGLE STIFFNESS

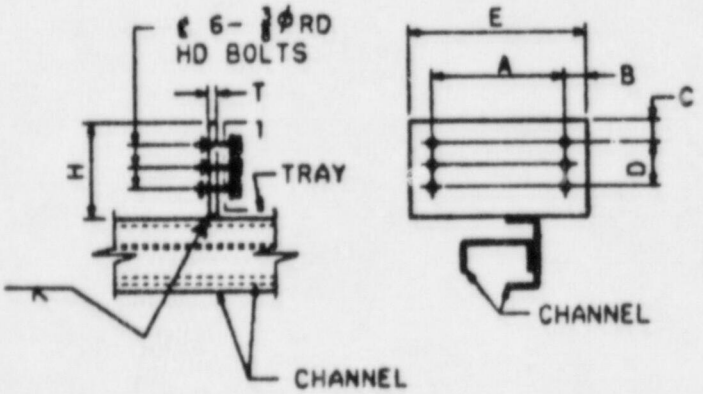
IMPELL CORPORATION

JOB NO 0210-040
CALC NO M-10

PAGE 1 OF 1

(2)	1 1/2	3/4	1 1/8	2	6	-	4	1/2
(1)	1 1/2	1 1/8	1 1/8	2	6	-	4	1/2
CLAMP NO.	A	B	C	D	E	F	H	T

CLAMP TYPE T



TYPE T

REF. 5k FOR TUGCO DRAWING NUMBER

REV	BY	DATE	CHECKED	DATE
0	[illegible]	12-4-85	DC	12-4-85

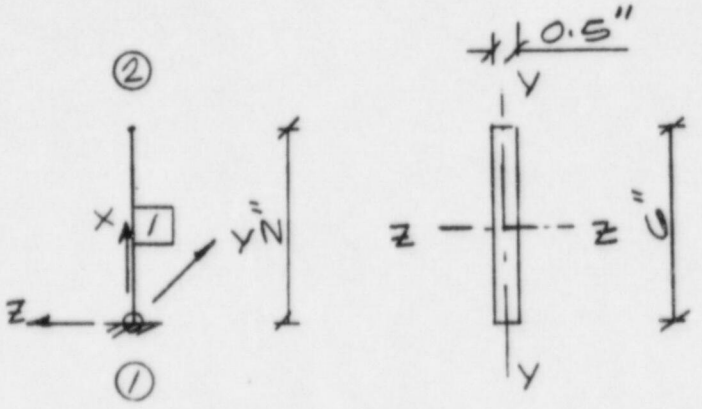
TUGCO
CABLE TRAY - CLIP ANGLE STIFFENESS

IMPELL CORPORATION

JOB NO 0210-040
CALC NO M-10
PAGE 1 OF 5

STIFFNESSES FOR CLIP TYPE "T" @ 1/2"

GTSTRUDL MODEL -



$b/d = 6/0.5 = 12 \quad \beta = 0.32$

$A_x = A_y = A_z = 0.5 \times 6 = 3.0 \text{ in}^2$

$I_{xx} = 0.32 \times 0.5^3 \times 6 = 0.24 \text{ in}^4$

$I_{yy} = 6 \times 0.5^3/12 = 0.0625 \text{ in}^4$

$I_{zz} = 0.5 \times 6^3/12 = 9.0 \text{ in}^4$

					TUGCO
					CABLE TRAY - CLIP ANGLE STIFFNESS
1	[illegible]	1.20.86	[illegible]	1.28.86	IMPELL CORPORATION
REV	BY	DATE	CHECKED	DATE	

JOB NO 0210-040

CALC NO M-10

PAGE 2 OF 5

APPLIED LOADS:

Kx : Fx = 1000 LBS NODE 2

Kxx : FOR ROTATIONAL $\theta = 1°$ $\Delta y_2 = \frac{W}{2} \tan 1°$

W = WIDTH OF CABLE TRAY

AT FIXED END NODE 2

CABLE TRAY SIZE	Δy_2
36 x 6 ORA	0.3141912"
30 x 6 ORA	0.2618260"
24 x 6 ORA	0.2094608"
18 x 6 ORA	0.1570956"
12 x 4	0.1047304"
6 x 4	0.0523652"

Ky : Fy = 1000 LBS NODE 2

Kyy : FOR ROTATIONAL $\theta = 1°$ $\Delta x_2 = \frac{W}{2} \tan 1°$

AT FIXED END NODE 2

CABLE TRAY SIZE	Δx_2
36 x 6 ORA	0.6283824"
30 x 6 ORA	0.523652"
24 x 6 ORA	0.4189212"
18 x 6 ORA	0.3141912"
12 x 4	0.2094608"
6 x 4	0.1047304"

REV	BY	DATE	CHECKED	DATE
1	[illegible]	1-20-86	[illegible]	1-20-86

TUGCO

CABLE TRAY - CLIP ANGLE STIFFNESS

IMPELL CORPORATION

JOB NO 0210-040

CALC NO M-10

STIFFNESSES FOR CLIP TYPE "T" - @ ½"

ATTACHMENT-9

K_X : $\Delta X_2 = 0.0002226"$

K_X FOR 2-CLIPS $= {}^2/_{\Delta X_2}$ $= 8,849.558 K/IN$ ($\hat{=}$ 8.9E3)

K_{XX} : $K_{XX} = K_{XX\theta} + K_{XX\Delta}$ $K_{XX\theta} = \frac{4M}{\theta} = \frac{4 \times 57.296M}{1^\circ} = 229.184M$

$K_{XX\Delta} = \frac{\nu F_Y}{\theta} = 57.296\,\nu F_Y$

SIZE OF CABLE TRAY WXH	$M\theta_2$ K-IN	$K_{XX\theta}$ K-IN/RAD	F_Y K	$K_{XX\Delta}$ K-IN/RAD	K_{XX} K-IN/RAD
36X60R4	24.713	5,663	5,325	109,836,432	1.098E8
30X60R4	↓	↓	4,437	7,626,670	7.626E6
24X60R4			3,550	4,881,619	4.881E6
18X60R4			2,662	2,745,395	2.745E6
12 X4			1,775	1,220,404	1.220E6
6 X4			887	304,924	3.049E5

$K_X = 3.0E5$ (MIN) K-IN/RAD

K_Y : $\Delta Y_2 = 0.0000665"$

K_Y FOR 2-CLIPS $= {}^2/_{\Delta Y_2}$ $= 30,075.188 K/IN$ ($\hat{=}$ 3.0E4)

K_{YY} : $K_{YY} = K_{YY\theta} + K_{YY\Delta}$ $K_{YY\theta} = 229.184M$

$K_{YY\Delta} = 57.296\,\nu F_X$

REV	BY	DATE	CHECKED	DATE
1	[illegible]	1-20-86	[illegible]	1-28-86

TUGCO

CABLE TRAY-CLIP ANGLE STIFFNESS

IMPELL CORPORATION

JOB NO 0210.040

CALC NO M-10

SIZE OF CABLE TRAY WXH	MGz K-IN	KyyΘ K-IN/RAD	Fx K	KxxΔ K-IN/RAD	Kxy K-IN/RAD
36X60R4	25.657	5,880	27,806	57,354,212	5.735E7
30X60R4			23,117	39,735,348	3.974 E7
24X60R4			18,537	25,490,302	2.549 E7
18X60R4			13,903	14,338,553	1.434 E7
12X4			9,269	6,372,919	6.373 E6
6X4			4,634	1,593,057	1.593 E6

Kyy = 1.0E6 (MIN)
K-IN/RAD

Kz : Fz FORCE IS ACTING AT THE BASE OF THE CLIP SIMILAR TO THE CLIP TYPE A AND CAN BE ASSUMG CONSERVATIVELY Kz = 7.1E3 K/IN

Kzz : $\theta_{zz} = 0.0004316^\circ$

Kzz FOR 2-CLIPS = $\frac{2 \times 57.296}{\theta}$ = 265,505.097 K-IN/RAD
(≈ 2.7E5)

REV	BY	DATE	CHECKED	DATE
1	[illegible]	1.20.86	[illegible]	1.21.86

TUGCO
CABLE TRAY-CLIP ANGLE STIFFNESS

IMPELL CORPORATION

JOB NO 0210-040
CALC NO M-10
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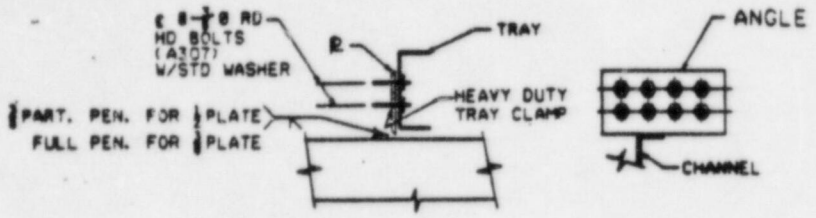
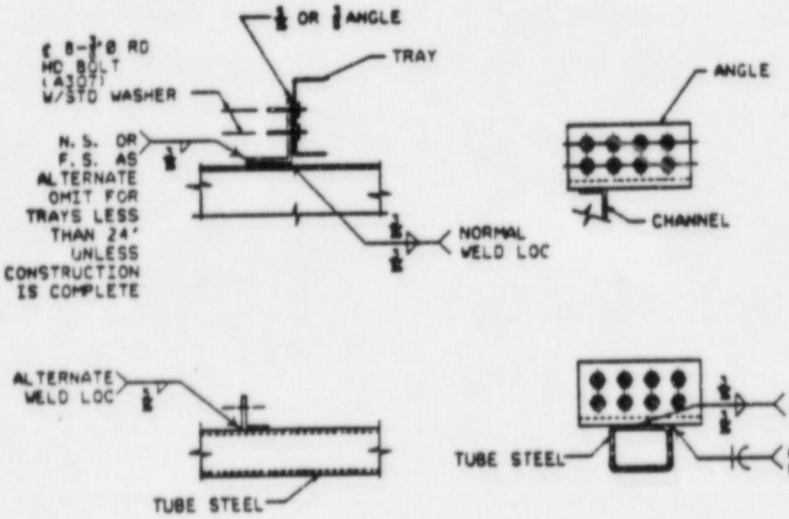


TYPE 'E' HEAVY DUTY CLAMP
AT TRAY SPLICE



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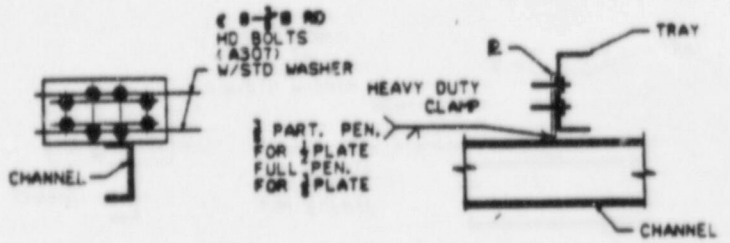
AT TRAY SPLICE



TYPE 'K' CLAMP HEAVY DUTY
AT TRAY SPLICE

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TUGCO
CABLE TRAY-CLIP ANGLE STIFFNESS

IMPELL CORPORATION

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STIFFNESSES FOR CLIP TYPE "E" OR "H" OR "K"

CLIP TYPES E, H AND K HAVE SOME SIMILARITY TO CLIP TYPE T. IN CLIP TYPE H, CLIP ANGLE ONE LEG IS WELDED TO HORIZONTAL MEMBER AND OTHER CLIP ANGLE LEG WILL ACT THE SIMILAR WAY OF CLIP T PLATE. CLIP TYPES H AND E HAVE 2 ROWS OF 4 BOLTS AND CLIP TYPE K HAVE 4 ROWS OF 2 BOLTS AS COMPARE TO CLIP TYPE T 3 ROWS OF 2 BOLTS. FOR STIFFNESS VALUES MORE THAN 10^3, CLIP CAN BE CONSIDER RIGID THERE FORE IT IS CONSERVATIVE TO USE CLIP TYPE T STIFFNESSES FOR THESE CLIPS E, H AND K.

CONCLUSION: FOR CLIP TYPE E OR H OR K STIFFNESSES, USE CLIP TYPE T STIFFNESS VALUES.

REV	BY	DATE	CHECKED	DATE
1	[illegible]	1-20-88	GMR	1-28-88

TUGCO
CABLE TRAY-CLIP ANGLE STIFFNESS
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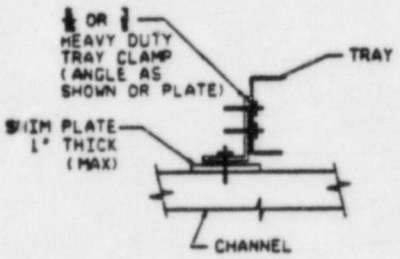
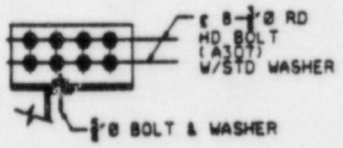
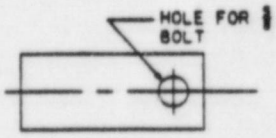






SHIM PLATE
(LENGTH & WIDTH TO SUIT)

NOTE:
THE ANGLE MAY BE BENT TO FIT THE CURVATURE OF TRAY.

TYPE 'F' HEAVY DUTY CLAMP
AT TRAY SPLICE

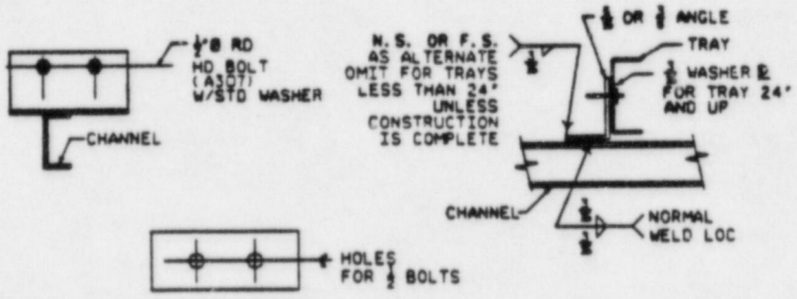
					TUGCO	
					CABLE TRAY - CLIP ANGLE STIFFNESS	
1	[illegible]	1-20-86	[illegible]	1-21-86	IMPELL CORPORATION	JOB NO 0210-040 / CALC NO M-10 / PAGE 1 OF 2
REV	BY	DATE	CHECKED	DATE		

STIFFNESSES FOR CLIP TYPE "F"

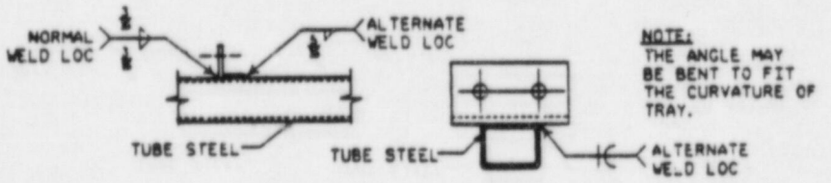
CLIP TYPE F IS SIMILAR TO CLIP TYPES "D" AND "M". ONLY THE DIFFERENCE IS THAT 2 ROWS OF 4 BOLTS ARE USED TO CONNECT CLIP TO CABLE TRAY INSTEAD OF 1 ROW OF 2 BOLTS. IT IS CONSERVATIVE TO USE CLIP TYPE D STIFFNESSES FOR THIS CLIP F.

CONCLUSION: FOR CLIP TYPE F STIFFNESSES, USE CLIP TYPE D OR M TYPE STIFFNESS VALUES

					TUGCO		
					CABLE TRAY - CLIP ANGLE STIFFNESS		
					IMPELL CORPORATION	JOB NO 0210-040	PAGE 2
1	[illegible]	1-20-86	6mn	1-28-86		CALC NO M-10	OF 2
REV	BY	DATE	CHECKED	DATE			



⅜ WASHER PLATE



TYPE 'J' CLAMP HEAVY DUTY

REV	BY	DATE	CHECKED	DATE	TUCO CABLE TRAY - CLIP ANGLE STIFFNESS JOB NO 0210-040 CALC NO M-10	PAGE 1 OF 2
1	[signature]	1-20-86	GMR	1-28-86	IMPELL CORPORATION	

STIFFNESSES FOR CLIP TYPE "J"

CLIP TYPE J IS SIMILAR TO CLIP TYPE B. IN CLIP TYPE J, CLIP ANGLE ONE LEG IS WELDED TO HORIZONTAL MEMBER AND OTHER CLIP ANGLE LEG WILL ACT THE SIMILAR WAY OF CLIP B PLATE. THERE FOR USE CLIP TYPE B STIFFNESSES FOR THIS CLIP J.

CONCLUSION: FOR CLIP TYPE J STIFFNESSES USE CLIP TYPE B STIFFNESS VALUES.

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1	[illegible]	1.20.86	[illegible]	1.24.86

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CABLE TRAY - CLIP ANGLE STIFFNESS
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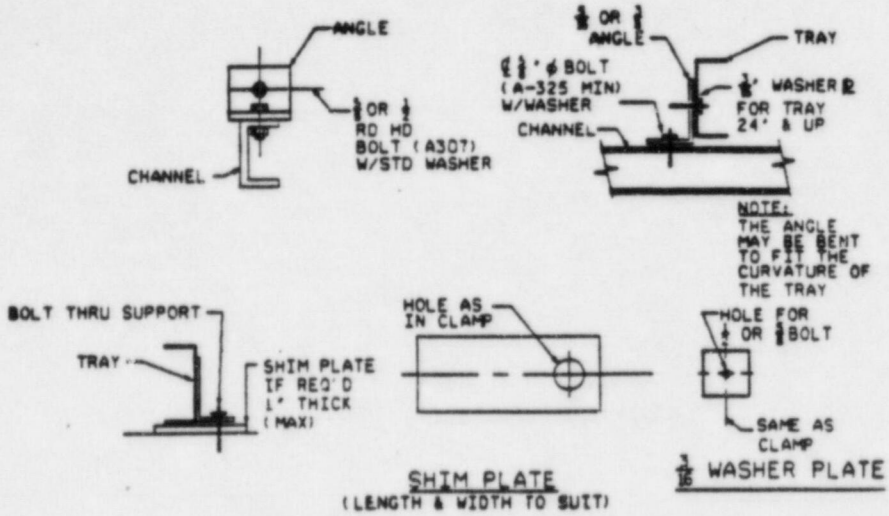
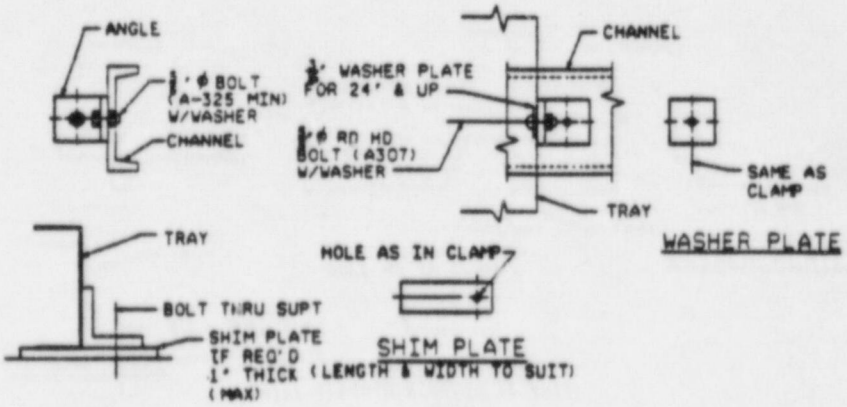


TYPE 'N' CLAMP HEAVY DUTY



TYPE 'O' CLAMP HEAVY DUTY

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1	[illegible]	1.20.86	[illegible]	1.21.86

TUGCO
CABLE TRAY - CLIP ANGLE STIFFNESS

IMPELL CORPORATION

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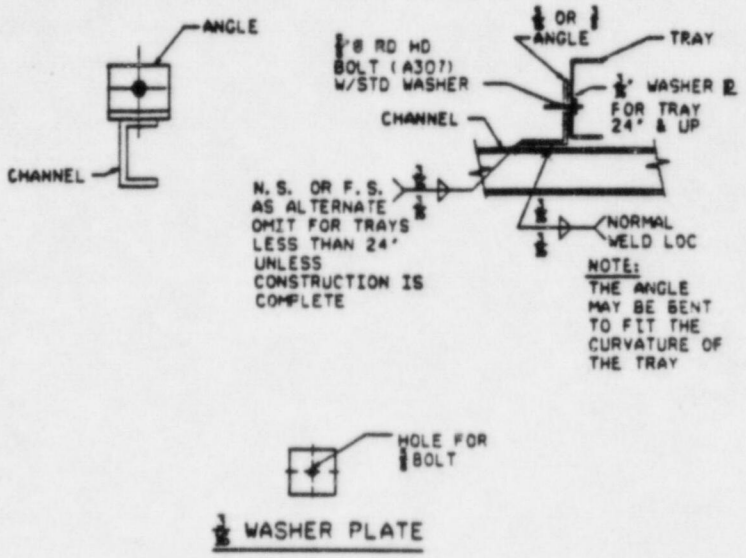
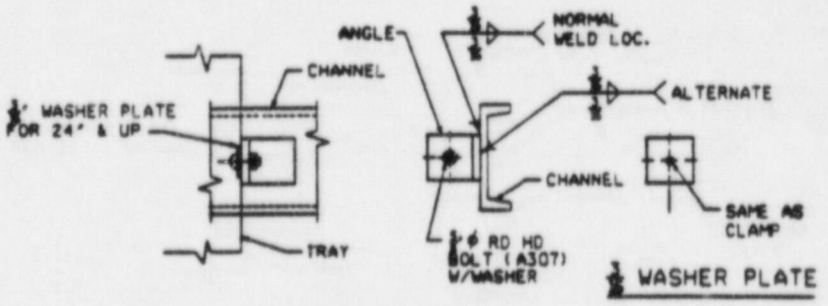
CALC NO M-10



TYPE 'P' CLAMP HEAVY DUTY



TYPE 'R' CLAMP HEAVY DUTY

REV	BY	DATE	CHECKED	DATE
1	[illegible]	1-20-82	bmn	1-29-82

TUGCO

CABLE TRAY - CLIP ANGLE STIFFNESS

IMPELL CORPORATION

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STIFFNESSES FOR CLIP TYPE "N" OR "P" OR "Q" OR "R"

CLIP TYPES N, P, Q AND R ARE DIFFERENT THAN MOST COMMON CLIP TYPES. THESE CLIPS HAVE ONE BOLT CONNECTION BETWEEN CABLE TRAYS AND CLIPS. IN CLIP TYPES P AND R, CLIP ANGLE ONE LEG IS WELDED TO HORIZONTAL MEMBER AND OTHER CLIP ANGLE LEG WILL ACT THE SIMILAR WAY OF CLIP B PLATE. THESE CLIPS WITH HORIZONTAL MEMBER HAS MOMENT RESISTING CAPACITY ABOUT Z-AXIS WHICH WILL CREAT PRYING ACTION. CLIP TYPES N AND Q ARE SIMILAR TO CLIP TYPE D. CLIP TYPE D STIFFNESSES WILL BE CONSERVATIVE VALUES THAN CLIP TYPE B.

