EBASCO SERVICES INCORPORATED

Seismic Design Criteria For Cable Tray Hangers For Comanche Peak Steam Electric Station No. 2

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EBASCO SERVICES INCORPORATED
2 World Trade Center
New York, NY 10048

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I. Purpose

A cable tray hanger is classified as a seismic Category I structure, and therefore, it shall be adequately designed for the effect of the postulated seismic event combined with other applicable and concurrent loads. The design requirements for seismic Category I structure are delineated in Regulatory Guide 1.29. This document provides the seismic design guideline for cable tray hangers of Comanche Peak Steam Electric Station Unit No. 2. These guidelines summarize the design parameters, applicable load combinations and their associated acceptance criteria, the various design approaches and their corresponding seismic input criteria. The following sections describe in detail the guidelines for the seismic design of the cable tray hangers and lists the applicable reference documents. In addition, cable trays shall be design verified per Reference 13 and cable tray clamps shall be design verified per Reference 14. R6

II. Reference Documents

The following lists the documents referenced or prepared by Gibbs & Hill Inc. which will continue to be used for the design of seismic Category I cable tray hangers for Comanche Peak Steam Electric Station Unit No. 2.

1. Applicable Codes And Regulatory Guides

- Regulatory Guide 1.29 - Seismic Design Classification, Rev. 3, September 1978.
- Regulatory Guide 1.61 - Damping Values for Seismic Design of Nuclear Power Plants, October 1973.
- Regulatory Guide 1.89 - Qualification of Class 1E Equipment for Nuclear Power Plants, Rev. 1, June 1984.
- Regulatory Guide 1.92 - Combining Modal Responses and Spatial Components in Seismic Response Analyses, Rev. 1, February 1976.
- NUREG 1.75 - Standard Review Plan Section 3.8.4, November 1975.
- AISC - Manual of Steel Construction, 7th Edition, including Supplements No. 1, 2 & 3.
- AWS D1.1-75 - Structural Welding Code.

2. Cable tray specification No. 2323-ES-19, Revision 1, dated Nov. 22, 1976.

3. CPSES/FSAR Section 3.8.4.3.3 "Load Combinations and Acceptance Criteria for Other Seismic Category I Steel Structures"

4. Design Criteria for Cable Tray Supports and Their Arrangement, Gibbs and Hill Calculation Book No. SCS - 113C 3/9-3/24

5. Structural Embedments Specification No. 2323-SS-30 Gibbs & Hill, Revision 2, June 13, 1986.

6. Design procedure: DP-1 Seismic Category I, Electrical Cable Tray Supports dated June 11, 1984.

7. Refined Response Spectra for Fuel Handling Building, dated Oct. 1985 for SSE and OBE.

8. Refined Response Spectra for Reactor Building Internal Structure, dated Jan. 1985 for SSE and Jan. 1983 for OBE.

9. Refined Response Spectra for Containment Building, dated Jan. 1985 for SSE and Jan. 1983 for OBE.

10. Refined Response Spectra for Auxiliary Building, dated Nov. 1984 for SSE and Jan. 1983 for OBE.

11. Refined Response Spectra for Electrical Building, dated Nov. 1984 for SSE and Nov. 1982 for OBE.

12. Refined Response Spectra for Safeguards Building, dated Nov. 1984 for SSE and Jan. 1983 for OBE.

13. Ebasco Comanche Peak SES Cable Tray Hanger Volume I, Book 1, Parts 1, 2 & 3, General Input Data, Revision 3, Revision 0, and Revision 0 respectively.

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14. Ebasco document SAG.CP19, Design Criteria and Procedures for Design Verification of Cable Tray Clamps for CPSES Units 1 & 2, Rev. 1, 1/15/87.

III. Design Parameters for Cable Tray Hangers

The parameters considered in the design of cable tray hangers are as follows:

1. Cable tray span length

"As-built" span lengths shall be used in the hanger design verification.

2. Maximum cable tray loading

Tray Size	Total Unit Weight (Lbs/Foot)
6"	18
12"	35
18"	53
24"	70
30"	88
36"	105

Note:

a. The above data is applicable for both ladder and solid bottom types of trays.

b. The above data is also applicable for various heights of tray side rails.

c. The above unit weight includes cable, tray, tray cover and side rail extension.

d. The above unit weight does not include fire proofing material weight (Thermolag and Thermoblanket).

e. For trays which are fire proofed, the unit weight of cable tray including the weight of fireproofing to be used is in the "General Instructions For Cable Tray Hanger Analysis".

f. All cable tray hangers shall be design verified based on "as-built" drawings (ie. hanger members, connection and anchorage details). |R6

g. All cable tray hanger components (members, connections base angles, base plates and anchor bolts) shall be design verified.

3. Material

a. Support structure is ASTM A36

b. Expansion anchors are Hilti Kwik and Super Kwik Drilled-in bolts

c. Screw anchors are Richmond inserts

d. Embedded plates (strip and area plates) are ASTM A36

4. Design loads

The cable tray hangers shall be designed for the following loads and load combinations:

a. Load definitions

Normal loads, which are those loads encountered during normal plant operation and shutdown, include:

D - Dead loads and their related moments and forces.