CONCLUSION: FOR TYPE N OR P OR Q OR R STIFFNESSES USE CLIP TYPE D OR M STIFFNESS VALUES

REV	BY	DATE	CHECKED	DATE
1	[illegible]	1.20.86	GMR	1.24.86

TUGCO

CABLE TRAY-CLIP ANGLE STIFFNESS

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1	[illegible]	1-20-88	[illegible]	1-24-88

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TUGCO

CABLE TRAY - CLIP ANGLE STIFFNESS

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4.0 CABLE TRAY CLIP STIFFNESSES SUMMARY TABLE

W = WIDTH OF CABLE TRAY H = HEIGHT OF SIDERAIL

CLIP TYPE	CABLE TRAY SIZE	TRANSLATIONAL STIFFNESS - K/IN			ROTATIONAL STIFFNESS K-IN/RAD		
		K_x	K_y	K_z	K_{xx}	K_{yy}	K_{zz}
"A" - NON RELEASED	36 x 6	3.60	0.70	7.1E3	570.0	2.3E3	29.0
	30 x 6	↓	↓	↓	390.0	1.6E3	↓
	24 x 6	↓	↓	↓	250.0	1.0E3	↓
	18 x 6	↓	↓	↓	140.0	580.0	↓
	36 x 4	8.50	1.70	↓	1.6E3	5.5E3	↓
	30 x 4	↓	↓	↓	1.2E3	3.8E3	↓
	24 x 4	↓	↓	↓	750.0	2.4E3	↓
	18 x 4	↓	↓	↓	420.0	1.4E3	↓
	12 x 4	↓	↓	↓	180.0	610.0	↓
	6 x 4	↓	↓	↓	40.0	150.0	↓
"A" - RELEASED	36 x 6	3.60	0.00	↓	0.00	2.3E3	0.00
	30 x 6	↓	↓	↓	↓	1.6E3	↓
	24 x 6	↓	↓	↓	↓	1.0E3	↓
	18 x 6	↓	↓	↓	↓	580.0	↓
	36 x 4	8.50	↓	↓	↓	5.5E3	↓
	30 x 4	↓	↓	↓	↓	3.8E3	↓
	24 x 4	↓	↓	↓	↓	2.4E3	↓
	18 x 4	↓	↓	↓	↓	1.4E3	↓
	12 x 4	↓	↓	↓	↓	610.0	↓
	6 x 4	↓	↓	↓	↓	150.0	↓

REV	BY	DATE	CHECKED	DATE
1	[illegible]	1-27-86	[illegible]	1-31-86

IMPELL CORPORATION

TUGCO

CABLE TRAY - CLIP ANGLE STIFFNESS

JOB NO 0210-040

CALC NO M-10

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4.0 CABLE TRAY CLIP STIFFNESSES SUMMARY TABLE

W = WIDTH OF CABLE TRAY H = HEIGHT OF SIDE RAIL

CLIP TYPE	CABLE TRAY SIZE	TRANSLATIONAL STIFFNESS - K/IN			ROTATIONAL STIFFNESS K-IN/RAD		
		K_x	K_y	K_z	K_{xx}	K_{yy}	K_{zz}
"B" - $\frac{3}{8}$"	W X H	4.3E4	1.2E4	7.1E3	1.4E5	7.7E5	5.4E4
- $\frac{1}{2}$"	W X H	5.9E4	1.7E4	↓	2.0E5	1.1E6	7.9E4
"C" - $\frac{1}{8}$" THICK NON RELEASED	36 X H	5.40	120.0	790.0	1.2E4	3.4E3	12.0
	30 X H	↓	↓	↓	↓	2.3E3	↓
	24 X H	↓	↓	↓	↓	1.5E3	↓
	18 X H	↓	↓	↓	↓	830.0	↓
	12 X 4	↓	↓	↓	↓	360.0	↓
	6 X 4	↓	↓	↓	↓	80.0	↓
RELEASED	36 X H	↓	0.0	380.0	0.0	3.4E3	0.0
	30 X H	↓	↓	↓	↓	2.3E3	↓
	24 X H	↓	↓	↓	↓	1.5E3	↓
	18 X H	↓	↓	↓	↓	830.0	↓
	12 X 4	↓	↓	↓	↓	360.0	↓
	6 X 4	↓	↓	↓	↓	80.0	↓
- $\frac{3}{16}$" THICK NON RELEASED	36 X H	18.0	250.0	1.5E3	1.1E4	1.1E4	36.0
	30 X H	↓	↓	↓	↓	7.8E3	↓
	24 X H	↓	↓	↓	↓	5.0E3	↓
	18 X H	↓	↓	↓	↓	2.8E3	↓
	12 X 4	↓	↓	↓	↓	1.2E3	↓
	6 X 4	↓	↓	↓	↓	280.0	↓

4.0 CABLE TRAY CLIP STIFFNESSES SUMMARY TABLE

W = WIDTH OF CABLE TRAY H = HEIGHT OF SIDERAIL

CLIP TYPE	CABLE TRAY SIZE	TRANSLATIONAL STIFFNESS - K/IN Kx	Ky	Kz	ROTATIONAL STIFFNESS K-IN/RAD Kxx	Kyy	Kzz
3/16" THICK RELEASED	36XH	18.0	0.00	740.0	0.00	1.1E4	0.00
	30XH					7.8E3	
	24XH					5.0E3	
	18XH					2.8E3	
	12X4					1.2E3	
	6X4					2800	
"D" OR "M" - 5/16 ANGLE	WXH	580.0	150.0	7.1E3	5.7E4	1.0E4	630.0
5/8" ANGLE	WXH	980.0	250.0		6.9E4	1.8E4	1.1E3
"DA" OR "L" OR "LA" - 1/2" LEG	WXH	6.5E3	730.0		9.8E4	2.3E5	3.0E3
- 7/16 LEG	WXH	980.0	260.0		7.2E4	1.8E4	1.1E3
"G" - RELEASED	WX6	170.0	0.00	1.0E4	0.0	2.8E4	0.00
	WX4	550.0				9.9E4	
"T"	WXH	8.9E3	3.0E4	7.1E3	3.0E5	1.6E6	2.7E5

REV	BY	DATE	CHECKED	DATE
1	[illegible]	1-27-88	[illegible]	1-27-88

IMPELL CORPORATION

TUGCO

CABLE TRAY - CLIP ANGLE STIFFNESS

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LOCAL AXIS DEFINATION -

X = DERIVE FROM Y-AXIS AND Z-AXIS

Y = PARALLEL TO TRAY DIRECTION

Z = PERPENDICULAR TO TRAY DIRECTION IN THE SAME PLANE

NOTE :

1) FOR CLIP TYPE 'E' OR 'H' OR 'K' STIFFNESSES, USE CLIP TYPE 'T' STIFFNESS VALUES
2) FOR CLIP TYPE 'F' STIFFNESSES, USE CLIP TYPE 'D' STIFFNESS VALUES
3) FOR CLIP TYPE 'J' STIFFNESSES, USE CLIP TYPE 'B' STIFFNESS VALUES
4) FOR CLIP TYPE 'N' OR 'P' OR 'Q' OR 'R', USE CLIP TYPE 'D' STIFFNESS VALUES

REV	BY	DATE	CHECKED	DATE
1	[illegible]	1.20.82	[illegible]	1.26.82

TUGCO

CABLE TRAY-CLIP ANGLE STIFFNESS

IMPELL CORPORATION

JOB NO 0210-040

CALC NO 10-10

5.0 REFERENCES

1. "MANUAL OF STEEL CONSTRUCTION-AISC" 8TH EDISION
2. "DESIGN OF WELDED STRUCTURES" BY BLODGETT
3. "FORMULA FOR STRESS & STRAIN" BY ROARK 5TH EDISION
4. IMPELL CALC. NO. 0210-0360.P1 REV 0
5. TUGCO DRAWINGS—

a)	SS-1077-193	REV. 4
	SS-1077-194	REV. 4
	CTH-2-9770	REV. 1
b)	CTH-2-11615	REV. 1
	CTH-2-11616	REV. 1
	CTH-2-11617	REV. 1
c)	CTH-2-9763	REV. 1
	CTH-2-11753	REV. 1
d)	CTH-2-11751	REV. 1
	CTH-2-11756	REV. 1
e)	CTH-2-9760	REV. 1
	CTH-2-11500	REV. 1
f)	CTH-2-11738	REV. 1
	CTH-2-11740	REV. 1
g)	CTH-2-11741	REV. 1
	CTH-2-11744	REV. 1
h)	CTH-2-11743	REV. 1
j)	CTH-2-11483	REV. 2
k)	CTH-2-9774	REV. 1

6. GTSTRUDL VERSION 1983-83.04
7. EBASCO "SPECIFICATION FOR DYNAMIC TESTING"-TYPE "C" CLAMP DET
   ; ROC-T. WELK & R.F. OSBORNE 11.13.85 & J. VORDERBRUEGGEN & D. CORBRANDY

REV	BY	DATE	CHECKED	DATE
1	[illegible]	12-20-85	MC	12-12-85
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IMPELL CORPORATION

TUGCO

CABLE TRAY-CLIP ANGLE STIFFNESS

JOB NO 0210-040

CALC NO M-10

CALCULATION/PROBLEM COVER SHEET

Calculation/Problem No: M-12

Title: Qualification Procedures for Cable Tray Support Evaluation

Client: Tugco Project: CPSES

Job No: 0210-040-1355

Design Input/References:

as Noted Within

Assumptions:

As Noted Within

Method:

As Noted within

Remarks:

REV. NO.	REVISION	APPROVED	DATE
0	Original Issue	R. [illegible]	3/28/86
RECORD CENTER	Develop criteria for Torsional Buckling of angles and consider Eccentric Tray Clip Bolt holes on support Behavior	[illegible]	5/6/86

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2.2 Tray-to-Tier Connections	5
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3.0 Evaluation of Torsional Warping Stresses	7
3.1 Tier Members	7
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					Tugco
1	WLC	4-20-86	RVP	5-2-86	JOB NO 0310-040 / PAGE 2
0	WLC	3-27-86	RVP	3-27-86	IMPELL CORPORATION / CALC NO M-12 / OF 27

1.0 Purpose

Modeling and qualification criteria for cable tray support evaluations are provided in Tugco Project Instructions PI-02 Rev 0 and PI-03 Rev 0 (References [1] and [2]). The purpose of this calculation is to develop additional refinements to these criteria which will form the basis of revised project instructions. The scope of this calculation is limited to the following:

1) Modeling of support member connection details:

   a) tier to post connection details
   b) tray to tier details
   c) bracing angle connection details

2) Calculation of torsional stresses.

3) Evaluation of Composite members.

4) Allowable stresses in support members:

   a) channels
   b) angles

2.0 Member Connection Details

2.1 Tier and Post Connection Details

Tier and post members are typically composed of channel members welded back-to-back as shown in Figure 1. Since the centroids and shear centers of these members are eccentric, the transfer of forces through this connection results in the generation of additional force couples acting on the structure. The behavior of this connection detail and acceptable modeling criteria are described in this section.

Typically the cable tray will impose forces and moments in three orthogonal directions as shown in Figure 1. The axial force in the tier member acts through its centroid. Since the tier and the post are eccentric, this force will create a torsional moment about the post. The moment arm is equal to the distance between the shear center of the post and the centroid of the tier.

Table 1 summarizes the net eccentricities between the shear center of the post and the centroid of the tier for typical members. Note that in all cases the net eccentricity is less than 0.27 inches. This eccentricity is very small in comparison to the overall dimensions of typical support structures and can therefore be neglected.

The vertical force from the tray (Fy in Figure 1) acts through the centroid of the tier member. This force is transferred to the post creating weak axis bending in the post. The moment arm in this case is equal to the distance between the centroids of the tier and post members. This moment arm is significant in most cases and must be considered in the model.

The eccentricity between the tier and the post for Fy can be modeled using rigid links as shown in Figure 2a. Alternatively, this eccentricity can be modeled by placing the tray load eccentric from the tier and modeling the tier and post using concentric beam elements as shown in Figure 2b. These two models produce equivalent forces in the post members. The latter model will be used since it utilizes fewer nodal points. The modeling of the tray and post connection is described in detail in the following section.

The force acting about the Z axis in Figure 1 does not create any additional moments due to eccentricities and therefore it does not require further attention.

2.2 Tray to Tier Connection

The connection of the tray to the support structure will be modeled as shown in Figure 3. Flex-element No. 1 in Figure 3 has only Y stiffness. This element is model with an eccentricity as described in the previous section. To envelop the eccentricity between the post and tier and the eccentricity between tier and tray, the eccentricity used in the model should be taken to be the larger of the following values:

1) The distance between the centroids of the tier and post.

2) The distance between the centroid and the shear center of the tier.

Since the former value will generally govern, the resulting torsion in the tier will be conservative. Methods for reducing this conservatism are described in Section 3.1. All other forces and moments are transmitted through flex-element No. 2 which has finite stiffnesses about all axes except for the Y axis.

Stiffness values of the flex-elements should be based on the properties of the tray clips (when these become available) as described in Project Instruction No. 2. It is recommended that the stiffnesses of nonactive directions be modeled with one-hundredth of the corresponding stiffnesses of the active directions to avoid numerical stability problems.

Effects of eccentricities between the centroid of the tier member and the clip connection point on weak axis bending of the tier is discussed in Appendix G, 1

2.3 Bracing Angles

Bracing angles, when provided, are connected to the post either by gusset plates as shown in Figure 4a or by a direct weld to the web of the post as shown in Figure 4b.

Gusset plate connections will be modeled with Mx and My moments released (see Figure 4a) since the gussets are flexible for bending in these directions. All other forces and moments will be restrained. (Note to maintain stability, Mx should be released at only one end of the member.) Eccentricities will not be considered.

Bracing angles which are directly welded to the webs of the posts will be modeled as restrained for all forces and moments. The Ebasco procedure however models the bracing angles as pinned-end members. The basis for deviating from the Ebasco procedure is the study contained in Appendix B which shows that moments induced in the angle members can be significant due to the fixity of the connection.

The most significant eccentricity for those bracing angles which are welded directly to post members is the eccentricity between the shear center of the post and the centroid of the angle. Table 2 summarizes this eccentricity for typical post and bracing angle members. Note that these eccentricities are relatively small and will therefore be neglected. The effect of this eccentricity is also examined in Appendix B.

3.0 Evaluation of Torsional Warping Stresses

Torsional warping stresses can be significant in any open thin walled cross section in which the plate elements of the cross section are not joined at a single point (see Ref [3]). For example, tube sections, angles, tees, and x-shaped sections do not have significant warping stresses while channels and I-shapes can have significant warping stresses. For a discussion of torsional warping behavior see Refs [3], [4], and [5].

This section describes a conservative methodology for calculating torsional warping stresses in cable tray support members.

3.1 Tier Members

Torsion is introduced in the tier as a result of the eccentric cable tray loads as shown in Figure 5.

It is not possible to obtain accurate closed-form solutions for warping stresses in the tier members due to the complex boundary conditions at the tier to post connections. At these connections, warping is restrained only in the web of the tier member as shown in Figure 6. Closed-form solutions are available only when warping deformations of the cross-section are fully restrained or totally unrestrained at the boundaries. The partial warping restraint at the boundary of the tier will result in greatly reduced warping stresses (warping stresses are only developed when warping deformations are restrained). Warping stresses in the flange at the boundary will be zero since the flange is free to warp.

A conservative method of evaluating warping stresses in the tier member is as follows:

1) Calculate warping stresses at the ends of the tier member in the web assuming that warping is fully restrained. These stresses can be obtained using Ref [4] or Ref [6]. Stresses in the flange can be neglected at this location.

2) Calculate warping stresses at the load point assuming the ends are free to warp using Ref [4] or Ref [6]. Stresses in the web and in the flange must be considered.

3) Combine the maximum warping stresses with the stresses obtained from bending and axial forces and compare these stresses to the allowable stresses established in Ref [2].

4) Since the maximum warping stresses do not necessarily occur at the same location as the maximum bending stresses, conservatism can be reduced in Step 3 by summing stresses at individual locations in the member.

Alternatively, warping stresses can be calculated using the conservative and simplified approach use in Intersupport (see Appendix D).

As noted in Section 2.2, the tray load in the vertical direction will generally be modeled with an eccentricity equal to the distance between the centroids of the tier and post members. Since the actual eccentricity for torsion in the tier is the distance from the centroid to the shear center of the tier, the resulting torsional moments in the tier will be conservative. Therefore, if required, torsional stresses in the tier can be reduced using actual eccentricities in a hand evaluation.

3.2 Post Members

Closed form solutions for warping stresses in the post members are also difficult to obtain due to the complex geometry. Warping stresses can conservatively be estimated by modeling the post as a cantilever with concentrated torsional moments applied at the tiers as shown in Figure 7.

Warping should be considered to be fully restrained at the supports if the web and both flanges are restrained as shown in Figure 8a. In such cases warping stresses should be evaluated in both the web and flanges and combined with bending and axial stresses for evaluation.

If only the web is supported, as shown in Figure 8b, then warping in the post will be partially restrained. This will result in greatly reduced warping stresses. In such cases, warping can conservatively be evaluated by neglecting warping stresses in the flange and considering warping stresses to occur only in the web. For this evaluation the support should be considered to be fully restrained from warping.

The following is a summary of procedure for evaluating warping stresses in the post:

1) If the web and flanges are restrained at the support, calculate warping stresses in the web and flanges using Ref [4] or Ref [6]. Combine warping stresses with bending and axial stresses and compare with allowables.

2) If the flanges are not restrained at the support, calculate warping stresses in the web assuming full restraint. Neglect warping in the flanges. Combine the maximum warping stresses with the maximum bending and axial stresses and compare to allowables.

3) If the flanges are not restrained from warping, then the maximum warping stresses will occur in the web. Maximum bending stresses will occur at the end of the flanges. Thus conservatism can be reduced in Step 2 if the stresses are combined individually at these locations.

Alternatively, warping stresses can be calculated using the conservative and simplified approach use in Intersupport (see Appendix D).

3.3 Bracing Angles

As described at the beginning of this section, angle members do not develop significant warping stresses and therefore do not require further consideration. However, it should be noted that pure torsional stresses still require evaluation.

4.0 Evaluation of Composite Sections

Composite channel sections as shown in Figure 9 are frequently used. Evaluations of these members can be simplified by neglecting composite action assuming that each channel acts only in strong axis bending. For example, stresses due to x-directional loading in Figure 9 can be calculated using the strong axis section modulus of channel number 1.

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5.0 Allowable Stresses

5.1 Material Properties

Unless otherwise noted, structural steel can be assumed to be A-36 with a yield stress of 36 ksi at 70 degrees. Ambient temperature in the Safeguards building is 120 degrees. Per Ref [7], there is approximately a 2 percent reduction in strength at this temperature. This reduction can reasonably be neglected in support evaluations.

5.2 Allowable Stresses in Channel Members

Allowable stresses in channel members shall be based on the criteria provided in Refs [2] and [8].

5.3 Allowable Stresses in Angles

Allowable stresses in angles shall be based on the criteria in Refs [2] and [8] provided that sections bent about their major axis are braced laterally in the region of compression stress at intervals not exceeding $76b_f/\sqrt{F_y}$ (see Ref [8]). If this requirement is not satisfied, then allowable bending stresses shall be based on the criteria contained in Appendix C.*

* Stresses in angle members will be calculated using geometric axis properties (axes parallel to the legs of the member). Stresses about the principal axes will be somewhat larger. To account for this, stresses calculated about the geometric axes shall be multiplied by 1.2 (see Appendix F).

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5.4 Torsional Buckling of Angle Members

Torsional buckling of thin walled members with zero warping rigidity such as angles is closely related to local plate buckling (see Reference [9]). For most cross sections, the torsional buckling capacity is greater than the flexural buckling capacity. Therefore, torsional buckling is rarely checked. Since the warping constant (C_w) of angles is zero, some angles may be susceptible to torsional buckling. This section establishes criteria for determining when torsional buckling of angles should be evaluated.

Torsional buckling is generally evaluated by establishing an equivalent radius of gyration for torsional buckling for use in flexural buckling equations (see Reference [9]). The equivalent radius of gyration for torsional buckling is given by the following equation (see Reference [9]):

$$\left(1 - \frac{y_o^2}{r_{ps}^2}\right) r_{tb}^2 - \left(r_y^2 + r_t^2\right) r_{tb}^2 + r_y^2 r_t^2 = 0$$

where: r_y = maximum principal axis of symmetry

r_{tb} = equivalent radius of gyration

r_{ps} = polar radius of gyration

$$r_t = \frac{C_w + 0.04 J (KL)^2}{I_{ps}}$$

$C_w = 0.$ for angles

J = torsion constant

I_{ps} = polar moment of inertia about shear center

y_o = distance from shear center to centroid

Equivalent torsional radii of gyration for selected angles are calculated in Table 3 for 3.5 foot members and in Table 4 for 5 foot members. These values are compared to the corresponding flexural radii of gyration so that it can be determined which failure mode governs. Note that torsional buckling tends to be more critical for shorter length members and for sections with larger b/t ratios. This is due to the similarity of torsional buckling to local plate buckling. Members shorter than 3.5 feet are not evaluated since their use is not practical.

Note that provided the leg dimension is less than or equal to 3.5 inches, the equivalent radii of gyration for torsional buckling are greater than (or are not significantly less than) the flexural radius of gyration.

Based on these results, torsional buckling of angle members will not be evaluated if the leg size is less than or equal to 3.5 inches. Torsional buckling of larger cross sections will be evaluated.

6.0 References

[1] Tugco Project Instructions PI-02, "Dynamic Analysis of Cable Tray Systems", Rev 0, Job No. 0210-040.

[2] Tugco Project Instructions PI-03, "Qualification of Cable Tray Supports", Rev 0, Job No. 0210-040.

[3] K. Zbirohowski-Koscia, "Thin Walled Beams from Theory to Practice", Crosby Lockwood & Son LTD London, 1967.

[4] Torsional Analysis of Rolled Steel Sections, Bethlehem Steel Corporation.

[5] Chen and Atsuta, "Theory of Beam-Columns", McGraw-Hill, 1977.

[6] General Instruction for Cable Tray Analysis for Comanche Peak Steam Electrical Station No. 2, August 26, 1985, Ebasco.

[7] ASME B&PV Code.

[8] Manual of Steel Construction, AISC, Eighth Edition.

[9] Gaylord and Gaylord, "Design of Steel Structures", McGraw-Hill Book Company, Second Edition

1

Table 1

Net Eccentricities* Between Typical Tier and Post Members for Fx Loading
(See Figure 1)

Tier Member	Post Member	Distance to Centroid of Tier	Distance to Shear Center of Post	Net Eccentricity
C 4x7.25	C 6x8.2	.459	.599	.140
C 8x11.5	C 10x15.3	.571	.796	.225
C 6x8.2	MC 6x12	.512	.725	.213
C 6x8.2	C 6x8.2	.512	.599	.087
C 6x8.2	C 8x11.5	.512	.697	.185
C 4x7.25	C 4x7.25	.459	.386	.073
C 4x7.25	MC 6x12	.459	.725	.266

*See appendix E for a discussion of the effects of this Eccentricity.

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Table 2

Eccentricities Between Typical Post and Bracing Angle Members

Bracing Member	Post Member	Distance to Centroid of the Bracing Angle	Distance to Shear Center of Post	Net Eccentricity
L 3x3x3/8	C 10x15.3	.888	.796	.092
L 3x3x3/8	MC 6x12	.888	.725	.163
L 3x3x3/8	C 6x8.2	.888	.599	.289
L 3x3x3/8	C 8x11.5	.888	.697	.191
L 3x3x3/8	C 4x7.25 *	.888	.386	.502

* Note: This section is rarely used as a post. When it is used as a post it generally does not have a bracing angle.

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Table 3 Equivalent Torsional Radii of Gyration for 3.5 ft member

Section	y (in)	A (in^2)	Iy (in^4)	I_z (in^4)	Ip (in^4)	r_z (in)	r_{max} (in)	J (in^4)	r_t (in)	r_{ps} (in)	rtb (in)
L6x6x3/8	1.640	4.360	15.400	15.400	44.754	1.190	2.377	.211	.58	3.20	.39
L4x4x1/2	1.180	3.750	5.560	5.560	14.588	.782	1.534	.333	1.27	1.97	.88
L4x4x3/8	1.140	2.860	4.360	4.360	12.067	.788	1.558	.141	.91	2.05	.71
L4x4x1/4	1.090	1.940	3.040	3.040	8.818	.795	1.582	.042	.58	2.13	.50
L3.5x3.5x1/4	.97	1.69	2.01	2.01	5.762	.69	1.377	.036	.67	1.85	.61
L3x3x3/16	.82	1.09	.96	.96	2.796	.60	1.187	.013	.58	1.60	.61
L3x3x3/8	1.160	2.110	2.110	2.110	6.820	.587	1.287	.105	1.04	1.80	.81
L2x2x3/8	.638	1.360	.479	.479	1.146	.389	.744	.070	2.08	.92	.89
L2x2x1/4	.592	.938	.348	.348	.915	.391	.768	.021	1.27	.99	.93

Assumptions:

Length (ft): 3.5

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Table 4 Equivalent Torsional Radii of gyration for 5.0 ft members

Section	y (in)	A (in^2)	Iy (in^4)	I_z (in^4)	Ip (in^4)	r_z (in)	r_{max} (in)	J (in^4)	r_t (in)	r_{ps} (in)	r_{tb} (in)
L6x6x3/8	1.640	4.360	15.400	15.400	44.754	1.190	2.377	.211	.82	3.20	.53
L4x4x1/2	1.180	3.750	5.560	5.560	14.588	.782	1.534	.333	1.81	1.97	1.02
L4x4x3/8	1.140	2.860	4.360	4.360	12.067	.788	1.558	.141	1.30	2.05	.88
L4x4x1/4	1.090	1.940	3.040	3.040	8.818	.795	1.582	.042	.82	2.13	.66
L3.5x3.5x1/4	.97	1.69	2.01	2.01	5.762	.69	1.377	.036	.95	1.85	.77
L3x3x3/16	.82	1.09	.96	.96	2.796	.60	1.187	.013	.82	1.60	.76
L3x3x3/8	1.160	2.110	2.110	2.110	6.820	.587	1.287	.105	1.49	1.80	.94
L2x2x3/8	.638	1.360	.479	.479	1.146	.389	.744	.070	2.97	.92	.88
L2x2x1/4	.592	.938	.348	.348	.915	.391	.768	.021	1.81	.99	.90

Assumptions:

Length (ft): 5.0

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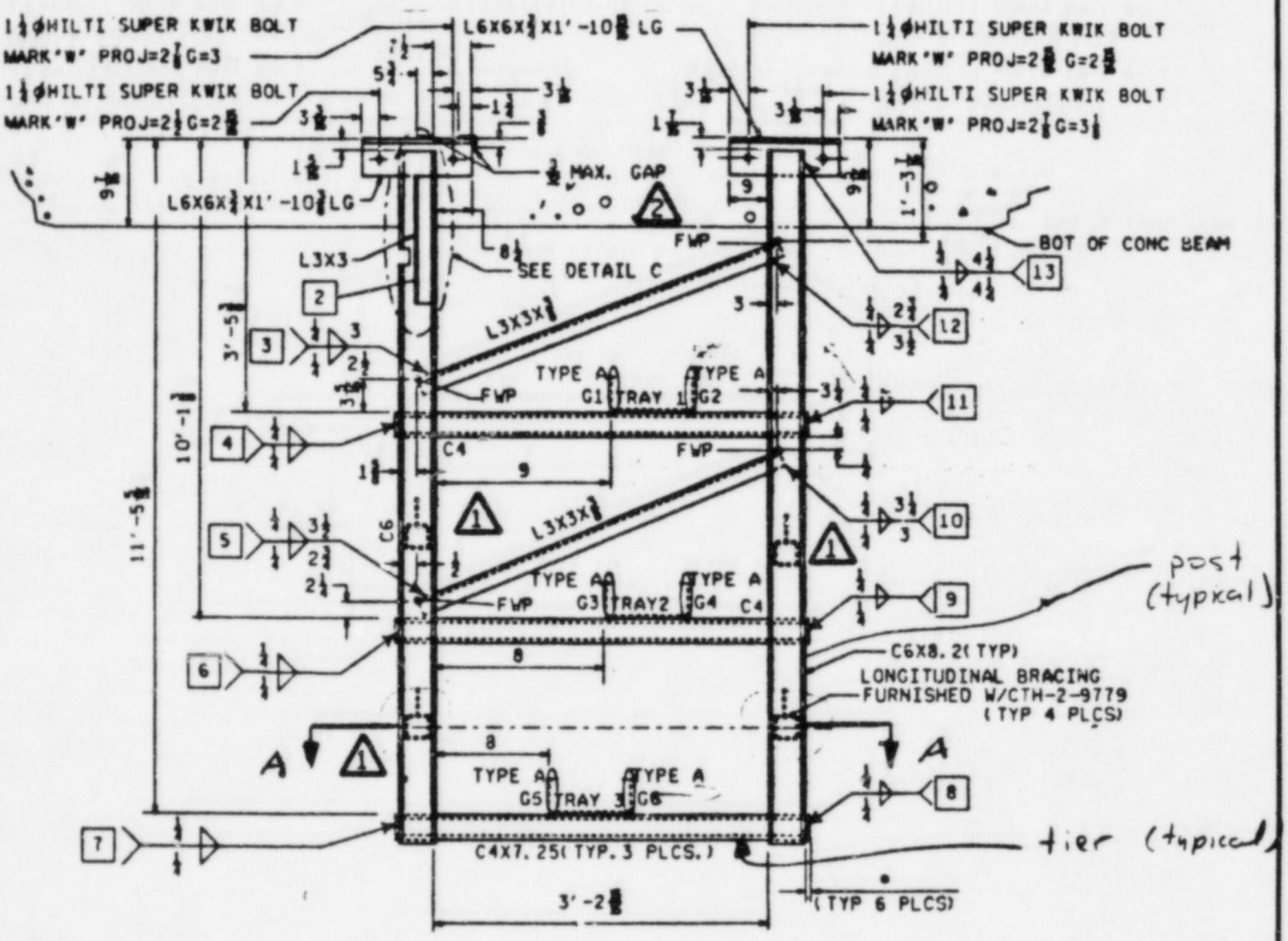

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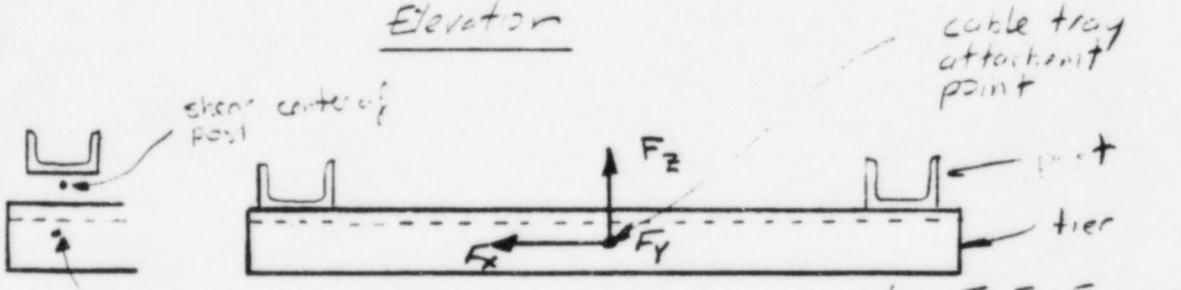

A – A

Note. F_x, F_y, F_z are imposed cable tray forces

Figure 1 Typical Trapeze Support

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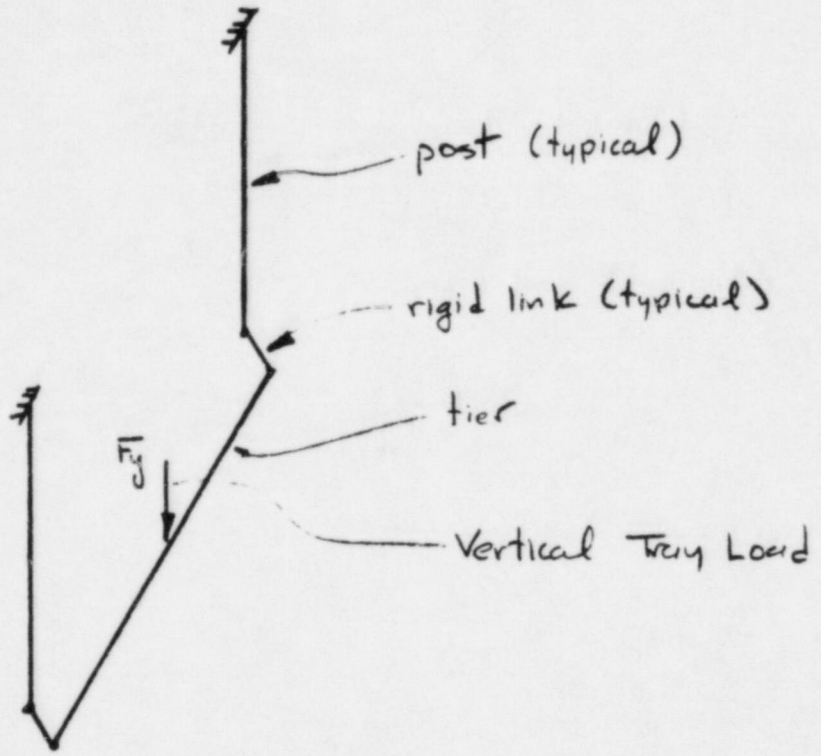

Figure 2a – Support Model with Rigid Links at Tier-to-Post Connects to Account for Eccentricities

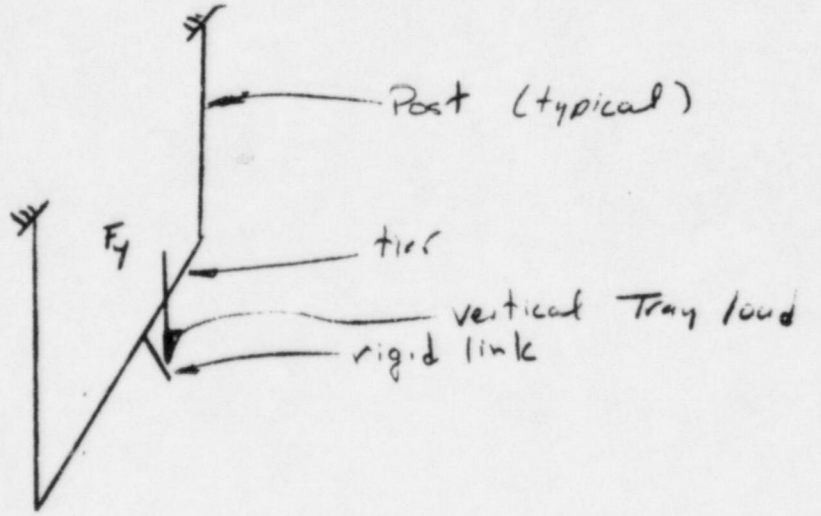

Figure 2b – Alternative Model of Tier-to-Post Eccentricity

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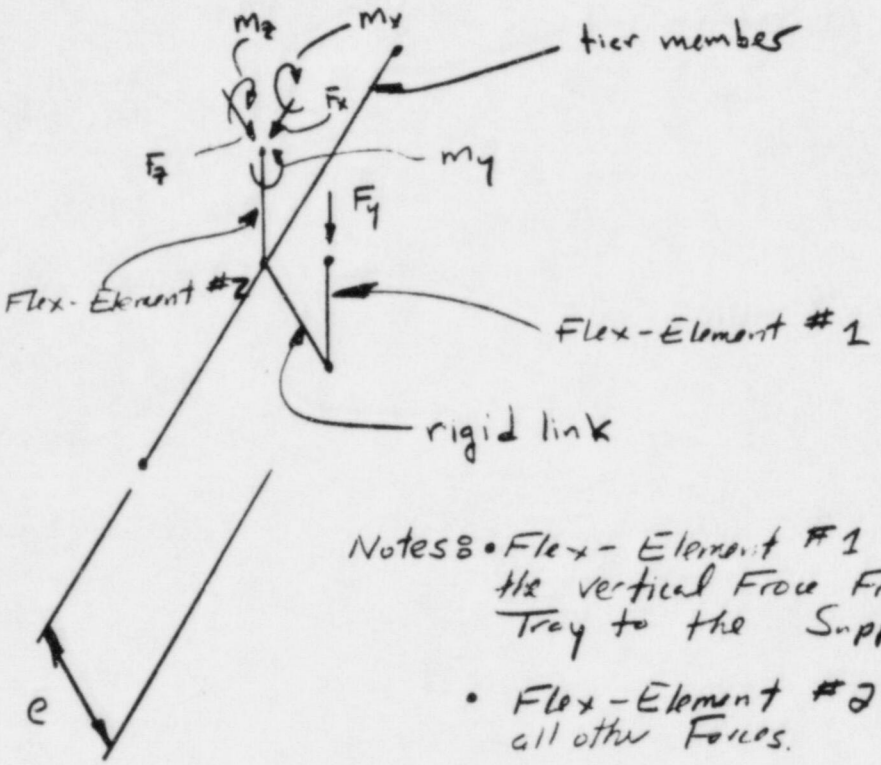

Notes: • Flex-Element #1 Transmits the vertical Force From the Tray to the Support

• Flex-Element #2 Transmits all other Forces.

Notation: F_x, F_y, F_z } Forces transmitted to the tier from the tray

M_x, M_y, M_z } moments transmitted to the tier from the tray

e: eccentricity between the centroid of the post and the tier.

Figure 3. Model of the Cable Tray Clip

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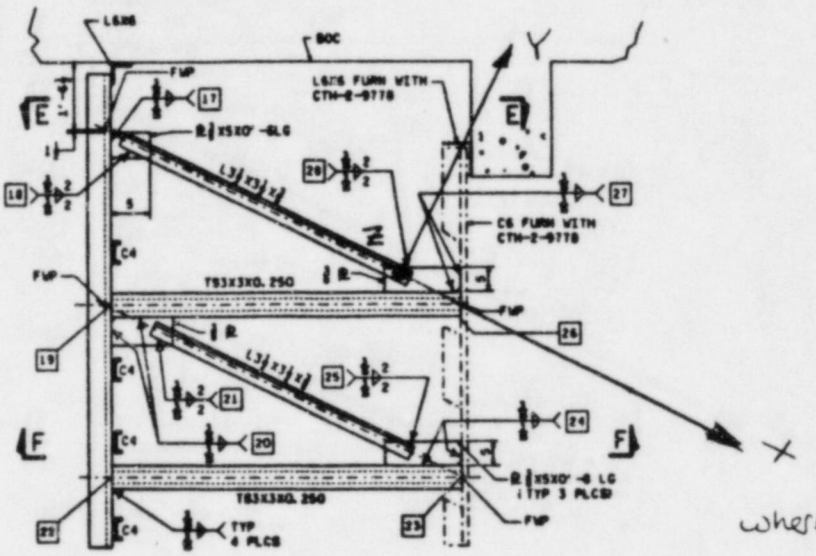

where x, y, z are local axes of the angle

Figure 4a Bracing Angles connected by Gusset Plates

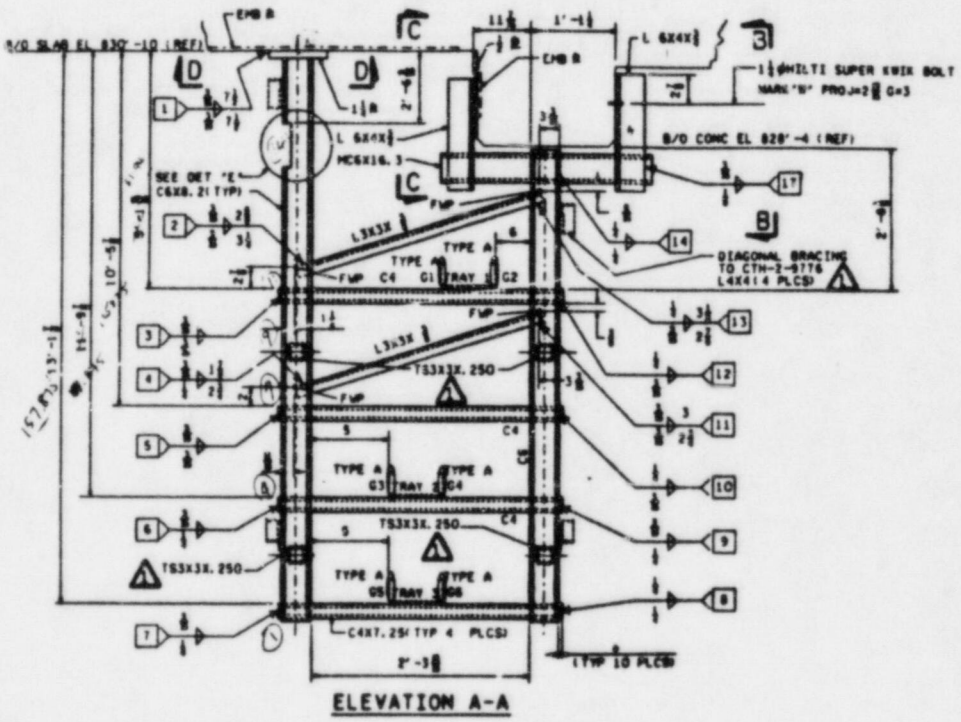

Figure 4b Bracing Angles Welded Directly to the Web of the Post

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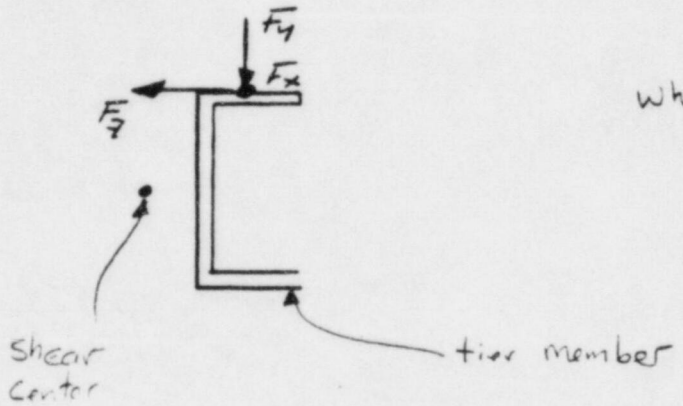

where F_x, F_y, F_z are tray loads

Note: F_y and F_z are eccentric from the shear center and therefore create torsion in the tier.

Figure 5. Torsional Loads in Tier members

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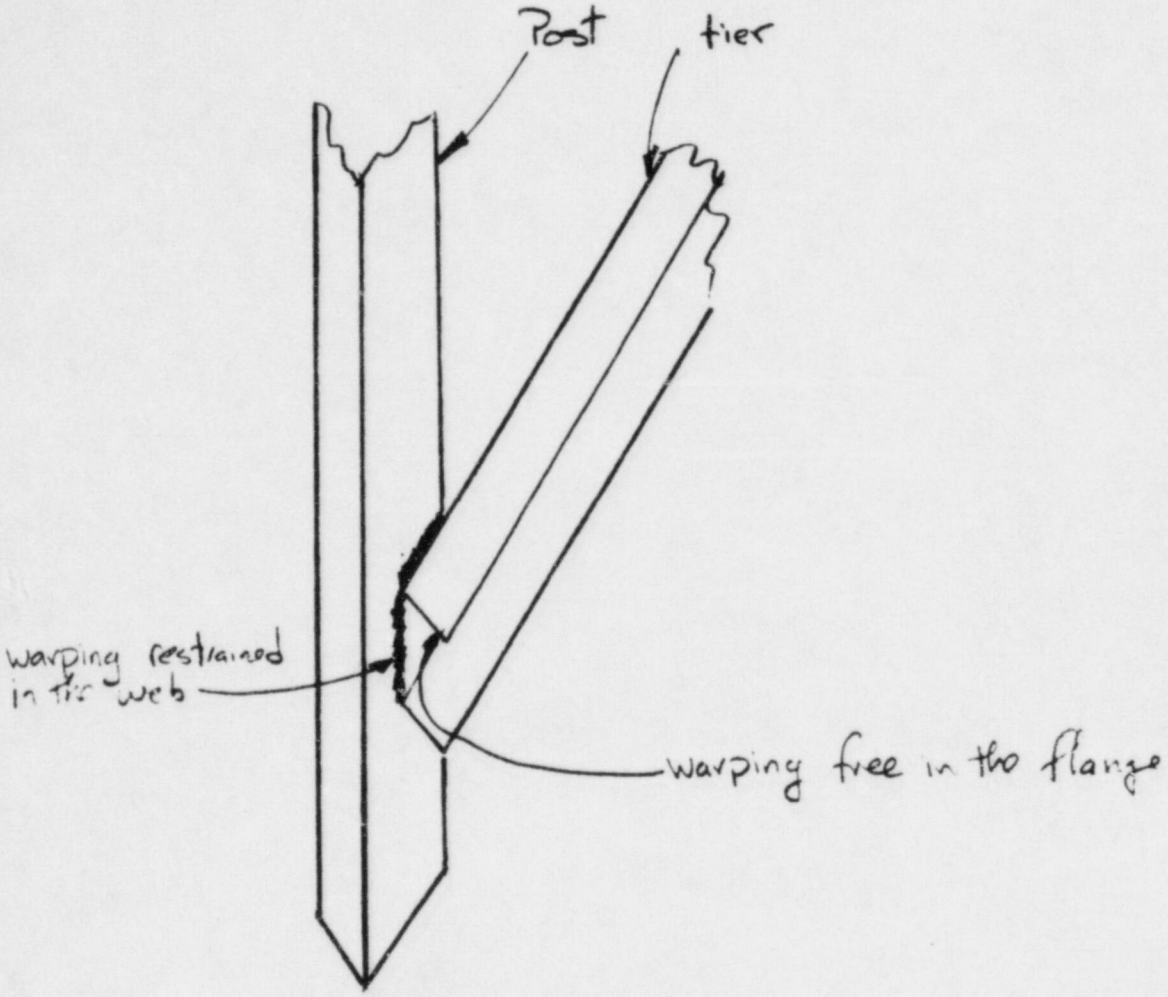

Figure 6. Warping End Conditions of Tier Members

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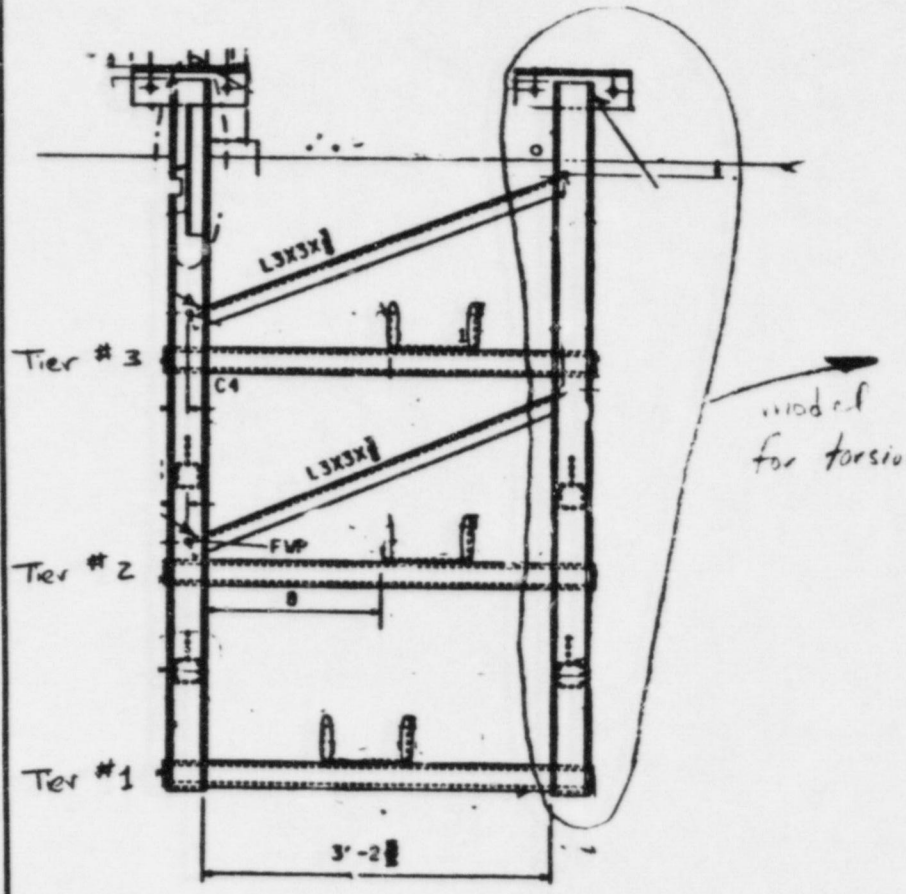

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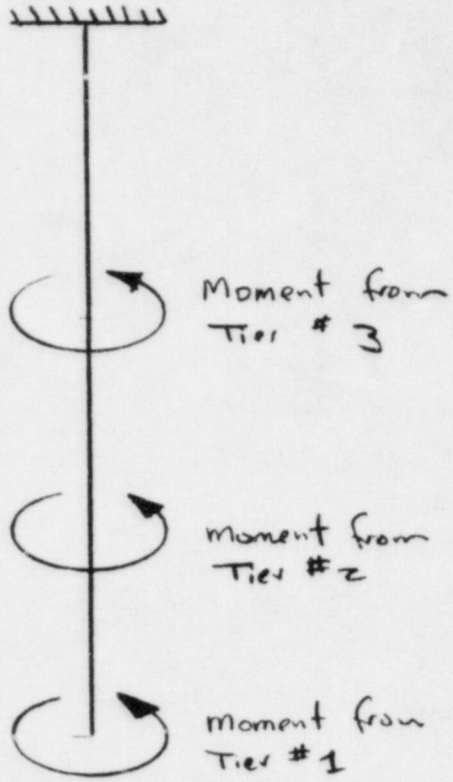

Model of the Rest for Torsion

Figure 7 Torsional Moments In Rest Members

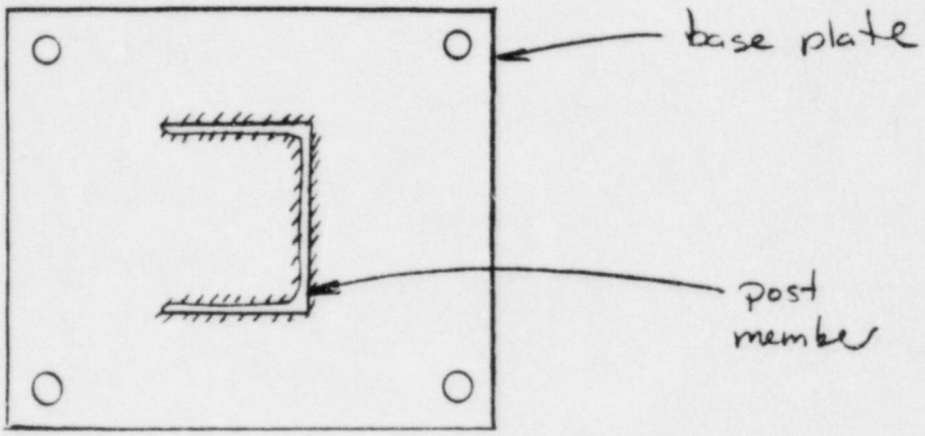

Figure 8a Post Member Welded Directly to a Base Plate (Warping Fully Restrained)

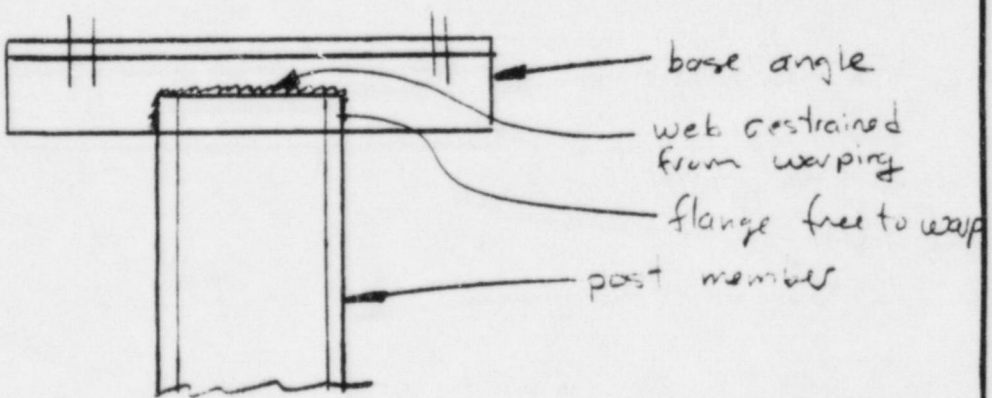

Figure 8b Post Member Welded to Base Angle (Warping Partially Restrained)

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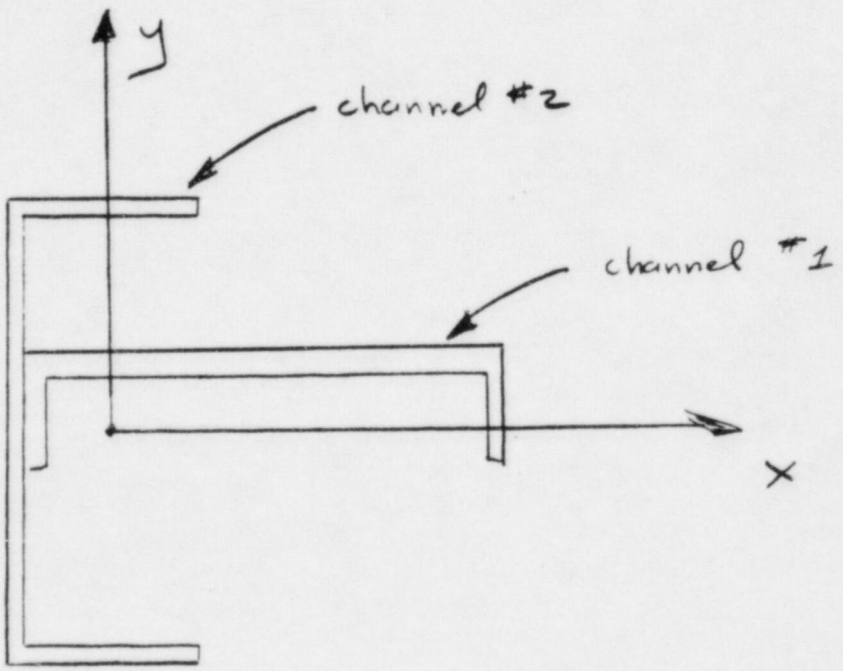

- For x-directional loading, calculate stresses using strong axis section modulus of channel #1
- For y-directional loading, calculate stresses using strong axis section modulus of channel #2

Figure 9 Composite Channel Sections

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Appendix A – Not Used

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APPENDIX B

BRACING ANGLE SENSITIVITY STUDY

TUGCO COMMANCHE PEAK UNIT 2

BRACING STUDY

PURPOSE: The purpose of this calculation is to determine the significance to the raceways and supporting structures stress evaluation of modeling the eccentricities of the cross bracing, and their end conditions

METHOD: Representative models are developed of support structures with and without the subject eccentricities and fixities. Results of with and without models are compared to each other to evaluate load and stress changes due to the eccentricity. The situations modeled are described below. Math models of each model are provided. Cases run include point forces, gravity in 3 separate directions and a response spectrum analysis.

EVALUATIONS

Cases are run with various eccentricities and member end releases,

Model 1

Hanger CTH-2-9762 (ref.1) is modeled with the connections to supplementary steel at 962A and 962H modeled as full anchors. All connections are fully fixed. The eccentricity of the brace is modeled as the distance from the C.G. of the 3×3×3/8 angle to the CG of the 6×8.2 channel (see backup calculations).

Model 2

As model 1 but the brace is modeled with no eccentricity considered.

Model 3

As model 1 but brace eccentricity is reduced to include only the distance from the back of the 6×8.2 channel to its C.G. The intent here is to eliminate unneeded conservatism of considering the angle eccentricity. See paragraph 4.15.3 of reference 3 for justification

Model 4

As model 1 but brace has rotations released on either end. This is in accordance with EBASCO procedures (ref. 4) sheet 2 I.C

Model 5

As model 2 but with end releases as in model 4.

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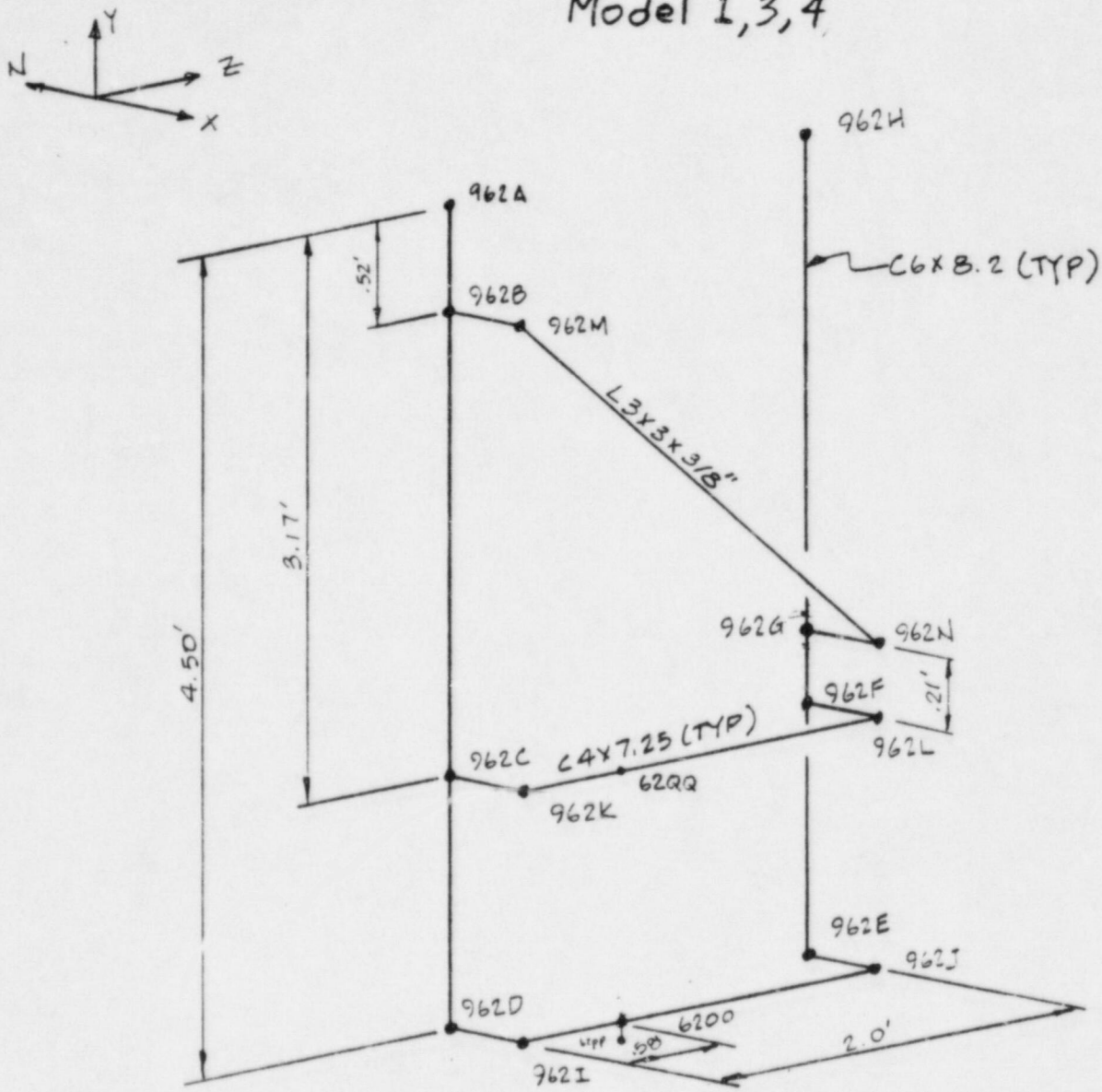
Models

Model 1,3,4



REFERENCE (1,3)

					TXU ELO - COMANCHE PEAK UNIT 2		
					BRACING TYP		
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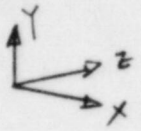
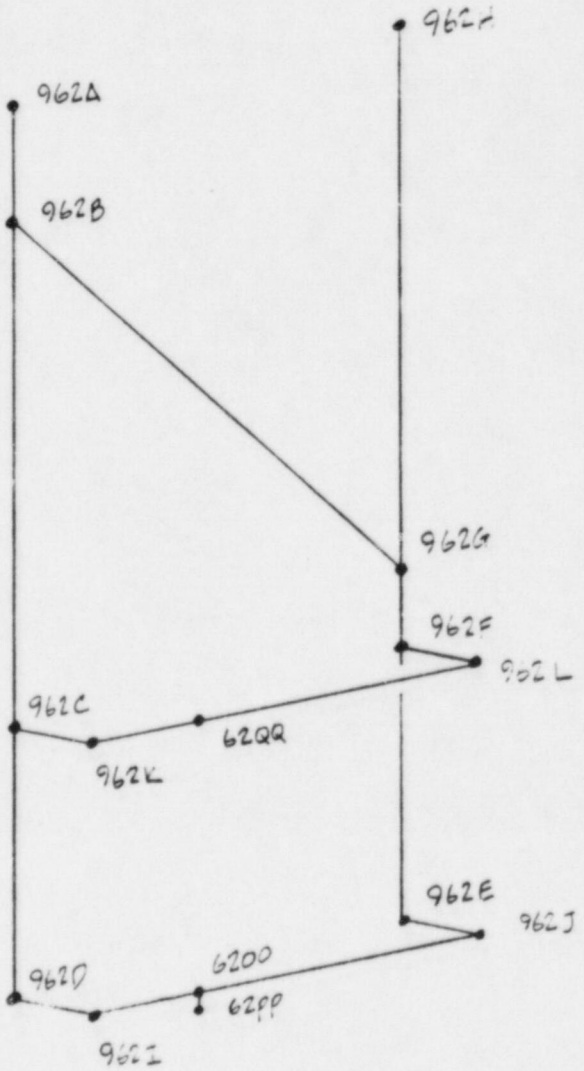
Model 2,5





REFERENCE (1,3)

					TXU ELECTRIC - COMANCHE PEAK UNIT 2	
					BRACING STUDY	
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COMPARISONS

Member Forces (Local)

DCP	LOCATION	MODEL	CASE	Fx. (LBS)	FY (LBS)	FZ (LBS)	MX (FT LBS)	MY (FT LBS)	MZ (FT LBS)
962N	BRACE BOTTOM	1	PFQX	119	-32	16	-5	36	55
962G	"	2	"	136	-2	16	-6	25	6
962N	"	3	"	135	-13	16	-6	30	24
"	"	4	"	27	0	0	0	0	0
"	"	1	PFQZ	-215	5	2	0	-14	-8
962G	"	2	"	-254	2	2	0	8	-4
962N	"	3	"	-251	3	2	0	-1	-5
"	"	4	"	-97	0	0	0	0	0.
"	"	1	SEIX	66	24	3	4	10	39
962G	"	2	"	27	1	2	4	14	2
962N	"	3	"	44	10	2	4	13	16
"	"	4	"	126	0	0	0	0	0
"	"	1	SEIZ	46	1	1	0	3	2
962G	"	2	"	56	1	0	0	0	1
962N	"	3	"	54	1	0	0	1	1
"	"	4	"	20	0	0	0	0	0

TUERO COMMANCHE PEAK
Bracing STUDY

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Member Forces (Local)

DCP	LOCATION	MODEL	CASE	Fx. (LBS)	FY (LBS)	FZ (LBS)	Mx (FT LBS)	My (FT LBS)	Mz (FT LBS)
962M	BRACE TOP	1	PFQX	119	-32	16	-5	-15	-48
962B	"	2	"	136	-2	16	-6	-24	1
962M	"	3	"	135	-13	16	-6	-19	-17
"	"	4	"	27	0	0	0	0	0
"	"	1	PFQZ	-215	5	2	0	-20	6
962B	"	2	"	-254	2	2	0	2	3
962M	"	3	"	-252	3	2	0	-8	4
"	"	4	"	-97	0	0	0	0	0
"	"	1	SEIX	66	24	3	4	16	38
962B	"	2	"	27	1	2	4	21	0
962M	"	3	"	44	10	2	4	20	14
"	"	4	"	126	0	0	0	0	0
"	"	1	SEIZ	46	1	1	0	5	1
962B	"	2	"	56	1	0	0	0	1
962M	"	3	"	54	1	0	0	2	1
"	"	4	"	20	0	0	0	0	0

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BRACING STUDY

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Member Forces (Local)

DCP	LOCATION	MODEL	CASE	Fx. (LBS)	FY (LBS)	FZ (LBS)	MX (FT LBS)	MY (FT LBS)	MZ (FT LBS)
9628 (L)	POST @ BRACE	1	PFQX	-87	49	129	-5	320	-17
"	"	2	"	-83	64	129	-5	322	-47
"	"	3	"	-86	60	129	-5	322	-38
"	"	4	"	-56	38	140	-4	347	7
"	"	1	PFQZ	-50	36	5	0	10	-48
"	"	2	"	-39	24	5	0	10	-28
"	"	3	"	-39	25	5	0	10	-30
"	"	4	"	-88	69	5	1	8	-101
"	"	1	SEIX	37	15	44	2	137	9
"	"	2	"	17	11	44	2	140	7
"	"	3	"	25	13	44	2	140	10
"	"	4	"	46	42	45	2	142	58
"	"	1	SEIZ	14	6	0	0	1	9
"	"	2	"	11	3	0	0	0	4
"	"	3	"	12	3	0	0	1	5
"	"	4	"	24	13	1	0	3	20

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Member Forces (Local)

DCP	LOCATION	MODEL	CASE	Fx. (LBS)	FY (LBS)	FZ (LBS)	Mx (FT LBS)	My (FT LBS)	Mz (FT LBS)
962G (L)	POST @ BRACE	1	PFQX	87	49	-72	-24	-37	-72
"	"	2	"	83	64	-72	-23	-36	-56
"	"	3	"	86	60	-72	-22	-36	-63
"	"	4	"	56	38	-62	-3	-36	-11
"	"	1	PFQZ	50	-165	5	-6	-3	-17
"	"	2	"	39	-177	5	-5	-3	-5
"	"	3	"	39	-176	5	-6	-3	-5
"	"	4	"	88	-132	5	-7	-5	-65
"	"	1	SEIX	38	15	29	5	36	47
"	"	2	"	18	11	29	5	35	16
"	"	3	"	26	13	29	5	36	28
"	"	4	"	47	41	28	8	36	48
"	"	1	SEIZ	14	23	0	1	0	2
"	"	2	"	11	26	0	0	0	2
"	"	3	"	12	26	0	0	0	1
"	"	4	"	24	16	1	1	1	13

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0	[illegible]	12-3-86	YKY	12-3-86

Member Forces (Local)

DCP	LOCATION	MODEL	CASE	Fx. (LBS)	Fy (LBS)	Fz (LBS)	Mx (FT LBS)	My (FT LBS)	Mz (FT LBS)
962M	BRACE TOP	4	PFQX	27	0	0	0	0	0
962B	"	5	"	166	0	0	0	0	0
962B (L)	POST AT BRACE TOP	4	"	-56	38	140	-4	347	7
"	"	5	"	-99	79	139	-3	344	-58
962G (L)	POST AT BRACE BOTTOM	4	"	56	38	-62	-3	-36	-11
"	"	5	"	99	79	-62	-4	-37	-63
962M	BRACE TOP	4	PFQZ	-97	0	0	0	0	0
962B	"	5	"	-241	0	0	0	0	0
962B (L)	POST AT BRACE TOP	4	"	-88	69	5	1	8	-101
"	"	5	"	-48	29	7	-1	14	-31
962G (L)	POST AT BRACE BOTTOM	4	"	88	-132	5	-7	-5	-65
"	"	5	"	48	-172	7	-1	-5	-13
962M	BRACE TOP	4	SEIX	126	0	0	0	0	0
962B	"	5	"	23	0	0	0	0	0
962B (L)	POST AT BRACE TOP	4	"	46	42	45	2	142	58
"	"	5	"	18	7	40	1	136	0

TUSCO COMMANCHE PEAK
BRACING STUDY

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Member Forces (Local)

DCP	LOCATION	MODEL	CASE	Fx. (LBS)	FY (LBS)	FZ (LBS)	Mx (FT LBS)	My (FT LBS)	Mz (FT LBS)
962G (L)	POST AT BRACE BOTTOM	4	SE1X	47	41	28	8	36	48
"	"	5	"	17	6	35	1	32	18
962M	BRACE TOP	4	SE1Z	20	0	0	0	0	0
962B	"	5	"	55	0	0	0	0	0
962B (L)	POST AT BRACE TOP	4	"	24	13	1	0	3	20
"	"	5	"	12	3	0	0	1	4
962G (L)	POST AT BRACE BOTTOM	4	"	24	16	1	1	1	13
"	"	5	"	12	26	0	0	1	2

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SUPPORT REACTIONS (GLOBAL)

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SUPPORT	DCP	MODEL	CASE	Fx (LB)	FY(LB)	Fz(LB)	Mx(FT LBS)	MY (FT LBS)	Mz(FT LBS)
62AS	962A	1	PFQX	145	25	-1	-65	6	423
"	"	2	"	145	23	-21	-35	6	420
"	"	3	"	145	26	-15	-48	7	421
"	"	4	"	140	-35	22	-4	-2	423
"	"	1	PFQZ	7	-219	169	-130	-2	9
"	"	2	"	7	-237	183	-120	-1	13
"	"	3	"	7	-236	182	-121	-2	11
"	"	4	"	5	-163	131	-169	-6	2
"	"	1	SEIX	49	29	8	43	8	185
"	"	2	"	49	4	6	5	8	184
"	"	3	"	49	15	7	20	8	184
"	"	4	"	48	52	38	39	8	177
"	"	1	SEIZ	1	50	46	31	0	1
"	"	2	"	0	55	49	29	0	1
"	"	3	"	1	54	49	29	0	1
"	"	4	"	1	39	38	39	2	3

SUPPORT REACTIONS (GLOBAL)

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SUPPORT	DCP	MODEL	CASE	Fx (LB)	FY(LB)	Fz(LB)	Mx(FT LBS)	MY (FT LBS)	Mz(FT LBS)
62HS	962H	1	PFQX	56	-25	1	15	-2	215
"	"	2	"	56	-23	21	-11	-2	218
"	"	3	"	56	-26	15	-4	-2	217
"	"	4	"	62	35	-22	75	-5	215
"	"	1	PFQZ	-7	219	32	-70	0	-9
"	"	2	"	-7	237	18	-43	0	-13
"	"	3	"	-7	236	19	-46	0	-11
"	"	4	"	-5	163	70	-143	0	-2
"	"	1	SEIX	45	28	9	17	1	150
"	"	2	"	45	3	6	4	1	152
"	"	3	"	45	14	8	10	1	153
"	"	4	"	48	50	39	68	1	166
"	"	1	SEIZ	1	50	12	16	0	2
"	"	2	"	0	55	8	9	0	1
"	"	3	"	1	54	9	10	0	1
"	"	4	"	2	39	20	31	0	4

Support Reactions (Global)

SUPPORT	DCP	MODEL	CASE	Fx (LB)	FY (LB)	Fz (LB)	Mx (FT LBS)	MY (FT LBS)	Mz (FT LBS)
62AS	962A	4	PFQX	140	-35	22	-4	-2	423
"	"	5	"	139	29	-26	-45	-3	417
62HS	962H	4	"	62	35	-22	75	-5	215
"		5	"	62	-29	26	-14	-4	221
62AS	962A	4	PFQZ	5	-163	131	-169	-6	2
"	"	5	"	7	-234	192	-125	-1	18
62AH	962H	4	"	-5	163	70	-143	0	-2
"	"	5	"	-7	234	19	-44	-1	-18
62AS	962A	4	SEIX	48	52	38	39	8	177
"	"	5	"	42	0	8	4	1	157
62AH	962H	4	"	48	50	39	68	1	166
"	"	5	"	57	1	8	5	1	198
62AS	962A	4	SEIZ	1	39	38	39	2	3
"	"	5	"	0	54	50	30	0	1
62AH	962H	4	"	2	39	20	31	0	4
"	"	5	"	0	54	8	9	0	1

TUGLO COMMANCHE PEAK
BRACING STUDY

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NATURAL MODE FREQUENCIES

MODEL	MODE 1	MODE 2
1	13.230	22.047
2	13.169	22.295
3	13.175	22.139
4	13.064	19.165

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TUGCO COMMANCHE PEAK UNIT2
BRACING STUDY

IMPELL CORPORATION

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OBSERVATIONS:

MODELS 1,2,3 - Differences due to eccentricity length

BRACES: Increased eccentricity lowers the axial load in the brace but increases the non torsional moments for static loadings Changes are roughly proportional to the amount of eccentricity modeled. For seismic analysis in the horizontal direction perpendicular to the support eccentricities increase the axial force and the inplane moment (local M_z). For the inplane direction the eccentricities decrease the axial force but increase the out of plane moment (local M_y) in the brace.

POSTS: Brace eccentricities increase the post axial force and the inplane (M_z) moment for statics and dynamics.

Supports: At the connections to supplementary steel the only notable changes are in the inplane force and moment (Global Fz; Mx) The inplane moment increases with increased eccentricity modeled for both static and dynamic cases. This is generally larger - for out of plane loadings.

Frequencies: Modes below 33hz are not significantly effected.

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OBSERVATIONS (CONT'D)

Effect of eccentricities on pinned braces Models 4 & 5

BRACES:

With the braces pinned the braces take only axial loads. The eccentricity in the braces lowers the axial load significantly in the static cases and the in plane seismic case. The out of plane seismic loads the brace more axially for the eccentric model.

Posts:

Adding eccentricities tends to increase the axial load in the post in all but the static out of plane loading. Other noted changes are to the out of plane force (F_y) and moment (M_z). Typically the introduction of brace eccentricities increases these loads except in the static out of plane loading (PFQX) where effects are reversed.

Supports:

Loads at attachment points are impacted somewhat by modeling the eccentricities at the braces. There is no clear trend as to the direction.

FREQUENCIES:

The frequency of modes under 33 Hz are not significantly affected by the brace eccentricity.

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CONCLUSIONS:

Eccentricities in fixed braces

Eccentricities modeled were:

Model	Eccentricity (IN)
1	1.399
2	0
3	.511

Actual load transfer should be from channel shear center to angle C.G. This is only .289"
(e_o = .599", .888 - .599 = .289")

In the braces using the largest differences from case 1 to case 2 for ME and MY simultaneously

$$\frac{(22+49)\times 12}{.833} = 1023 \text{ psi} \qquad \frac{1023}{.6(36000)} = 4.7\%$$

Looking at the more realistic case 2 to case 3 moments

$$\frac{(10+18)\times 12}{.833} = 403 \text{ psi} \qquad \frac{403}{.6(36000)} = 1.9\%$$

In the posts case 1 to case 2

$$\frac{31\times 12}{.492} + \frac{3\times 12}{4.38} = 764 \text{ psi} \qquad \frac{764}{.6(36000)} = 3.5\%$$

Case 2 to 3

$$\frac{12\times 12}{.492} + \frac{1\times 12}{4.38} = 295 \text{ psi} \qquad \frac{295}{.6(36000)} = 1.4\%$$

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Given the small magnitude of the changes even when combining worst case moments from different load cases, and considering load magnitudes increasing somewhat, it does not appear significant to model the brace eccentricity. Note that even in the less conservative case 2 to case 3 change the eccentricity modeled was still almost double the actual load path of angle C.G to post shear center.

Fixity in braces

Evaluating the change in moments from case 1, fixed brace with large eccentricity, to case 4, pinned brace with the same eccentricity we find increases in both the brace and posts for the fixed case. Using worst case changes

POST $\frac{27\times12}{4.38} + \frac{6\times12}{.492} = 1562$ psi $\quad \frac{1562}{.6(36000)} = 7.2\%$

BRACE $\frac{(36+55)\times12}{.833} = 1310$ psi $\quad \frac{1310}{.6(36000)} = 6.1\%$

Evaluating case 2 to case 5, fixed and pinned with no eccentricities we have

POST $\frac{22\times12}{4.38} + \frac{11\times12}{.492} = 329$ psi $\quad 1.5\%$

BRACE $\frac{(25+16)\times12}{.833} = 447$ psi $\quad 2.1\%$

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It is seen that the end condition has more impact on the stress change for the eccentric case. The eccentricity modeled here was 1.399/.239 = 4.8 times greater than the proper, brace shear center to post C.G. offset. Accounting for this factor the case 1 to 4 evaluation becomes similar to the case 2 to 5 stresses. Thus the eccentricity effect is still negligible. In determining the impact of fixity on the braces some load increase must be considered. If the case 2 to 5 stresses are increased by a factor of 10 there could be some impact especially on the ports as they already are subjected to considerable normal warping stresses. It is therefore judged good practice to model the fixity conditions where they exist.

This appendix explicitly addresses eccentricities of in-plane members. However, conclusions are valid for out of plane members since the geometry of the connection of these members is similar

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TUGCO - COMANCHE PEAK UNIT 2
BRACING STUDY

IMPELL CORPORATION

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MAX. POINT SPACING:

$$S_m = \frac{1}{2}\left(\frac{\pi}{24}\right)^{1/2}\left(\frac{EI_a}{w}\right)^{1/4}$$

a) C6x8.2

$I_{x-x} = 13.1\ in^4$
$I_{y-y} = .693\ in^4$ ← critical
$w = 8.2\ \#/ft$
$= .683\ \#/in$

$$S_m = \frac{1}{2}\left(\frac{1.56}{33}\right)^{1/2}\left(\frac{(29.5E6)(.693)(32.2\times12)}{.683}\right)^{1/4}$$

$= 35.651$ in OR 2.971'
vs. 2.96' used in SUPERPIPE

b) C4x7.2

$I_{x-x} = 4.59\ in^4$
$I_{y-y} = .433\ in^4$ ← critical
$w = 7.25\ \#/ft = .604\ \#/in$

$$S_m = .109\left(\frac{(29.5E6)(.433)(32.2\times12)}{.604}\right)^{1/4}$$

$= 32.772$ in OR 2.731'
vs. 142' used in SUPERPIPE

c) L3x3x3/8

$I_{x-x} = 1.76\ in^4$
$I_{y-y} = 1.76\ in^4$
$w = 7.2\ \#/ft = .6\ \#/in$

$$S_m = .109\left(\frac{(29.5E6)(1.76)(32.2\times12)}{.6}\right)^{1/4} = 46.610\ in$$

OR 3.884'
vs. 3.8' by SUPERPIPE

Reference (2, 3)

REV	BY	DATE	CHECKED	DATE
0	VKY	11/14/95	HKS	11/18/95

TXU - COMANCHE PEAK UNIT 2
BRACING STUDY

IMPELL CORPORATION

JOB NO 0210-040-1352
CALC NO M-12

LUMPED WEIGHT:

TRAY 1 - 12x4 (Ref 1)

L1 = 5'-3" CTH-9761
L2 = 6'-3" CTH-9770

TOTAL LENGTH = 2.625' + 3.125'
= 5.75'

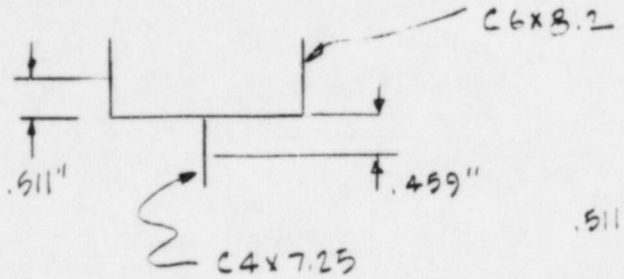
CABLE/TRAY WEIGHT = 35 #/FT. (Ref 2)

@ WEIGHT AT SUPPORT = 5.75 FT x 35 #/FT
= 201.25 #

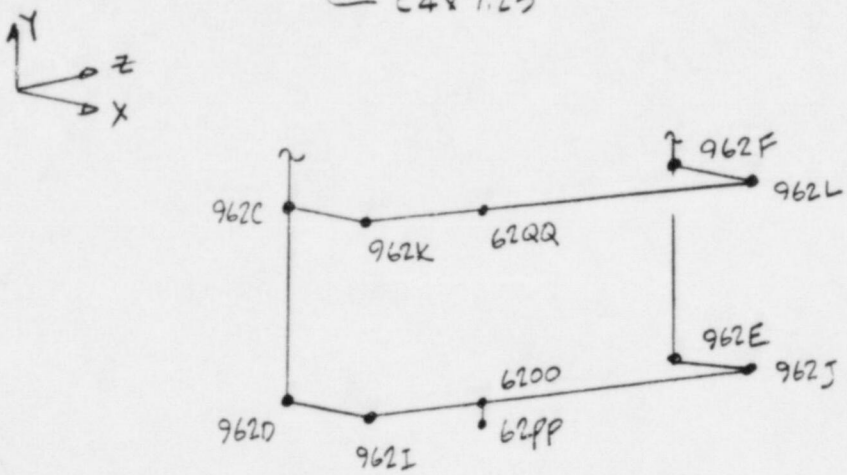
REFERENCE (1, 2)

					TYCO-COMANCHE PEAK UNIT 2 BRACING STUDY		
						JOB NO 0210-040-1352	PAGE B.2
0	VKY	11/14/85	HKS	11/18/85	IMPELL CORPORATION	CALC NO M-12	OF 34
REV	BY	DATE	CHECKED	DATE			

ECCENTRICITY AT TIERS



.511" + .459" = .970"
= .081'



NODE	A	B	C	I	J
962K	OFF	.081	–	–	962C
962L	OFF	.081	–	–	962F
962I	OFF	.081	–	–	962D
962J	OFF	.081	–	–	962E

REFERENCE (1, 3)

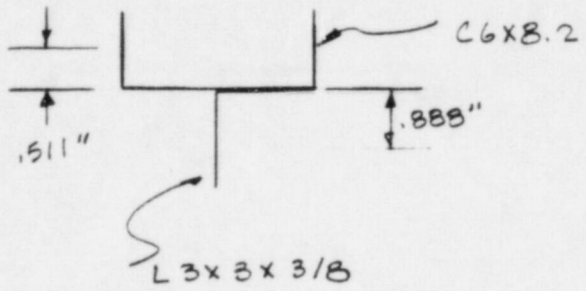
REV	BY	DATE	CHECKED	DATE
0	VKY	11/14/85	HKS	11/18/85

TUGCO · COMANCHE PEAK UNIT 2
BRACING STUDY

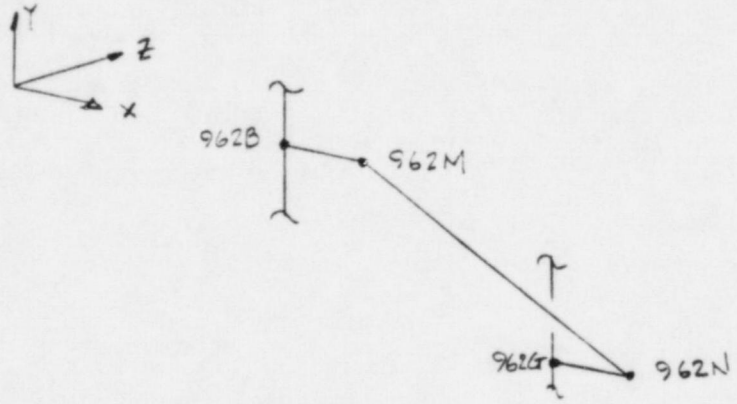
IMPELL CORPORATION

JOB NO 0210-040-1352
CALC NO M-12

ECCENTRICITY AT BRACE MEMBER (MODEL 1)



.511 + .888 = 1.399"
= .117'



NODE		A	B	C	I
962M	OFF	.117	—	—	962B
962N	OFF	.117	—	—	962G

REFERENCE (1, 3)

					TUGCO- COMANCHE PEAK UNIT 2
					BRACING STUDY
0	VR	11/14/85	HKS	11/18/85	IMPELL CORPORATION
REV	BY	DATE	CHECKED	DATE	

JOB NO 0210040+352
CALC NO M-12
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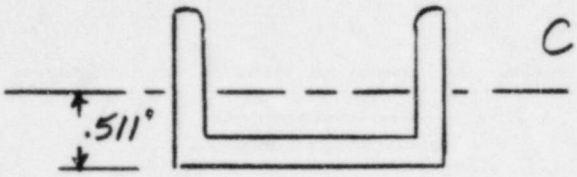
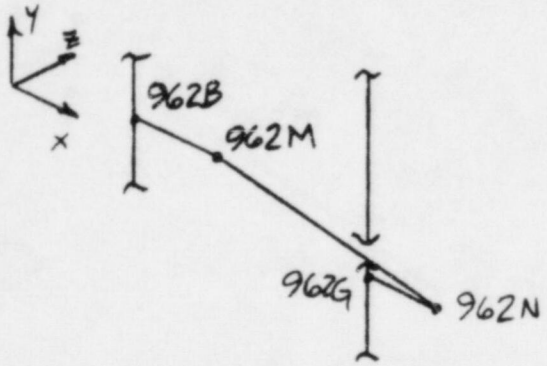
ECCENTRICITY AT BRACE Model 3

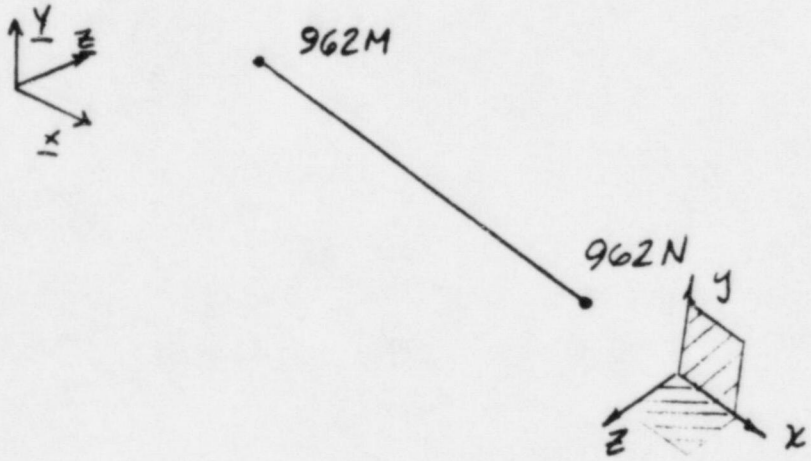


C 6×8.2

.511/12 = .043'



Pinned Connections for Models 4 & 5



MEMBER SPECIFICATION

3 BEAM L3×3×3/8 E29.5E6 962M 962N

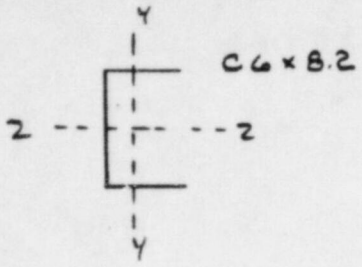
3 BEAM L3×3×3/8 E29.5E6 962M 962N FFFF*RR FFFF*RR

5 7-10 13-20 22-35 37-40 42-45 63-68 70-75

* Low torsional stiffness is input to avoid instability

					TUSCO COMMANCHE PEAK BRACING STUDY	
0	HKS	11/20/85	VKY	12-3-85	IMPELL CORPORATION	JOB NO 02100401352
REV	BY	DATE	CHECKED	DATE		CALC NO M-12

INFORMATION ONLY



$A_x = 2.40 \text{ IN}^2$

$A_y = 6 \times .2 = 1.2 \text{ IN}^2$

$A_z = 2(1.92 \times .343) = 1.32 \text{ IN}^2$

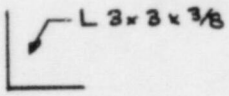
$I_{xx} = 0.075 \text{ IN}^4$

$I_{yy} = 0.692 \text{ IN}^4$

$I_{zz} = 13.1 \text{ IN}^4$

BEAM C6×8.2

SECP	2.40	1.20	1.32	0.075	0.692	13.1
MASS	8.2					



$A_x = 2.11 \text{ IN}^2$

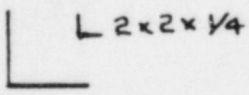
$A_y = A_z = 1.125 \text{ IN}^2$

$I_{xx} = \frac{1}{3} s t^2 = \frac{1}{3}(5.625)(.375)^3 = 0.099 \text{ IN}^4$

$I_{yy} = I_{zz} = 1.76 \text{ IN}^4$

BEAM L3×3×3/8

SECP	2.11	1.13	1.13	0.099	1.76	1.76
MASS	7.2					



$A_x = 0.938$

$A_y = A_z = 0.500$

$I_{xx} = \frac{1}{3} s t^3 = \frac{1}{3}(3.75)(.25)^3 = 0.0195$

$I_{yy} = I_{zz} = 0.348$

BEAM L2×2×1/4

SECP	0.938	0.500	0.500	0.0195	0.348	0.348
MASS	3.19					

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0	BLC	10-3-85	WJW	10/9/85

IMPELL CORPORATION

0210-040

JOB NO [redacted]

CALC NO M-12

PAGE [illegible] OF [illegible]

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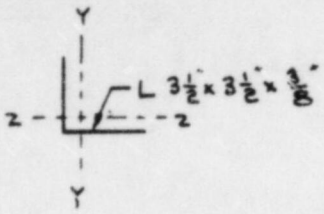
INFORMATION ONLY



$A_x = 2.48\ IN^2$

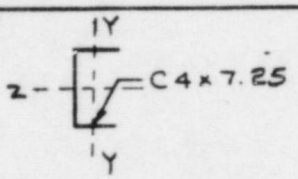
$A_y = A_z = 3.5 \times .375 = 1.31\ IN^2$

$I_{xx} = \beta s t^3 = (.31)(6.625)(.375)^3 = 0.108\ IN^4$

$I_{yy} = I_{zz} = 2.87\ IN^4$

BEAM L 3.5 x 3/8

SECP	2.48	1.31	1.31	0.108	2.87	2.87
MASS	8.5					



$A_x = 2.13\ IN^2$

$A_y = 4 \times .321 = 1.28\ IN^2$

$A_z = (2)(1.721 \times .296) = 1.02\ IN^2$

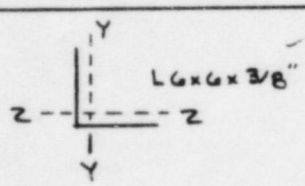
$I_{xx} = 0.082$

$I_{yy} = 0.432\ IN^4$

$I_{zz} = 4.59\ IN^4$

BEAM C4 x 7.25

SECP	2.13	1.28	1.02	0.082	0.432	4.59
MASS	7.25					



$A_x = 4.36\ IN^2$

$A_y = A_z = 6 \times .375 = 2.25\ IN^2$

$I_{xx} = \frac{1}{3}(11.625)(.375)^3 = 0.204\ IN^4$

$I_{yy} = I_{zz} = 15.4\ IN^4$

BEAM L6x6x3/8

SECP	4.36	2.25	2.25	0.204	15.4	15.4
MASS	14.9					

REV	BY	DATE	CHECKED	DATE
0	GLC	10-3-85	WJW	10/16/85

IMPELL CORPORATION

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Excerpts From Ref [43]

— For Information Only ——

ATTACHMENTS

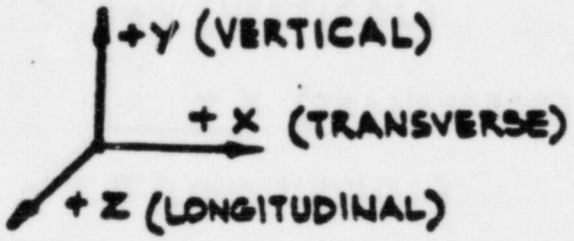
INFORMATION ONLY

INFORMATION ONLY

I – MODEL (excert from Ref [4])

a) A three dimensional model shall be used and all members shall be represented by lines with relative eccentricities determined in accordance with guidelines contained herein.

1. The user shall prepare the model using the global axis shown below



b) For both the static analysis model and the frequency analysis model, a rigid member between post and tier shall represent eccentricities and joint loading shall be used for both.

c) All nodal points shall be as follows:
For bracing:

- pin connection shall be assumed on connection with plate. (SEE ATTACHMENT "J")

- pin connection shall be assumed for braces welded to back of posts (SEE ATTACHMENT "H")

For post to tiers:

- all shall be fixed connections for hanger to building connections. See Section III.

INFORMATION ONLY

d) Local eccentricity shall be considered for gusset plate verification. The total eccentricity considered should be equal to half the thickness of the gusset plate plus half the thickness of the angle leg welded to the gusset plate.

e) For other eccentricities, information and details; see Attachment "E".

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INFORMATION ONLY

(excerpt from Ref [43])

IV - ECCENTRICITIES

For computer or hand analysis, various eccentricities must be considered to realistically account for the application of loads and interconnections between structural members:

1. For load eccentricities application on tiers, see attachment "B1" and "B2"

2. For connecting eccentricities between tiers and posts, use a rigid link with a length equal to the distance $\bar{x}$ (post) - x_o (tier) + $\frac{tw}{2}$ (tier). (SEE ATT. "E", SH. 1)

3. For bracing eccentricities for working point condition and gusset plates, see Attachment I and J respectively.

4. For eccentricities consideration due to brace welding to the back of the post, see Attachment "H"

V - NODAL POINTS

A) Assume one nodal point if the dimension between the top of the horizontal tier and the bottom of the diagonal brace is within $d/2$ inches for $\alpha \geq 50°$ and $d/3$ inches for $\alpha < 50°$. The "d" is the width of the post to which bracing is welded. Refer also to ATTACHMENT "I".

B) Assume one nodal point if a gusset plate provides the anchorage for the diagonal brace. Refer also to Attachment "J".

VI - WARPING STRESSES

INFORMATION ONLY

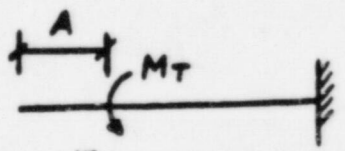
After the static analysis results are obtained torsional moments are found in the various members of the hanger. These torsional moments generate warping stresses (both normal and shear) which have to be added to the normal and shear stresses obtained from the frame analysis done by computer. For this purpose the attachment M and procedure below should be used.

a) Cantilever Condition

The following outlines the procedure on how to obtain the torsional stress (both normal and shear) at fixed end of a cantilever member subject to torsional moment M (K-in).

1. Obtain the torsional moment M_T (K-in)

2. Determine the distance "A" from the free end to the point of application of torsional moment M_T.



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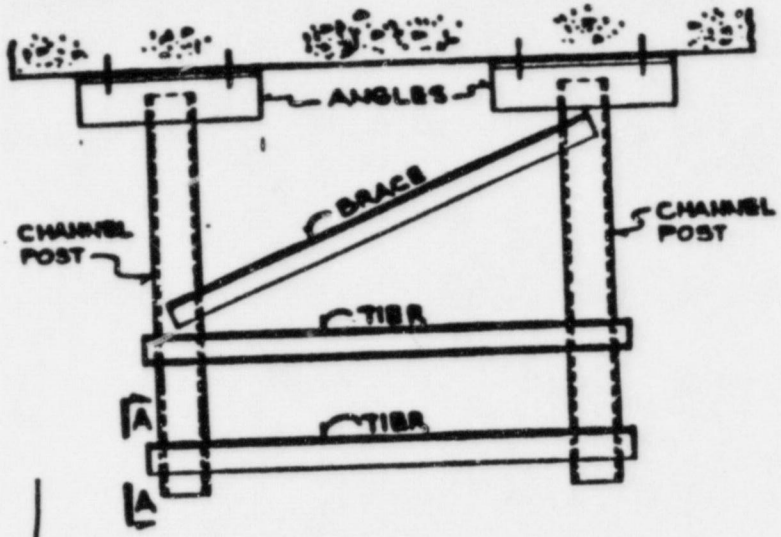
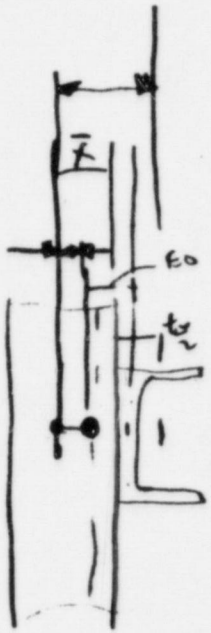
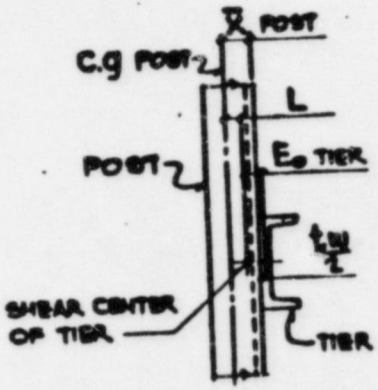
ATTACHMENT E - ADDITIONAL NOTES ON MODELS (SH. 1)

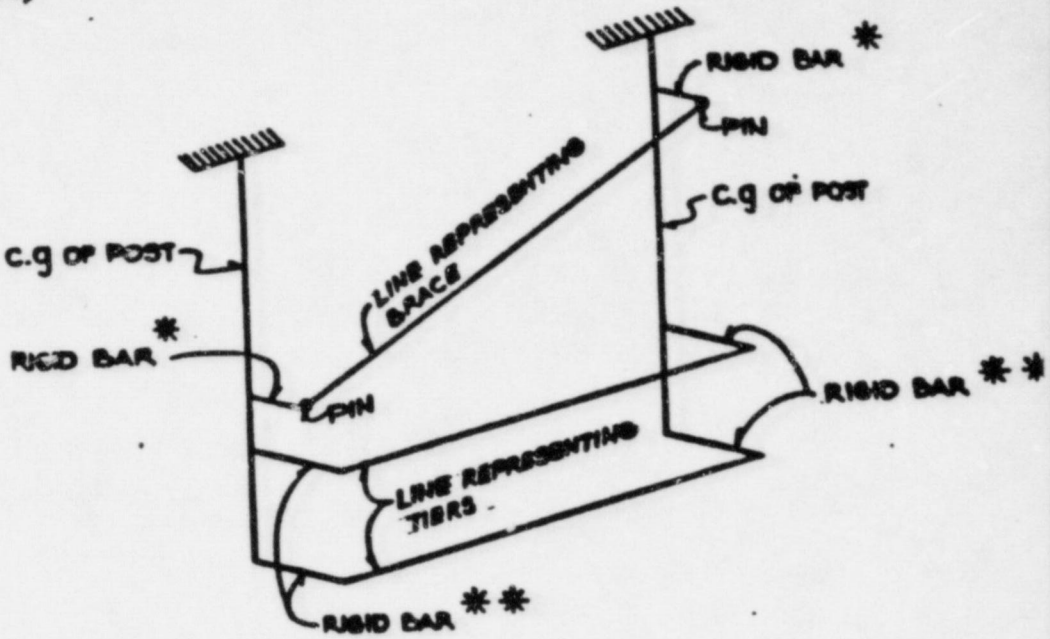


ACTUAL SUPPORT CONFIGURATION





SECTION A-A



MODEL

* SEE ATTACHMENT H FOR LENGTH OF RIGID BAR TO BE INPUT. AFTER RUNNING MODEL, GO BACK AND CHECK BRACE WITH AN ECCENTRICITY EQUAL TO $C.g_{POST} + C.g_{ANGLE}$

** L = LENGTH OF RIGID BAR REPRESENTING THE POST-BEAM EFFECTIVE ECCENTRICITY

$L = \bar{X}_{(POST)} - E_{0(TIER)} + \frac{t_w}{2}_{(TIER)}$

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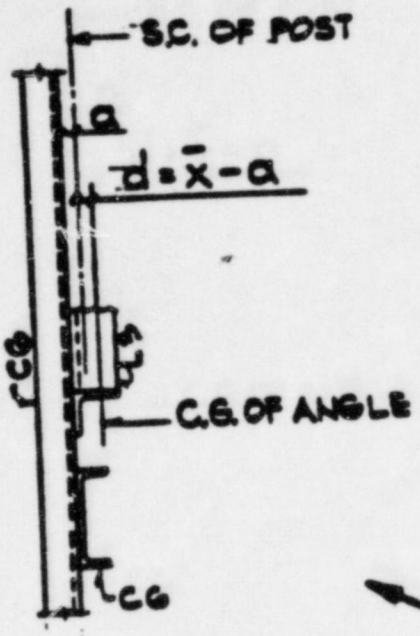
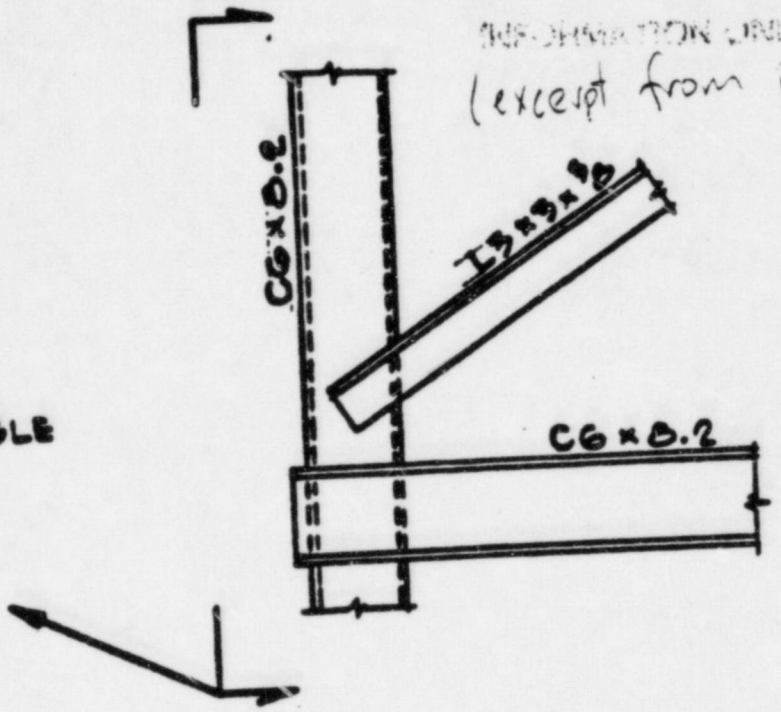
Proj

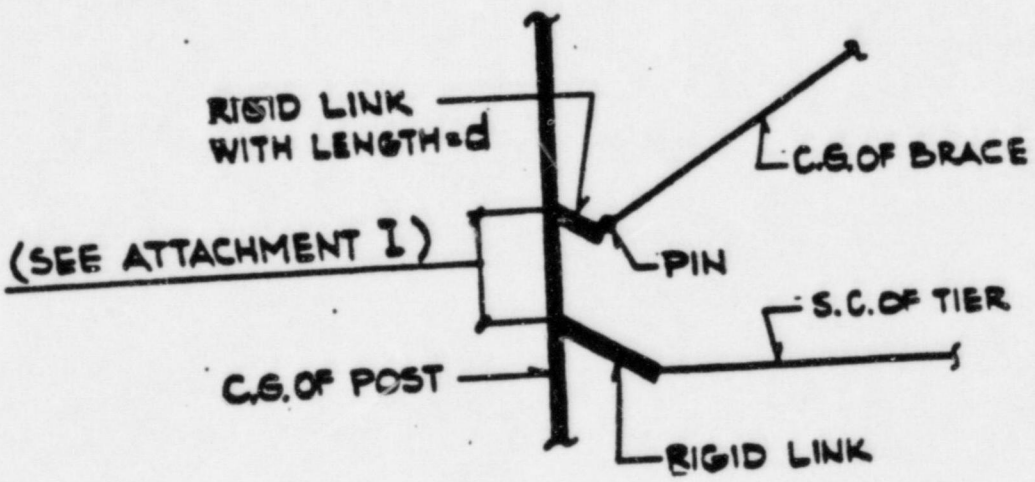
(except from Ref [12])

ATTACHMENT H - ECCENTRICITIES FOR BRACES WELDED TO THE BACK OF VERTICAL POST



(excerpt from Ref [4])





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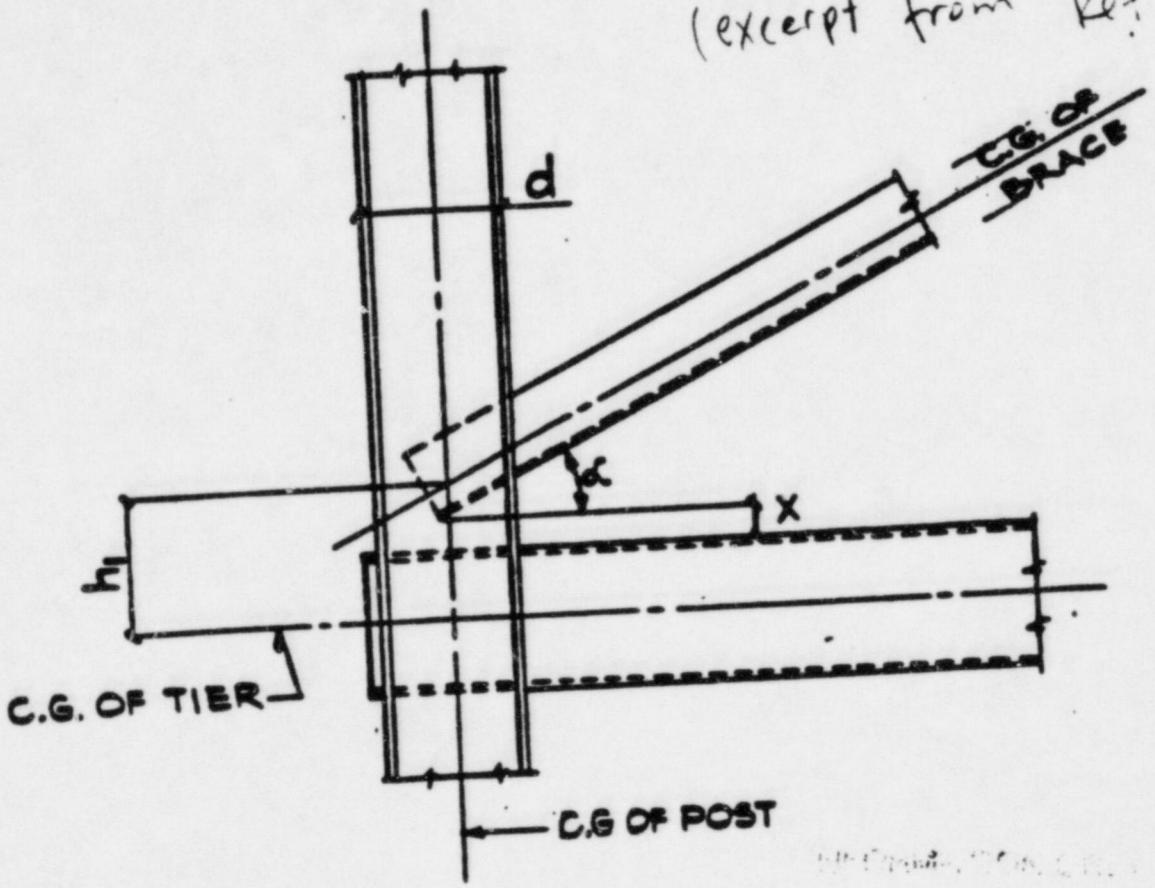
Calc # VII-12 page # B31/34

(6/14/85)

ATTACHMENT I - WORKING POINT BRACE ECCENTRICITY

INFORMATION ONLY

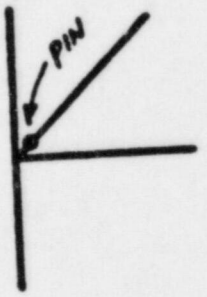
(excerpt from Ref [43])



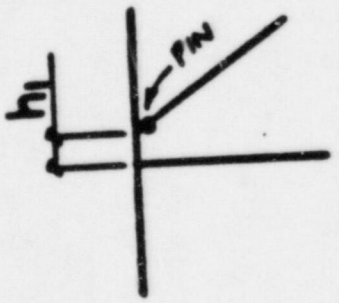
CONDITIONS:

$X < \frac{d}{2}$ FOR $\alpha \geq 50°$

$X < \frac{d}{3}$ FOR $\alpha < 50°$



MODEL WHEN CONDITIONS ARE MET
(SEE ATTACHMENT H)



MODEL WHEN CONDITIONS ARE NOT MET
(SEE ATTACHMENT H)

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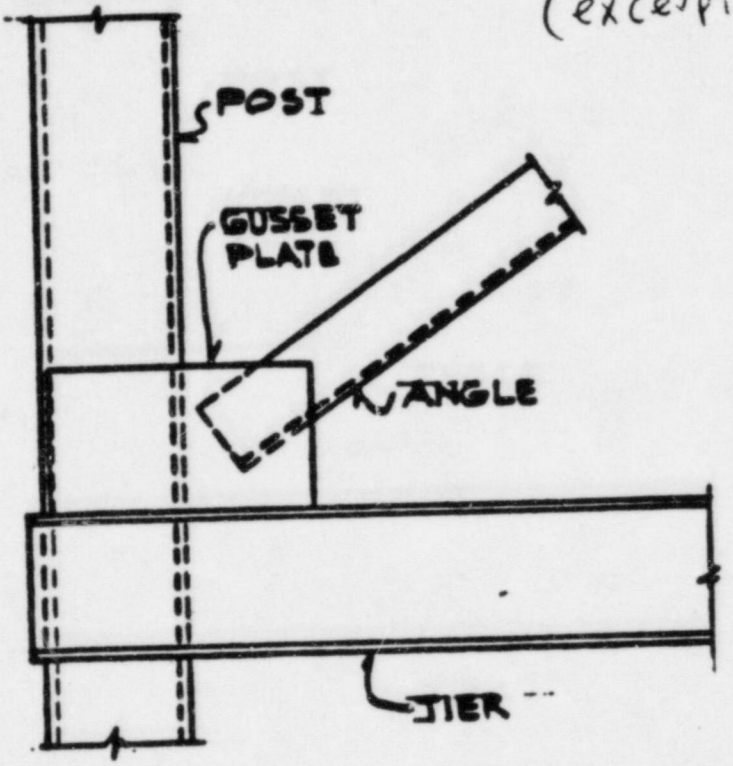
Calc # M1-12 Page # B32

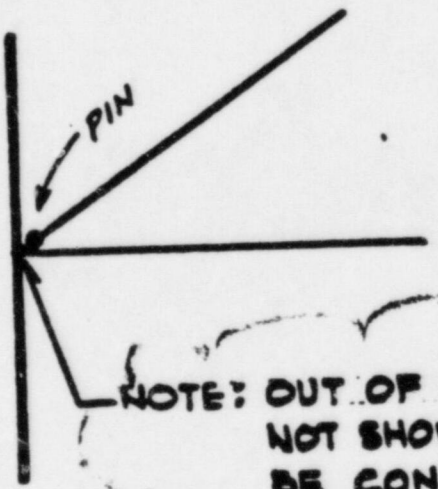
REV	BY	DATE	CHECKED	DATE			
					IMPELL CORPORATION	JOB NO CALC NO	PAGE OF

(excerpt from Ref [4])

ACTUAL SUPPORT DETAIL



COMPUTER MODEL INPUT



NOTE: OUT OF PAPER ECCENTRICITY NOT SHOWN HERE BUT SHOULD BE CONSIDERED IF IT EXISTS.

(6/14/05)

SHEET 43

Job # 0210.040

Calc # A1-12 Page 1333/34

REFERENCES

1. TUGCO DRAWING CTH-2-9762 REV. 01 6/24/85
2. IMPELL PROJECT INSTRUCTION PI-02 "DYNAMIC ANALYSIS OF CABLE TRAY SYSTEMS" REV. A JOB 0210-040
3. MANUAL OF STEEL CONSTRUCTION AISC, 7th Edition
4. EBASCO SERVICES INC. "GENERAL INSTRUCTIONS FOR CABLE TRAY HANGER ANALYSIS FOR COMMANCHE PEAK STEAM ELECTRICAL STATION NO.2" REV. 1 8/23/85

POINT FORCES, GRAVITY & SEISMIC

					TUGCO COMMANCHE PEAK UNIT 2 BRACING STUDY	
0	HKS	12-3-85	VKY	12-3-85	IMPELL CORPORATION	JOB NO 0210-040-1352 CALC NO M-12 PAGE 34 OF 34
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APPENDIX C

ALLOWABLE STRESSES IN ANGLE MEMBERS CONSIDERING LATERAL TORSIONAL BUCKLING

This Appendix identifies an acceptable procedure for evaluating Lateral-Torsional Buckling of Angle Sections. This criteria was originally derived for the V.C. Summer Project, It is based on the following References.

[1] Johnston, Bruce G., Editor. "Guide to Stability Design Criteria for Metal Structures," Third Edition, Wiley, 1975

[2] Gaylord and Gaylord, "Design of Steel Structures," Second Edition, McGraw Hill

					Tugco		
0	WL	1/14/86			IMPELL CORPORATION	JOB NO 0210-040	PAGE C1
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DETERMINATION of EQUIVALENT KL/r OF AN ANGLE SECTION FOR USE WITH AISC TABLE 1-36 AND DETERMINATION of ALLOWABLE BENDING STRESS

$$\sigma_c = \frac{C_4 \sqrt{I_y J}\sqrt{EG}}{S_x L}$$ EQ 6.11 REF. 1

$$C_4 = \frac{C_1}{K}\pi\left[\sqrt{1+\frac{\pi E C_w}{(KL)^2 GJ}(C_2^2+1)} \pm \frac{C_2\pi}{KL}\sqrt{\frac{E C_w}{GJ}}\right]$$ Eq. 6.12

C_w = TORSION WARPING CONSTANT TAKEN AS ZERO FOR ANGLE SECTION

$$C_4 = \frac{C_1\pi}{K}$$

$$\sigma_c = \frac{C_1\pi}{K}\frac{\sqrt{I_y J}\sqrt{EG}}{S_x L}$$ I

DEFINE

$$P_c = \frac{\pi^2 E I}{(KL)^2} = \frac{\pi^2 E A r^2}{(KL)^2}$$ EULER FORMULA

$$\sigma_c = \frac{P_c}{A} = \frac{\pi^2 E r^2}{(KL)^2} = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}$$ II

REV	BY	DATE	CHECKED	DATE		JOB NO 0218-040	PAGE 12
0	JAY	2/24/84	NW	2/28/84	IMPELL CORPORATION	CALC NO —	OF 1

SUBSTITUTE II → I

$$\frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{C_1 \pi}{K} \frac{\sqrt{I_x J}\sqrt{EG}}{S_x L} \qquad \text{III}$$

DEFINE C_0 AND SUBSTITUTE INTO III

$$C_0 = \frac{\pi \sqrt{I_y J}\sqrt{EG}}{S_y}$$

$$\frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{C_1}{KL} C_0 \qquad \text{IV}$$

SOLVE FOR $\left(\frac{KL}{r}\right)$ EQUIVALENT

$$\left(\frac{KL}{r}\right)_{EQ} = \sqrt{\frac{\pi^2 E KL}{C_1 C_0}} = \pi \sqrt{\frac{E KL}{C_1 C_0}}$$

C_1 IS AS DEFINED IN FIG. 6.5 of THE ABOVE REFERENCE

EQ 6.12 IS APPLICABLE TO DOUBLY SYMMETRIC I-SHAPES BUT ITS APPLICABILITY TO OTHER CROSS SECTIONS IS IMPLIED IN SECTION 6.11 of THE ABOVE REFERENCE.

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Excerpts from V.C. Summer Project

Instructions

— For Information Only —

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PROJECT INSTRUCTIONS

CLIENT / PROJECT:	SOUTH CAROLINA ELECTRIC & GAS COMPANY V.C. SUMMER NUCLEAR STATION UNIT 1	No.: Revision: 0
TITLE:	ENGINEERING PROCEDURES FOR CONSIDERING LATERAL-TORSIONAL BUCKLING OF ANGLE SECTIONS	Page 1 of 5

ATTACHMENT O

INFORMATION ONLY

1.0 INTRODUCTION

The AISC formulae do not take lateral-torsional buckling into consideration for angles. In certain cases, therefore, the AISC formulae are not conservative. Specifically, for relatively long spans, the allowable bending stress, F_b, must be reduced.

This procedure outlines the methodology to be used to account for lateral-torsional buckling of laterally unsupported angle sections on the V.C. Summer Project.

2.0 METHODOLOGY

INFORMATION ONLY

The methodology falls into two steps: First, the elastic buckling stress, σ_c, must be calculated and then, secondly, the corresponding allowable bending stress, F_b, must be determined.

Step 1

σ_c is calculated from

$$\sigma_c = \frac{C_1}{KL} \times C_o$$

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where

$$C_o = \pi \frac{\sqrt{I_y J}}{S_x} \frac{E}{\sqrt{2(1+\nu)}}$$

K = Effective length factor
L = Unbraced length
I_y = Second moment of area about minor principal axis
J = Torsional constant
S_x = Section modulus about major principal axis
E = Modulus of elasticity
ν = Poisson's ratio (0.3 for steel)

C_1 is a constant which depends on the load distribution and end restraint conditions. Values of C_1 are shown on Table I.

Values of C_o for equal angle sections commonly used on this project are given in Table 2.

The effective length factor, K, is given in AISC Section 1.8, Table C-1.8.1.

Step 2

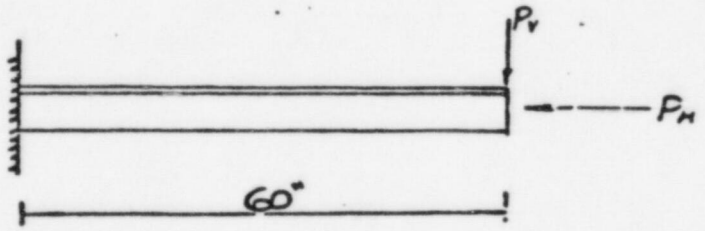
The value of F_b is determined by calculating an equivalent slenderness ratio $\left(\frac{KL}{r}\right)$ from the formula:

$$\left(\frac{KL}{r}\right)_{eq} = \pi\sqrt{\frac{E}{\sigma_c}}$$

F_b is then obtained from AISC Table I-36.

3.0 EXAMPLE PROBLEM

Consider a 60" cantilevered L2x2x3/8 span with the loading condition shown; check allowable compressive bending stress.



Step 1

$$\sigma_c = \frac{C_1}{KL} C_o \qquad L = 60"$$

From Table I, Case 7: $C_1 = 1.3$

From Table II, for L2x2x3/8: $C_o = 11.7 \times 10^3$

From AISC Table C-1.8.1: $K = 2.1$

$$\sigma_c = \frac{1.3}{2.1 \times 60} \times 11.7 \times 10^3$$

$$= 120.7 \text{ ksi}$$

Step 2

$$\left(\frac{Kl}{r}\right)_{eq} = \pi\sqrt{\frac{E}{\sigma_c}}$$

$$= 3.1416\sqrt{\frac{29 \times 10^3}{120.75}}$$

$$= 48.7$$

From AISC Table I-36,

$$F_b = 18.5 \text{ ksi}$$

The value of f_b, f_a, and F_a are calculated in the normal manner.

See Table III for allowable compressive bending stresses for equal angle sections.

Reference: Johnston, Bruce G., Editor. "Guide to Stability Design Criteria for Metal Structures," Third Edition, Wiley, 1975.

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TABLE I

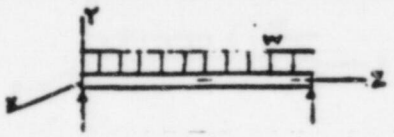
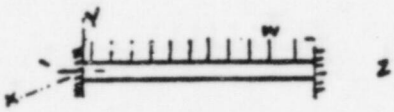
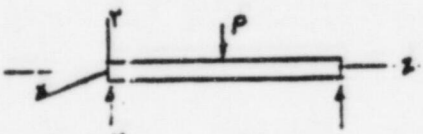
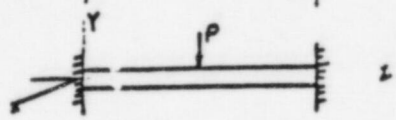
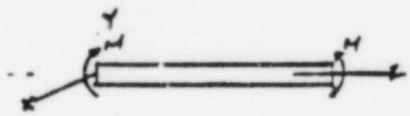
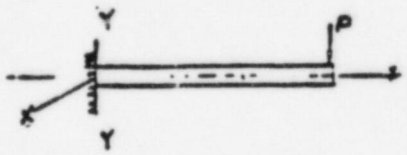
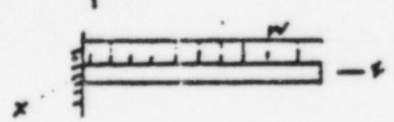
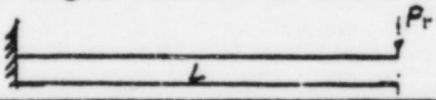
Case	Loading and End Restraint about X-Axis	End Restraint about Y-Axis	C_1
1		None	1.13
		Full	0.97
2		None	1.30
		Full	0.86
3		None	1.35
		Full	1.07
4		None	1.70
		Full	1.04
5		None	1.04
6		None	1.00
7		Full	1.30
8		Full	2.05

TABLE II

Angle Section	C_o
L2 x 2 x 3/8	11.7×10^3
L3 x 3 x 3/8	11.5×10^3
L3½ x 3½ x 3/8	11.4×10^3
L4 x 4 x 3/8	11.4×10^3
L6 x 6 x 3/8	11.3×10^3

TABLE III

Allowable compressive bending stresses (ksi) for cantilever angle section as shown.



Angle Section \ Length	12"	24"	36"	48"	60"
L2 x 2 x 3/8	20.5	19.9	19.4	18.9	18.5
L3 x 3 x 3/8	20.5	19.9	19.4	18.9	18.4
L3½ x 3½ x 3/8	20.5	19.8	19.3	18.8	18.4
L4 x 4 x 3/8	20.5	19.8	19.3	18.8	18.4
L6 x 6 x 3/8	20.5	19.8	19.3	18.8	18.4

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APPENDIX D

INTERSUPPORT TORSIONAL STRESS EVALUATION METHODOLOGY

The following pages are excerpts from the Intersupport Technical Manual which describes a simplified and conservative methodology for evaluating torsional warping stresses. This method is acceptable for use on this project.

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2.1.1.4.2. Warping Stresses

Inspection of Vlasov's equation shows that the warping depends on the higher order derivatives of the twist angle (the Saint Venant torsion is proportionnal to the first derivative of the twist angle).

The warping stresses may be of two kinds.

i. Direct warping stresses related to the non uniform torsion of the beam,

$$f_{aw} = E W_s \frac{d^2\theta}{dx^2}$$

ii. Shear warping stresses induced by the varying direct stress,

$$f_{vw} = - \frac{E S_s}{t} \frac{d^3\theta}{dx^3}$$

Where
W_s and S_s are warping functions defined below.

These stresses may not be directly derived from finite element results as they are proportionnal to the second and third order derivatives of the twist angle which are not available. However, an upper boundary may be derived for a number of special cases to which the actual cases are assimilated.

It may be easily demonstrated that the warping stresses depend on the end conditions of the beams, i.e., on the boundary conditions for the warping.

It is not the scope of this report to develop in detail the warping theory. However, the following three boundary conditions may be expected.

Sketch	Torsion Twist angle	Warping	Combined
	Free	Free $\theta'' = 0$	Free
	Fixed $\theta = 0$	Free $\theta'' = 0$	Pinned
	Fixed $\theta = 0$	Fixed $\theta' = 0$	Fixed

Table 2.1.1.4.2.1. - Combined boundary conditions

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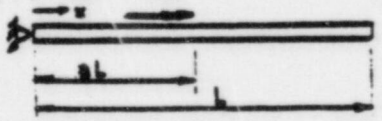
Out of these three combined boundary conditions, six beams may be defined as typical. Each of them shall be solved in order to defined the maximum stresses.

Beam A : free - free

This kind of beam is unstable and no general solution may be provided. This beam could not be solved by using the finite element method.

Beam B : pinned - free



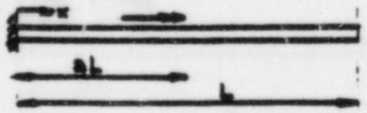
The twist angle is given by

$$\theta(x) = \frac{M_w(x).x}{G\,R} \qquad \text{for } x \leq aL$$

$$\theta(x) = \frac{M_w(x).a.L}{G\,R} \qquad \text{for } x \geq aL$$

From which it is seen that no warping stresses are induced for this beam. The externally applied torque is entirely balanced by Saint Venant shear stresses.

Beam C : fixed - free



$$C = \sqrt{\frac{E\,C_w}{GR}}$$

The solution for this beam is given by,

$$\theta(x) = \frac{M_w(x)}{G\,R}\left(K_1\left(\cosh\frac{x}{\ell} - 1\right) - \sinh\frac{x}{\ell} + \frac{x}{\ell}\right) \qquad \text{for } x \leq aL$$

$$\theta(x) = \frac{M_w(x)}{G\,R}\left(K_2 - K_3 + K_4 + \frac{aL}{\ell}\right) \qquad \text{for } x \geq aL$$

Where the constants are defined as follows,

$$K_1 = \sinh\frac{aL}{\ell} - \tanh\frac{L}{\ell}\cosh\frac{aL}{\ell} + \tanh\frac{L}{\ell}$$

$$K_2 = \tanh\frac{L}{\ell}\cosh\frac{aL}{\ell} - \tanh\frac{L}{\ell} - \sinh\frac{aL}{\ell}$$

$$K_3 = \left(\cosh\frac{aL}{\ell} - 1\right)\left(\tanh\frac{L}{\ell}\cosh\frac{aL}{\ell}\right)$$

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$$K_4 = \left(\cosh\frac{aL}{\zeta} - 1 \right) \sinh\frac{aL}{\zeta}$$

Differentiation of above equations allows to settle the following formula for the warping stresses,

$$f_{aw} = \frac{EW_s}{GR} \frac{M}{\zeta} \left(\sinh\frac{aL}{\zeta} - \tanh\frac{L}{\zeta} \cosh\frac{aL}{\zeta} + \tanh\frac{L}{\zeta} \right)$$

$$f_{vw} = \frac{ES_s}{GR} \frac{M\zeta}{t} (K_a K_b - K_c)$$

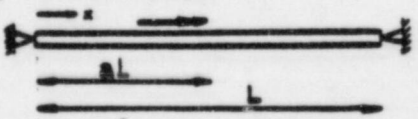
Where

$$K_a = \sinh\frac{aL}{\zeta} - \tanh\frac{L}{\zeta} \cosh\frac{aL}{\zeta} + \tanh\frac{L}{\zeta}$$

$$K_b = \frac{1}{\zeta^2} \sinh\frac{0.6aL}{\zeta}$$

$$K_c = \frac{1}{\zeta^3} \cosh\frac{0.6aL}{\zeta}$$

Beam D : pinned - pinned



$$\zeta = \sqrt{\frac{E\,C_w}{GR}}$$

$$\theta(x) = \frac{M_w(x)L}{GR} \left\{ \frac{(1-a)x}{L} + \left[\frac{\sinh\frac{aL}{\zeta}}{\tanh\frac{L}{\zeta}} - \cosh\frac{aL}{\zeta} \right] \frac{\zeta}{L} \sinh\frac{x}{\zeta} \right\}$$

for $x \leq aL$

$$\theta(x) = \frac{M_w(x)L}{GR} \left\{ \frac{(L-x)a}{L} + \left[\frac{\sinh\frac{aL}{\zeta}}{\tanh\frac{L}{\zeta}} \sinh\frac{x}{L} - \sinh\frac{aL}{\zeta} \cosh\frac{x}{L} \right] \frac{\zeta}{L} \right\}$$

for $x \geq aL$

The second and third order derivatives of these functions are maximum at the loaded point. The warping stresses are given by,

$$f_{aw} = \frac{EW_s}{GR} \frac{M}{\zeta} \sinh\frac{aL}{\zeta} \left(\frac{\sinh\frac{aL}{\zeta}}{\tanh\frac{L}{\zeta}} - \cosh\frac{aL}{\zeta} \right)$$

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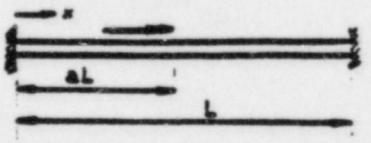
$$f_{vw} = \frac{ES_s}{GR} \frac{M}{t\zeta^2} \cosh\frac{aL}{\zeta} \left(\frac{\sinh\frac{aL}{\zeta}}{\tanh\frac{L}{\zeta}} - \cosh\frac{aL}{\zeta} \right)$$

Beam E : fixed - pinned

The case is assimilated to the next case.

Beam F : fixed - fixed



$$C = \sqrt{\frac{E\,C_w}{GR}}$$

The solution for this beam is given as follows :

$$\theta(x) = \frac{M_w(x)\,\zeta}{(1+H)GR} \left[(H\,K_1 + K_2)\left(\cosh\frac{x}{\zeta} - 1\right) - \sinh\frac{x}{\zeta} + \frac{x}{\zeta} \right]$$

for $x \leq aL$

$$\theta(x) = \frac{M_w(x)\,\zeta H}{(1+H)\,GR} \left[\frac{K_3}{H} + K_4 + K_5 \cosh\frac{x}{\zeta} + K_6 \sinh\frac{x}{\zeta} - \frac{x}{\zeta} \right]$$

for $x \geq aL$

where the constants are defined as follows :

$$H = \frac{K_6}{K_7}$$

$$K_1 = \frac{1}{\sinh\frac{L}{\zeta}} + \sinh\frac{aL}{\zeta} - \frac{\cosh\frac{aL}{\zeta}}{\tanh\frac{L}{\zeta}}$$

$$K_2 = \sinh\frac{aL}{\zeta} + \frac{1}{\tanh\frac{L}{\zeta}} - \frac{\cosh\frac{aL}{\zeta}}{\tanh\frac{L}{\zeta}}$$

$$K_3 = \frac{-1}{\sinh\frac{L}{\zeta}} \left(1 - \cosh\frac{aL}{\zeta}\right)$$

$$K_4 = \frac{1}{\sinh\frac{L}{\zeta}} \left(\cosh\frac{aL}{\zeta} - \cosh\frac{L}{\zeta} + \frac{L}{\zeta}\sinh\frac{L}{\zeta}\right)$$

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$$K_5 = \frac{1}{H} \frac{1}{\tanh\frac{L}{\zeta}} (1 - \cosh\frac{aL}{\zeta} + \frac{1}{\sinh\frac{L}{\zeta}} (1 - \cosh\frac{aL}{\zeta} \cosh\frac{L}{\zeta})$$

$$K_6 = \frac{1}{\tanh\frac{L}{\zeta}} (1 - \cosh\frac{aL}{\zeta} + \frac{1}{\sinh\frac{L}{\zeta}} (\cosh\frac{aL}{\zeta} - 1) + \sinh\frac{aL}{\zeta} - \frac{aL}{\zeta}$$

$$K_7 = \frac{1}{\sinh\frac{L}{\zeta}} (\cosh\frac{L}{\zeta} + \cosh\frac{aL}{\zeta} \cosh\frac{L}{\zeta} - \cosh\frac{a1}{\zeta} - 1) + \frac{L}{\zeta} (a-1) - \sinh\frac{aL}{\zeta}$$

$$K_8 = \frac{1}{H} (\cosh\frac{aL}{\zeta} - 1) + \cosh\frac{aL}{\zeta}$$

The maximum warping stresses occur at the fixed end. They are given by the following formula,

$$f_{aw} = \frac{EW_S}{GR} \frac{M}{\zeta} (HK_a + K_b)$$

$$f_{vw} = \frac{ES_s}{GR} \frac{M}{t\zeta^2}$$

where,

$$K_a = \frac{1}{\sinh\frac{L}{\zeta}} + \sinh\frac{aL}{\zeta} - \frac{\cosh\frac{aL}{\zeta}}{\tanh\frac{L}{\zeta}}$$

$$K_b = \sinh\frac{aL}{\zeta} + \cosh\frac{L}{\zeta} - \frac{\cosh\frac{aL}{\zeta}}{\tanh\frac{L}{\zeta}}$$

In above developed warping stress formula, a number of functions have been introduced. They are section shpae depedent as shown in the following definition,

$$W_o = \int_0^s \rho \, ds$$

$$A = \int_0^b t \, ds$$

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$$W_s = \int_0^a \frac{W_o}{A} t \, ds - W_o$$

$$S_s = \int_0^s W_s t \, ds$$

$$C_w = \int_0^b W_s^2 t \, ds$$

where the notations are defined on figure below.

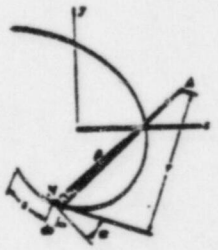


Figure 2.1.1.4.2.1. - Warping functions.

It may easyly be demonstrated that warping is not to be considered for angles and tees as their shear centers are located at the intersection of the legs. For these sections all the warping functions are identifically equal to zero.

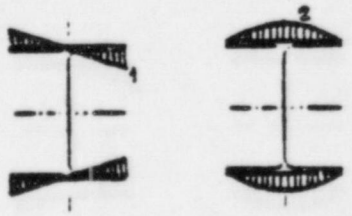
Among the standard sections considered in the program, only I-beams and channels are subject to warping stresses.

I-beams, H-beams, W-beams

$$C_w = \frac{d^2 I_1}{4}$$

$$W_{s1} = \frac{db_f}{4}$$

$$S_{s2} = \frac{db_f^2 t_f}{16}$$



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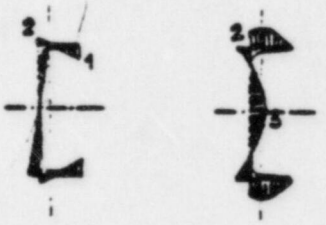
U-beams, Channels

$$C_w = \frac{(b_f - 3e)\, d^2 b_f^2\, t}{6} + e^2 I_2$$

$$W_{s1} = \frac{(b_f - e)d}{2}$$

$$W_{s2} = \frac{ed}{2}$$

$$S_{s3} = S_{s2} - \frac{ed^2 t_w}{8}$$



Substitution of these warping functions into the stress formula developed above allows to compute the maximum warping stresses in a beam subjected to non uniform torsion.

2.1.1.4.3. Warping Boundary Conditions

In the previous section theor tical boundary conditions for warping and combined torsion-warping end conditions have been defined. Figures below show some practical solutions corresponding to these end conditions.

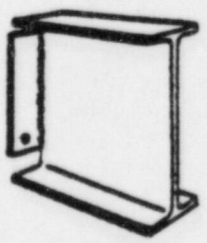


Figure 2.1.1.4.3.1. - pinned conditions

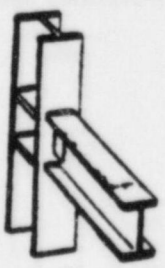


Figure 2.1.1.4.3.2. - fixed conditions

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2.1.1.5. Combination Procedure

The combination procedure for the "exact" stress formula is described in the following sections.

2.1.1.5.1. Notations

f_a	Direct stress due to axial load.
f_{min}	Maximum direct stress due to minor bending moment.
f_{maj}	Maximum direct stress due to major bending moment.
f_{vji}	Shear stress at location i for external shear j.
f_{svi}	Shear stress at location i under Saint Venant torsion component by assuming component equal to external torque.
$f_{aw\ min}$	Maximum direct stress on flanges under warping torsion component by assuming component equal to external torque.
$f_{aw\ maj}$	Maximum direct stress on web under warping torsion component (applicable only to channels).
f_{vwi}	Shear stress at location i under warping torsion component.

2.1.1.5.2. Procedure

The following combination are considered :

Axial Stress

f_a

Minor Bending Stress

$f_{min} + f_{aw\ min}$

Major Bending Stress

$f_{maj} + f_{aw\ maj}$

Shear Stress

envelope (f_{vi}, i = 1,n)
where

$f_{vi} = f_{v1i} + f_{v2i} + f_{vti}$

f_{vti} = enveloppe (f_{svi}, f_{vwi})

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2.1.2. DESIGN STRESS FORMULA

In this section simplified design stress formula are derived. There are more conservative than those derived in Section 2.1.1., but may be usefull in the preliminary stages of the design of a pipe support.

In this simplified procedure both the stress calculations, and the combination of stresses of same kind are different (and more conservative) than in the "exact" procedure.

The default action of the program is to use the "exact" procedure.

2.1.2.1. Axial Load and Bending Moment

Direct stresses due to axial load and to both bending moments are computed in the same way as in the "exact" procadure (see § 2.1.1.1. and § 2.1.1.2.).

2.1.2.2. Shear Loads

Externally applied shear loads are balanced by pure shear stress fields. Derivation of maximum shear stresses is done for each section by inspecting the shape of the section.

I-Shape, H-Shape, W-Shape
U-Shape, Channel

$$f_{v1} = \frac{F_1}{A_w}$$

$$f_{v2} = \frac{3F_2}{4A_f}$$

L-Shape, Angle

$$f_{v1} = \frac{3F_1}{2dt}$$

$$f_{v2} = \frac{3F_2}{2bt}$$

T-Shape, Tee

$$f_{v1} = \frac{3F_1}{2Aw}$$

$$f_{v2} = \frac{3F2}{2Af}$$

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Boxes, Pipes

The same formula as for the "exact" procedure are used.

2.1.2.3. Torsion Moment

As in the "exact" procedure, two components of the torsion moment are considered : the Saint Venant component and the warping component.

The Saint Venant shear stress is computed as in the "exact" procedure but considering only the maximum shear stress regardless of the section's element to which it is applied.

The warping component is considered for a cantilever beam only and is assumed to be applicable to all channels and I-beams regardless of their boundary conditions.

Direct warping stress will thus be

$$f_{aw} = \frac{M}{Z_1}\sqrt{2.6\,\frac{I1}{R}}$$

and maximum shear stress

$$f_{vw} = \frac{M}{dA_f}$$

2.1.2.4. Combination Procedure

The combination procedure for the design stress is conservative as it adds together stresses which are not applied at the same location of the beam section.

Explicitely the following stresses are combined

Axial Stress	Stress due to axial load
Shear Stress	Maximum stresses due to both external shear
	Maximum stress due to Saint Venant component by assuming the component equal to the external torque.
	Maximum stress due to warping component by assuming the component equal to the external torque (for I-beams and channels only)
Minor Bending	Maximum stress due to minor bending moment
	Maximum axial stress due to warping component (for I-beams and channels only).
Major Bending	Maximum stress due to major bending moment.

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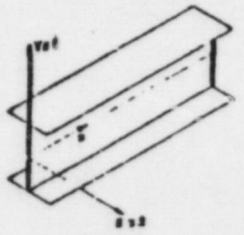
		Axial Stress	Shear Stress	Minor Bending Stress	Major Bending Stress
I	I - Beam H - Beam W - Beam	$\frac{F_x}{A}$	$f_{v_x}\left(\frac{B_3}{J}+\frac{1}{B_1B_2B_3}\right)+\frac{3F_z}{4B_2B_3}+\frac{F_y}{B_4B_5}$	$\frac{M_y}{Z_y}+\frac{M_x}{Z_y}\sqrt{2.6\frac{I_y}{J}}$	$\frac{M_z}{Z_z}$
[	H - Beam Channel	$\frac{F_x}{A}$	$f_{v_x}\left(\frac{B_3}{J}+\frac{1}{B_1B_2B_3}\right)+\frac{3F_z}{4B_2B_3}+\frac{F_y}{B_4B_5}$	$\frac{M_y}{Z_y}+\frac{M_x}{Z_y}\sqrt{2.6\frac{I_y}{J}}$	$\frac{M_z}{Z_z}$
L	L - Beam Angle	$\frac{F_x}{A}$	$f_{v_x}=\frac{1}{B_3}\left\lvert\frac{M_x}{B_3(B_1+B_2-B_3)}+\frac{F_z}{2B_2}+\frac{F_y}{2B_1}\right\rvert$	$\frac{M_y}{Z_y}$	$\frac{M_z}{Z_z}$
T	T - Beam Tee	$\frac{F_x}{A}$	$f_{v_x}=\frac{1}{B_3}\left\lvert\frac{M_x}{B_3(B_1+B_2-B_3)}+\frac{F_z}{2B_2}+\frac{F_y}{2B_1}\right\rvert$	$\frac{M_y}{Z_y}$	$\frac{M_z}{Z_z}$
⊖	Pipe	$\frac{F_x}{A}$	$2\sqrt{\frac{F_y^2+F_z^2}{A}}+\frac{M_xB_1}{4I_z}$	0	$\frac{\sqrt{M_y^2+M_z^2}}{Z_z}$
□	Box	$\frac{F_x}{A}$	$\frac{M_x}{2B_1B_2B_3}+\frac{3F_z}{8B_3(B_2-B_3)}+\frac{3F_y}{8B_3(B_1-B_3)}$	$\frac{M_y}{Z_y}$	$\frac{M_z}{Z_z}$

NOTE : All summations are absolute sums

Table 2.1.2.4.1. design stress formula

NOTATIONS

- A — Cross Section
- A_{sy} — Shear Section along minor axis
- A_{sz} — Shear Section along major axis
- J — Torsion Resistance
- I_y — Bending Inertia about minor axis
- I_z — Bending Inertia about major axis
- Z_y — Section Modulus about minor axis
- Z_z — Section Modulus about major axis
- B_1 to B_5 — Refer to Specific Section Dimensions
- F_x — Axial Load
- F_y — Shear Load along minor axis
- F_z — Shear Load along major axis
- M_x — Torsion Moment
- M_y — Bending Moment about minor axis
- M_z — Bending Moment about major axis



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Appendix E

Effects of Eccentricities Between Typical Tier and Post members.

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Appendix E

The purpose of this Appendix is to demonstrate that the net eccentricity between typical tier and the post members for X-axis loading, as defined in Figure 1, can be neglected. Other loading directions are not considered since their eccentricities are explicitly included in the support models.

The effect of the tier-to-post eccentricity for X-axis loading is to generate torsional moments in the post. Table E-1 identifies the maximum torsional stresses generated in post members as a result of the eccentricities calculated in Table 1. The following assumptions were made in the generation of this table.

1) Length of the post member is 5 feet.

2) The post is subjected to a lateral load (X-axis loading) of 1 kip. Other loads are not considered since they do not directly generate torsional moments in the post member as a result of this eccentricity.

3) The post member is assumed to be supported by a base angle (this is by far the most common connection detail). Therefore, warping stresses in the flanges of the post members are neglected. Warping stresses are otherwise calculated using the methodology outlined in the main body of this calculation (warping normal stresses are based on Ref [4], while warping shear stresses are based on Appendix D).

As can be seen from Table E-1, torsional stresses due to the eccentricity between the tier and the post are relatively small considering the relatively high lateral load (1.0 kip). The maximum warping normal and torsional shear stresses are less than 1.5 ksi. Since shear stresses will rarely govern the evaluation, the torsional shear stresses can be considered to be insignificant and they will not be discussed further.

The maximum warping normal stress in Table E-1 is shown to be 11 percent of the maximum bending stress. However, for most supports, the ratio of the peak warping normal stress generated by this eccentricity to the peak normal stress generated by all other factors is expected to be less than 5 percent. This inference is based on the following considerations:

1) Most post members will be longer than 5 feet. The bending stress in the post will increase in proportion to its length; however, the warping stress will only increase a relatively small amount for the cross sections shown. As a result, the ratio of the warping normal stress in the post to the bending stress will decrease significantly.

2) The post will carry other loads not considered in Table E-1. These loads will tend to increase the net normal stress without significantly increasing the warping component of this stress. These loads include:

- the vertical tray load which introduces axial force and weak axis bending in the post.

- the longitudinal tray load which introduces weak axis bending.

- self-excitation loads which can introduce axial forces, strong and weak axis bending, and warping normal stresses.

- loads due to the asymmetry of the support which can introduce axial forces, strong and weak axis bending, and warping normal stresses.

3) Warping stresses in Table E-1 are greatly over predicted due the conservatism of assuming that the base angles provide full warping restraint to the cross-section (see main body of this calculation).

For the relatively few cases where base plates are used, warping normal stress will be somewhat higher. However, based on the above considerations, they still can be considered to be negligible.

In conclusion, stresses in post members resulting from this eccentricity can be considered to be insignificant. Therefore the effect of this eccentricity can be neglected.

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Table E-1 Maximum Torsional Stresses in Post Members as a Result of the Tier-to-Post Eccentricity

Cross Section	Torsion Constant (in**4)	a (in)	d (in)	Flange Width (in)	Flange Thickness (in)	Web Warping Constant (in**4)	Maximum Eccentricity (in)	Warping Stress (ksi)	Section Modulus (in**3)	Bending Stress (ksi)	Warping Stress/ Bending Stress	Warping Shear (ksi)	Pure Torsional Shear (ksi)
C10X15.3	.211	23.70	10.00	2.60	.436	4.38	.225	.51	13.50	4.44	.11	.02	.46
C8X11.5	.131	18.10	8.00	2.26	.390	3.07	.185	.62	8.14	7.37	.08	.03	.55
C6X8.2	.076	12.70	6.00	1.92	.343	1.98	.140	.74	4.38	13.70	.05	.04	.63
C4X7.25	.082	6.25	4.00	1.72	.296	1.01	.073	.37	2.29	26.20	.01	.04	.26
MC6X12	.155	13.70	6.00	2.50	.375	2.47	.266	.80	6.24	9.62	.08	.05	.64

Notes : a is defined in Ref [4].
d is the depth of the web.
The warping constant is defined in Ref [4].

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Appendix F

Angle Stresses

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Appendix F

Angle stresses are calculated by the project using section properties about the geometric axes of the cross-section (axes parallel to the legs of the angle). Stresses will however, be somewhat if the principal axes are used.

This appendix evaluates the differences in stresses calculated about the principal axes and the geometric axes. It derives a factor which is to be applied to angle stresses to account for the stress differences.

Stresses about the principal axes are calculated from the following equation [Ref F-1]:

$$\sigma_{max} = \frac{M_y I_x - M_x I_{xy}}{I_x I_y - I_{xy}^2} C_x + \frac{M_x I_y - M_y I_{xy}}{I_x I_y - I_{xy}^2} C_y$$

where $M_{x,y}$ = moment about the x or y axis

$I_{x,y}$ = moment of inertia about the geometric x or y axis

I_{xy} = product of inertia

C_x, C_y = distance from the centroid to the extreme fiber.

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I_{xy} is calculated as follows:

for equal leg angles:

$$I_{xy} = r_z^2 A - I_x$$

where r_z = minimum radius of gyration as given in Ref F-2

for unequal leg angles:

$$I_{xy} = \frac{\tan 2\phi\,(I_y - I_x)}{2}$$

where ϕ = angle from the geometric axes to the principal axes

Table F-1 calculates stresses using principal axis and geometric axis properties for a 1.0 kip·in moment applied about the x-axis, for typical angles. Values of I_x, I_y and ϕ are taken from Ref F-2, except for the L6×3×3/4 where all properties are derived from Ref F-1. I_{xy} and principal axis stresses are calculated

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using the above equations. The stress about the geometric axis is calculated as follows:

$$\sigma = M_x / S_x$$

where S_x = section modulus about the geometric axis.

As can be seen from Table F-1, the maximum stress increase is 15 percent. To be conservative, the project will apply a 1.2 increase factor to stresses calculated about geometric axes.

References

F-1 McCormac, "Structural Steel Design", International Textbook Co, 1965

F-2 AISC Manual, 7th Edition

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Table F-1 Ratio of Principal Stresses to Geometric Stresses in Angles

Cross Section	Ix (in**4)	Iy (in**4)	Angle (degree)	Ixy (in**4)	Sx (in**3)	Cx (in)	Cy (in)	Principal Stress (ksi)	Geometric Stress (ksi)	Ratio
L 3x3x3/8	1.76	1.76	45	-1.0330	.83	-.888	2.112	1.38	1.20	1.15
L 3x3x3/16	.962	.962	45	-.5748	.44	-.820	2.180	2.73	2.27	1.20
L 4x3x3/8	3.96	1.92	28.85	-1.6135	1.46	-.782	2.720	.79	.68	1.16
L 3x2x1/2	1.92	.672	22.50	-.6240	1	-.583	1.920	1.03	1	1.03
L 6x4x3/4	24.50	8.70	Note 1	-8.51	6.25	-1.080	3.920	.18	.16	1.11

Note 1 : The angle is not required, Ixy is provided in Reference F-1

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Appendix G

Effects of Eccentricities Between the Centroid of Ties Members and the Clip Connection Point

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Appendix G

Since gage loactions for bolt holes in channel members are generally not at the centroid of the cross section, lateral tray loads will generate weak axis bending moments in the attached tier members as shown in Figure G.1. This bending moment will be resisted by the tier and the tray in proportion to their relative stiffnesses. The tier will respond in weak axis bending while the tray will respond in strong axis bending. This appendix accesses this behavior using the model shown in Figure G.2.

The following assumptions are made in the model:

- Support spacing is assumed to be 9 feet. (This is an upper bound (see Reference G.1)).
- Trays are assumed to have 100 percent cable fill and to have thermolag.
- The bounding acceleration of 3.0g for the reactor building internal structures and safe guards building is used in the evaluation (see Reference G.1).
- The tray is assumed to be supported at the mid point of the tier.
- The tray is assumed to be simply supported at the adjacent supports.
- The tier is assumed to be simply supported at the posts.
- Only OBE is investigated since the ratio of SSE accelerations to OBE accelerations is less than the ratio of SSE allowables to OBE allowables (see Reference G.1).
- Stiffness of the tier and tray are taken to be 12EI/L (kip*in/rad) (see Reference G.2).

Taken together, the above assumptions will result in conservative tier stresses.

The following tier cross sections are evaluated:

- C4x7.5
- C8x11.5
- C6x8.2

These cross sections are used extensively and are considered to be representative.

The following tray types are investigated:

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- 24x4 Trough
- 36x6 Trough
- 24x4 Ladder

These tray types represent the more massive trays found in th plant and will therefore produce bounding tier stresses. Th also represent a wide range of tray stiffness properties.

Tier stresses are calculated in Table G.1. Note that the maxi bending stress is less than 6% of the allowable. Given conservative assumptions made in the calculations, this st level is considered to be insignificant. Therefore, eccentricity between the gage location and the centroid o tier can be neglected.

References

G.1 Impell project instruction PI-01, Job # 0240-040, Revision 2.

G.2 Blodgett "Design of Welded Structures", James F. Lincoln Arc Welding Foundation, June 1966

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1	WLC	4-6-96	RVP	5-2-96	IMPELL CORPORATION
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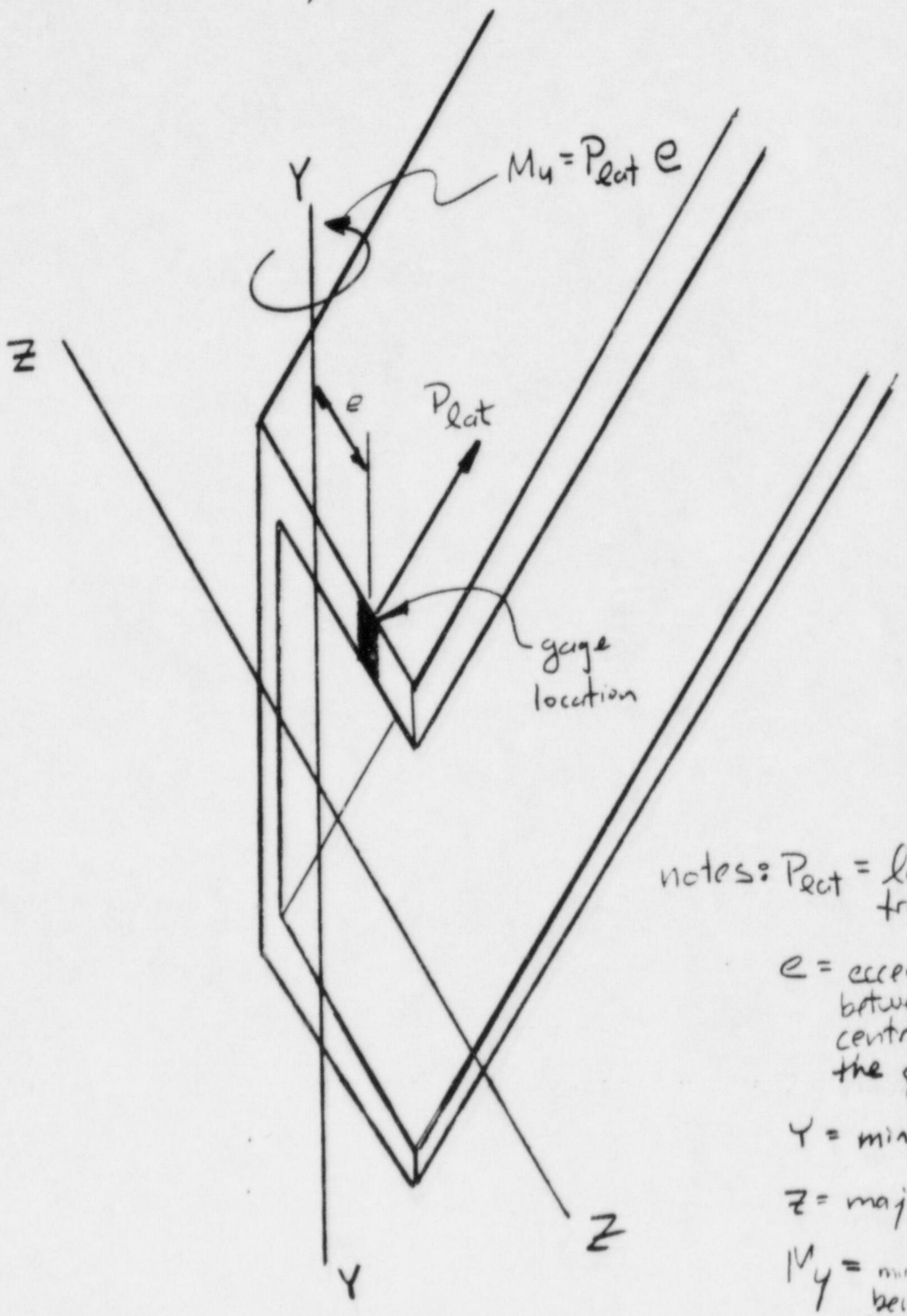
Figure G.1 Minor Axis Bending Moments Generated By Both Hole Eccentricities



notes: P_{lat} = lateral tray load

e = eccentricity between the centroid and the gage location

Y = minor axis

Z = major axis

M_y = minor axis bending due to P_{lat}

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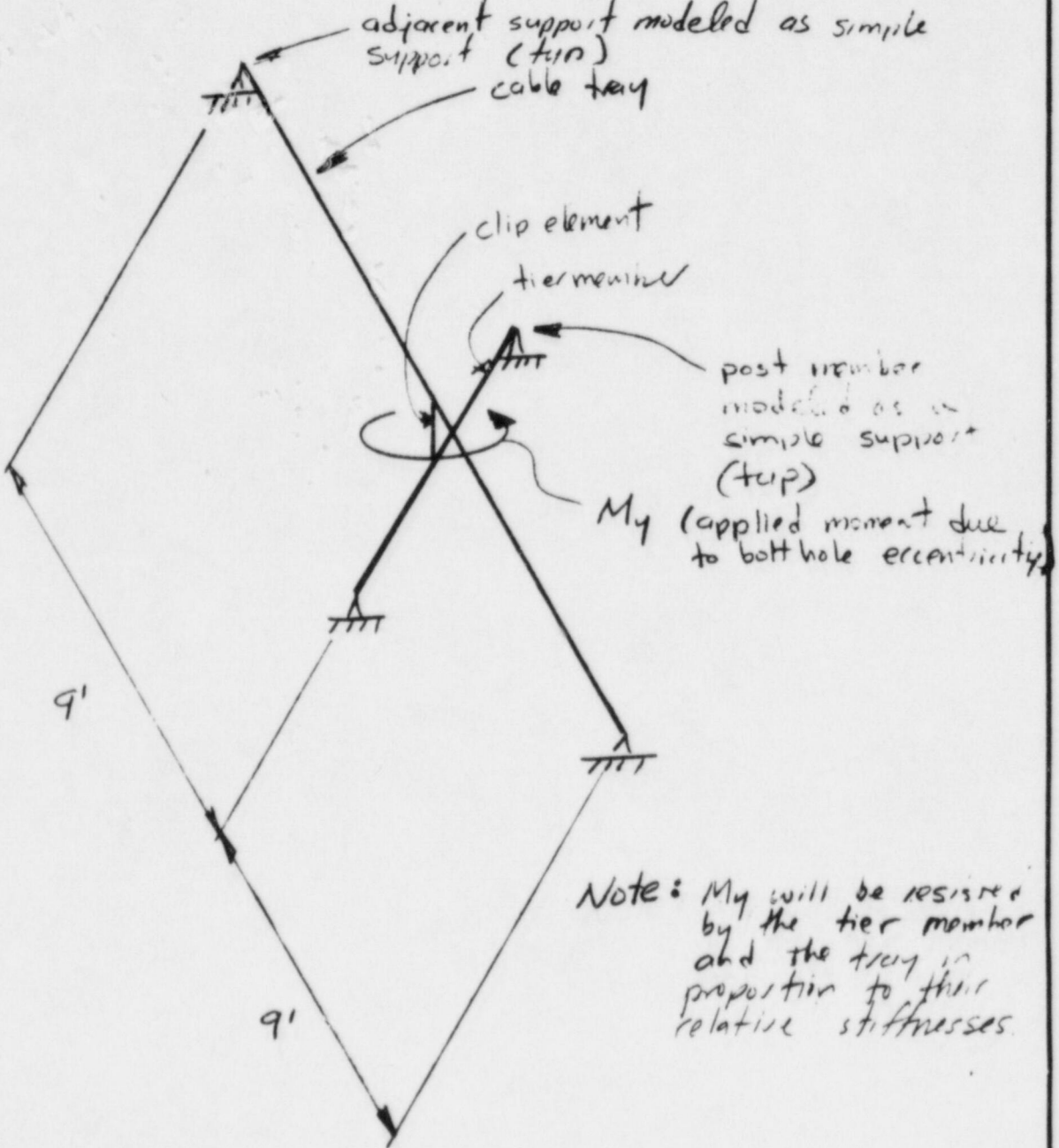
Figure C.2 Analysis Model.



Note: My will be resisted by the tier member and the tray in proportion to their relative stiffnesses.

					Typical
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Table 6.1

Effect of Eccentric Bolt Holes on Weak Axis Bending of Tier Members

Tier Member Size	Tray Size	Tier Minor Axis Properties Iyy (in^4)	Syy (in^3)	Support Stiffness (kip*in/rad)	Tray Moment of Inertia (in^4)	Tray Stiffness (kip*in/rad)	Tray Weight (lb/ft)	Eccentricity (in)	Tray Load (kips)	Maximum Moment (kip*in)	Support Moment (kip*in)	Tier Stress (ksi)	Percent of Allowable
C 4x7.25	24x4 Trough	.432	.343	4176.00	8.92	14371.11	99.5	.54	2.69	1.45	.164	.48	2.2
C 8x11.5	24x4 Trough	1.320	.781	12760.00	8.92	14371.11	99.5	.80	2.69	2.16	.500	.65	3.0
C 6x8.2	24x4 Trough	.692	.492	6689.33	8.92	14371.11	99.5	.61	2.69	1.65	.262	.53	2.5
C 4x7.25	36x6 Trough	.432	.343	4176.00	5.77	9296.11	147.0	.54	3.97	2.15	.333	.97	4.5
C 8x11.5	36x6 Trough	1.320	.781	12760.00	5.77	9296.11	147.0	.80	3.97	3.19	.923	1.18	5.5
C 6x8.2	36x6 Trough	.692	.492	6689.33	5.77	9296.11	147.0	.61	3.97	2.43	.509	1.03	4.8
C 4x7.25	24x4 Ladder	.432	.343	4176.00	4.39	7072.78	99.5	.54	2.69	1.45	.270	.79	3.6
C 8x11.5	24x4 Ladder	1.320	.781	12760.00	4.39	7072.78	99.5	.80	2.69	2.16	.695	.89	4.1
C 6x8.2	24x4 Ladder	.692	.492	6689.33	4.39	7072.78	99.5	.61	2.69	1.65	.400	.81	3.8

Assumptions

Tier Length (ft): 3.00
Support Spacing (ft): 9.00
Acceleration (g): 3.00
Allowable Stress (ksi): 21.60

The eccentricity is taken to be the distance between the usual gage and the centroid.

Bolt holes are assumed to be located at the standard gage locations specified in the AISC Manual 7th Edition.

Stiffnesses are taken to be 12EI/L.

Tray weight data assumes 100% fill.

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chk RVP 5-2-86

CALCULATION/PROBLEM COVER SHEET

Calculation/Problem No: 020-040-N19

Title: Clip Stiffness Production Value

Client: TUSCO Project: CPSES

Job No: 0210-040

Design Input/References:

SEE p. 13

Assumptions:

DOCUMENTED WITHIN.

Method:

DOCUMENTED WITHIN.

Remarks:

—

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1	Remove the subgroup of A, G and C clips	Linda J Barrett	3/25/86
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Appendix

"Cable Tray Clip Stiffness Summary Table", 0210-040-M10, Rev. 1 page 1.2 and 3 of 4

1. Purpose

The purpose of this calculation is to establish the methodology to calculate the production values of clip stiffness used for cable tray systems at CPSES, and to calculate the clip stiffness based on the guidelines established. The verification of the methodology against the critical values of clip stiffness will be performed later.

2. Introduction

Cable trays, clips and support assembly forms an integral cable tray system. To analyze the system, the component properties must be determined. Normally this information is unique provided that the component types are known. However, due to the inassessability caused by the coverage of thermolag or other reasons, the component types may not be known. In order to perform the analysis for cable tray system, the selection of properties for unidentifiable components becomes important. Furthermore, if there are numerous types of components that exist, to perform the analysis becomes increasingly complex.

An approach will be taken to reduce the amount of

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					Clip Stiffness Production Value		
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variation in component properties.

In this calculation, we focus on the simplification of cable tray clips. To achieve that, we group the clip types into transverse and longitudinal depended upon their functions, and select the appropriate production values for each group which governs the critical values of its group. To remove the unnecessary uncertainty and reduce the variation of critical values,

the production values of transverse clips are made as tray size dependent. Longitudinal clips are to be grouped. 1

3.0 Calculation

See next five pages

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					Clip STIFFNESS PRODUCTION VALUE		
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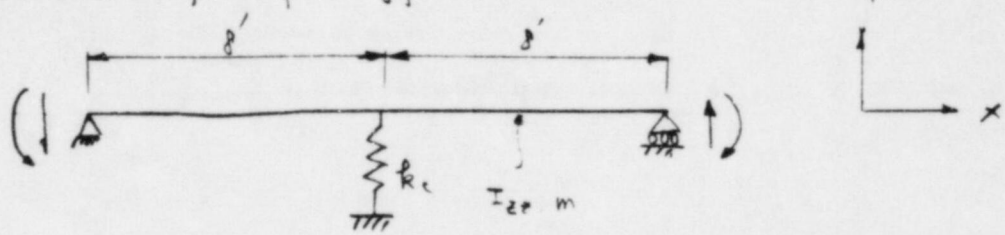
3.1 Production value of clip stiffness k_x

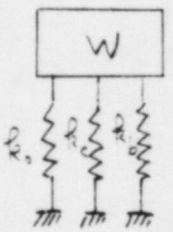


For smallest $I_{zz} = .81\ in^4$, $M_{max} = 142\ \#/ft$ (Ref 2)

Let k_o represents the stiffness of cable tray (simply supported)

$$k_o = \frac{48EI}{L^3} = 163\ \#/in$$

Let the stiffness k_b represents the boundary conditions generated by the continuation of tray system. Then the above system can be idealized as



where $k_c, k_b > 0$.

The effective inertia mass will be half of total weight.

$$W_{ef} = \frac{1}{2} \times 142 \times 16 = 1136\ \#$$

The frequency of idealized model is

$$f = \frac{1}{2\pi}\sqrt{\frac{(k_o + k_c + k_b)g}{W}}$$

Let the production value of $k_c = k_{min} + \Delta k_c$

$$f = \frac{1}{2\pi}\sqrt{\frac{(k_o + k_{min} + \Delta k_c + k_b)g}{W}}$$

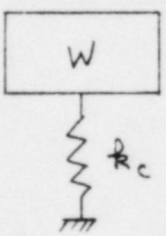
For smallest value of $k_{min} = 3.6$ k·p/in (Ref. 1)

$$f > \frac{1}{2\pi}\sqrt{\frac{(163 + 3600)\,g}{1136}} = 5.69 \text{ Hz}$$

The peak accelerations of safeguard building spectrum @ different elevations are occured around 4 to 10 Hz. In order to pick the highest possible acceleration level in calculating the system response, the k_c should be equal to zero, ie, the low values of clip stiffness k_x should be used for production purpose.

3.2 Production value of longitudinal clip stiffness K_y

To estimate the longitudinal frequency of a cable tray system, we assume 10 tributary span per longitudinal support, each span with 8' and maximum weight 142 #/ft. The spring mass system is used to calculate the longitudinal frequency



$$W = 10 \times 8 \times 142 = 11360 \text{ \#}$$

$$f = \frac{1}{2\pi}\sqrt{\frac{k_c g}{W}}$$

					TUGIO Clip Stiffness Production Value		
						JOB NO 0210-060	PAGE 6
0	CMH	3/6/86	RPL	3/7/86	IMPELL CORPORATION	CALC NO 0210-060-M19	OF 13
REV	BY	DATE	CHECKED	DATE			

Let the production value of $k_c = k_{min} + \Delta k_c$

$$f = \frac{1}{2\pi}\sqrt{\frac{(k_{min}+\Delta k)g}{w}}$$

The minimum k_y for the longitudinal support is

$$k_{min} = 150 \text{ kip/in} \qquad \text{(Ref. 1)}$$

Then

$$f > \frac{1}{2\pi}\sqrt{\frac{150000 \times 386.4}{11360}} = 11.368 \text{ Hz}$$

In order to obtain the maximum system response, the low value of K_y of longitudinal type clips should be used for production purpose as explained before. Transverse clips will use method shown below for K_y. ⚠

3.3 Production value of other components of clip stiffness

For other components of clip stiffness, the same methodology may be applied. However, they are highly depended upon the clip location and the tray supporting system. In the other words, they are system dependent. The method used above for estimating the production values of other components is not suitable.

As we know, frequency is proportional to square root of stiffness. In order to minimize the variation of system frequency and responses, the production value of clip stiffness

REV	BY	DATE	CHECKED	DATE	TUSCO Clip Stiffness Production Value		
1	cyh	3/19/86	RTK	3/24/86	IMPELL CORPORATION	JOB NO 0210-040	PAGE 7
0	cyh	3/6/86	RTK	3/17/86		CALC NO 0210-040-M19	OF 13

should be related to the low and high value of clip stiffness in the same manner. The relationship is developed as follows.

For low stiffness k_ℓ, the frequency f_ℓ

$$f_\ell = A\sqrt{k_\ell} \qquad \text{where } A \text{ is a constant.}$$

For high stiffness k_h, the frequency is

$$f_h = A\sqrt{k_h}$$

To minimize the frequency variation, $f_p = \frac{1}{2}(f_\ell + f_h)$

$$f_p = \tfrac{1}{2}\left[A\sqrt{k_\ell} + A\sqrt{k_h}\right] = A\left[\frac{\sqrt{k_\ell} + \sqrt{k_h}}{2}\right]$$

For a given k_p the f_p is

$$f_p = A\sqrt{k_p}.$$

Therefore, the value of k_p should be.

$$k_p = \left[\frac{k_\ell^{\frac{1}{2}} + k_h^{\frac{1}{2}}}{2}\right]^2$$

Furthermore, if the clip stiffness is much higher than that of tray system, the variation in system responses due to change of clip stiffness is expected to be minimal. The clip stiffness of 10^3 k-in/in and 10^3 k-in/RAD is used as cut off points. The above formula for k_p can, then, modified to

					TUSCO		
					Clip Stiffness Production Value		
0	CJH	3/6/86	RTL	3/7/86	IMPELL CORPORATION	JOB NO 0210-0060	PAGE 8
REV	BY	DATE	CHECKED	DATE		CALC NO 0210-060-M19	OF 13

$$K_p = \left[\frac{K_\ell^{1/2} + K_h^{1/2}}{2} \right]^2 \qquad \text{if } K_\ell, K_h < 10^3 \text{ or } K_\ell, K_h > 10^3$$

$$\text{or} \quad K_p = \left[\frac{K_\ell^{1/2} + (10^3)^{\frac{1}{2}}}{2} \right]^2 \qquad \text{if } K_\ell < 10^3 < K_h$$

4.0 Summary of Calculation

See next four pages for tables.

					TUSCO Clip Stiffness Production Value		
						JOB NO 0210-040	PAGE 9
0	CYH	3/6/86	RRL	3/7/86	IMPELL CORPORATION	CALC NO 0210-040-M1D	OF 13
REV	BY	DATE	CHECKED	DATE			

IMPELL CORPORATION

CALCULATION/PROBLEM COVER SHEET

Calculation/Problem No: M-27

Title: Thermal Load Position Paper

Client: Tuscco Project: CPSES

Job No: 0210-040

Design Input/References:

Noted within

Assumptions:

Noted within

Method:

Noted within

Remarks:

This calculation summarizes Tuscco's position on the evaluation of thermal loads for cable tray hangers

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Sheet _____ of _____

1.0 INTRODUCTION

This paper describes Texas Utilities Generating Company's (TUGCO) licensing position regarding the evaluation of thermal loading as part of the design verification of cable tray systems at Comanche Peak Steam Electrical Station (CPSES) Units 1 and 2.

Thermal loading was not initially evaluated in the design verification of cable tray systems at CPSES. As part of the Comanche Peak Independent Assessment Program (IAP), Cygna commented that thermal effects should be considered where such loads occur. Based on the considerations presented in this paper, TUGCO's position is that it is not necessary to consider thermal loads in the evaluation of cable tray systems.

2.0 BEHAVIOR OF CABLE TRAY SYSTEMS UNDER THERMAL LOADS

2.1 Nature of Thermal Loads

Thermal stresses in cable tray systems can be classified as secondary stresses since they are developed as a result of self-constraint and are not generated by external loads. Local yielding, joint slippage, or minor distortions of the system can satisfy the expansion conditions which cause the stresses to occur.

Thermal loading is not significant for cable tray systems due to the secondary nature of the loads and due to the flexibility of the cable tray systems which can adequately accommodate thermal growth without generating significant loads. The flexibility of these systems results from the high flexibility of the trays and supports in addition to the existence of gaps in bolted splices and clips.

As a result of the above considerations, the general industry practice is to neglect thermal loading in the design of cable tray systems.

2.2 Thermal Loading Conditions

Two thermal loading conditions exist: T_o and T_a. T_o is defined as the thermal loading that occurs during normal operation or shutdown conditions and T_a is defined as thermal loading during a postulated break inside containment.

The maximum temperature change under T_o conditions is +32 degrees Fahrenheit (Reference [1]). Since this small temperature change occurs gradually, the building structures will heat up at the same rate as the enclosed cable tray systems. Therefore, an effective coefficient of expansion equal to the difference in the coefficients of expansion of steel and concrete can be used to evaluate thermal effects. Since the coefficient of thermal expansion of concrete is nearly equal to that of steel (the difference is approximately 1.0×10^{-6}) and the temperature change for this load case is relatively small, T_o conditions will not result in significant thermal loads.

T_a conditions are limited to the reactor building. For this load case the temperature change is considerably higher; however, as mentioned in the following section, it is not required to evaluate T_a conditions for cable tray systems.

3.0 FSAR LOAD COMBINATIONS

Cable trays systems are designed for load combinations specified for "other structures" in Section 3.8.4 of the CPSES FSAR (Reference [2]). The following is a summary of these load combinations as they apply to the evaluation of cable tray systems:

(1) $S = D + L + F_{eqo}$

(2) $1.5\ S = D + L + T_o + F_{eqo}$

(3) $1.6\ S = D + L + T_o + F_{eqs}$

(4) $1.6\ S = D + L + T_a + F_{eqo}$

(5) $1.7\ S = D + L + T_a + F_{eqs}$

where: S = required section strength based on the elastic design methods as defined in Part I of the AISC Manual of Steel Construction (Reference [3])

D = dead load

L = live load

T_o = thermal loads during normal operation or shutdown conditions

T_a = thermal loads under thermal conditions generated by the postulated break and including T_o

F_{eqo} = loads generated by half the SSE

F_{eqs} = loads generated by the SSE

Per Section 3.8.4 of the FSAR, thermal loads do not have to be considered in load combinations (3), (4), and (5) when the material is ductile since they are secondary and self-limiting in nature. Additionally, the overall high system flexibility, including general looseness of connections, prevents potentially nonductile components such as concrete anchors from experiencing thermal loads. Therefore, it is not necessary to evaluate T_a conditions to satisfy FSAR commitments.

4.0 PRECEDENT

Based on a review of the FSARs and other design documents, Millstone 3, Braidwood, Byron, and Catawba have not considered thermal loads in the evaluation of cable tray systems. All of these plants are of the same vintage Westinghouse NSSS design as CPSES and share the same licensing position presented herein.

5.0 CONCLUSION

In accordance with FSAR commitments it is not necessary to consider T_a conditions for cable tray systems. Since T_o conditions will not be significant, it is not necessary to consider either thermal load case in the evaluation of cable tray systems. This position has been successfully applied to other plants of the same vintage design as CPSES.

REFERENCES

[1] "General Instructions for Cable Tray Hanger Analysis for Comanche Peak Steam Electrical Station No. 1 and 2," Revision R2, December 20, 1985, by Ebasco Services Incorporated.

[2] Comanche Peak Steam Electric Station, Final Safety Analysis Report (Amendment 55, July 19, 1985).

[3] AISC - Manual of Steel Construction 7th Edition, including Supplements No. 1, 2, and 3.