L - Live load equals zero. |R6

To - Thermal effects and loads during normal operating or shutdown conditions, based on the most critical transient or steady state condition.*

Severe environmental load includes:

F_{eqo} - Loads generated by the operating basis earthquake including secondary wall displacement effects. |R6

Extreme environmental load includes:

F_{eqs} - Loads generated by the safe shutdown earthquake including secondary wall displacement effects. |R6

* Except for anchorage components, accident temperatures (Ta) are not considered in design verification per CPSES FSAR (Pg. 3.8-83 and 3.8-110). Accident thermal loads on anchorages are considered generically by studies. Furthermore, per AISC Manual of Steel Construction (Pg. 6-9), no reductions in F_y are required for temperatures up to 700°F. |R6

b. Load combinations

The following load combinations shall be considered in design of cable tray hangers:

i. $D + L + F_{eqo} = S$
ii. $D + L + To + F_{eqo} = 1.5S$
iii. $D + L + To + F_{eqs} = 1.6S$

where S is the required strength based on elastic design methods and the allowable stresses defined in Part 1 of the AISC "Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings" (published in the Manual of Steel Construction, seventh edition). In no case shall allowable stress exceed 0.90 F_y for normal tensile stresses and 0.50 F_y for shear stresses. |R6

IV. Seismic Design Approaches, Seismic Input Requirement, and Design Acceptance Criteria

There are several analytical methods available which will be used in design or design verification of cable tray hangers. Because the level of sophistication is not the same for each method, the seismic input requirement must vary in order to compensate for whatever the method lacks in sophistication, and therefore the conservatism of results associated with each analysis method also varies.

For span layouts not in conformance with Appendix 4 of this design criteria, design verification may be performed by the Response Spectrum Method (Section IV.3) or, if appropriate, by the Equivalent Static Method (Section IV.2) per Attachment Y of the General Instructions.

The following procedures describe the three (3) most acceptable methods: static analysis, equivalent static method and response spectrum method. The seismic input criteria for each analysis method is also addressed.

IV.1 STATIC ANALYSIS

a. Finite Element Model

A 3-D model shall be prepared to represent cable tray hangers. An offset or eccentricity due to the assemblage of various types of structural members and/or transmission of loads shall be considered in the preparation of the computer model.

Boundary conditions at anchorage points shall properly represent the actual anchorage condition. |R6

b. Cable Tray Loading

The total cable tray loading for each run shall be calculated based on the size of tray and the actual tray span lengths which are shown on the support drawing.

The cable tray loading shall be lumped as a nodal weight at the actual location on the tier and, if not known, at such a location on the tier that it will induce the worst member stress responses and the maximum anchorage reactions.

c. Seismic Input "g" Values

For a static analysis the peak spectral "g" values from the 4% damping OBE curves and the 7% damping SSE curves which were generated at the mounting locations of cable tray hangers shall be used multiplied by a coefficient to account for multimode response. These peak spectral "g" values for various buildings and different floor elevations can be found in the Appendix 1. For the case where the hangers were supported off the wall, the envelope of the the response spectrum curves for the floor immediately above and below the hanger location shall be used. The required seismic design "g" values in three (3) orthogonal directions are 1.25 (multimode response multiplier-MRM) times the peak spectral "g" values. R6

d. Static Analysis

The seismic load effect on the cable tray hangers will be treated as a static load. The dynamic effect from both seismic event and response characteristics of support structure are conservatively considered by using the 1.25 times the peak spectral "g" value as an input. However, for transverse type cantilever and trapeze cable tray hangers, the seismic load effect due to the hanger's self-weight in the longitudinal direction (direction parallel to tray run) shall be determined by multiplying the spectral "g" value corresponding to the CTH fundamental (lowest) longitudinal frequency by 1.25 regardless of whether that frequency is to the left or right of the peak response frequency.

If the cable tray hanger is attached to a steel structure, use 1.5 times the peak spectral "g" value and a fixed base boundary condition.

The static analysis shall be performed for the following load cases individually:

i) Dead load

ii) Seismic load in vertical direction

iii) Seismic load in transverse direction

iv) Seismic load in longitudinal direction

v) Thermal load if any

Note: Seismic load includes both OBE and SSE events.

e. Analysis Results

The following maximum responses shall be obtained for each load combination:

i) Maximum member stresses (bending, axial and shear) and nodal displacements shall be obtained. The stresses and displacements resulting from the simultaneous effect of three earthquake components shall be obtained by using the SRSS method. |R6

ii) Maximum anchorage reactions shall also be obtained by using SRSS method to account for the simultaneous effect of three earthquake components.

f. Seismic Design Acceptance of Cable Tray Hangers and their Anchorages

The cable tray hangers and their anchorages are considered to be acceptable when the structural member and connection stresses and the anchorage reactions, which are induced by the load combinations described in Sections III.4.b, are within the allowable stress limits and allowable anchorage carrying capacity. The following describes the acceptance criteria for both support structure and anchorages:

i. Support Structure

The structural member seismic design acceptance shall be evaluated using AISC interaction formula with modification for various load combinations as follows:

$\left(\frac{fa}{Fa} + \frac{fbx}{Fbx} + \frac{fby}{Fby}\right) \leq 1.0$ for load combination III.4.b.i

$\left(\frac{fa}{Fa} + \frac{fbx}{Fbx} + \frac{fby}{Fby}\right) \leq 1.5$ for load combination III.4.b.ii

$\left(\frac{fa}{Fa} + \frac{fbx}{Fbx} + \frac{fby}{Fby}\right) \leq 1.6$ for load combination III.4.b.iii

$f_v \leq F_v$ for load combination III.4.b.i

$f_v \leq 1.5F_v \leq 0.50\ Fy$ for load combination III.4.b.ii |R6

$f_v \leq 1.6\ F_v \leq 0.50\ Fy$ for load combination IV.4.b.iii |R6

where fa = axial stress

f_v = shear stress

fbi = bending stress

Fa, Fbi and F_v = allowable stresses for axial, bending and shear stress, per AISC 7th edition, and in all cases no more than 0.90 Fy for normal stress and 0.50 Fy for shear stress. |R6

ii. Anchorage (anchors)

- o Kwik-bolt and Super Kwik-bolt.

  The design criteria and allowable loads for above driven-in bolts are tabulated in Appendix 2.

- o Screw Anchors.

  The design criteria and allowable loads for screw anchors are contained in Appendix 2. When a redline drawing does not identify the bolt/thread rod material in a Richmond Insert, A-36 material shall be assumed in the cable tray hanger design verification.

Note: 1. The allowable loads for Hilti expansion anchors for the load combination involving OBE are the load capacities corresponding to a safety factor of 5, and for the load combination involving SSE are the load capacities corresponding to a safety factor of 4.

2. The safety factors for Richmond Anchors are 3.0 for both OBE and SSE. | R6

3. Prying action on anchor bolt, if any, shall be included. The effects of the flexibility of the base plate on the anchor bolt shall be considered.

4. For floor-mounted CTHs in building areas with concrete topping, the actual anchor bolt embedded length (as determined from the redline drawing) shall be reduced by two inches (2") to account for the topping.

IV.2 EQUIVALENT STATIC METHOD

a. Finite Element Model

See Section IV.1.a

b. Cable Tray Loading

See Section IV.1.b

c. Seismic Input "g" Value

i. The fundamental (lowest) frequency of cable tray hanger (f_h) shall be determined in each of three (3) orthogonal directions separately.

ii. Determine the frequency of cable tray itself corresponding to the actual span length (f_c) in each of three (3) orthogonal directions separately.

iii. Determine the system frequency using the following conservative formula:

$$\frac{1}{fsys^2} = \frac{1}{f_c^2} + \frac{1}{f_h^2}$$

When f_c or f_h are 33 H_z or larger this term's contribution to system frequency may be disregarded.

The above system frequency will be calculated for each of three (3) orthogonal directions separately.

iv. Obtain the spectral "g" value corresponding to the system frequency (fsys) for each direction separately when fsys is on the right side of the peak response frequency. If fsys is at the left side of the peak frequency, the peak spectral "g" value shall be used except as noted in Section IV.1.c and d.

v. Determine the required seismic design "g" values for the cable tray hanger by multiplying 1.25 to the above "g" value (obtained in Step iv) to account for multimode response except as noted in Section IV.1.c and d.

d. Equivalent Static Method

The stress analysis for the cable tray hangers shall be performed on the 3-D finite element model using the "g" value obtained in Step c. The load cases which shall be considered are the same as those listed in Section IV.1.d.

e. Analysis Results

See Section IV.1.e.

f. Seismic Design Acceptance of Cable Tray Hangers and their Anchorages

See Section IV.1.f.

IV.3 RESPONSE SPECTRUM METHOD

a. 3-D Model of Cable Tray and Tray Hangers

Construct a 3-D model of tray systems which include and therefore simulate the dynamic behavior of cable tray itself and cable tray hangers.

In order to adequately simulate the seismic response of the cable tray system, a minimum of 4 cable tray spans shall be included in the model, with two spans on each side of the hanger under consideration. The cable tray will be represented by a beam type finite element in the 3-D model, with properties obtained from tray Vendor's static load test report.

The stiffness of longitudinal supports shall also be considered and simulated by a spring constant attached to the ends of 3-D model.

b. Frequency Analysis

Perform a frequency analysis on the above 3-D model which includes all modes up to 33 Hz. Total modal mass shall be 90% of the total mass. If it is not, the residual mass shall be multiplied by the largest spectral acceleration at or beyond the cut-off frequency and applied as a rigid body force on the structure. R6

c. Spectral Analysis

Perform seismic response analysis for the above 3-D model using the appropriate floor response spectrum as an input. NRC Reg. Guide 1.92 shall be followed in calculating the modal response.

The 4% damping of OBE curves and 7% damping of SSE curves shall be used as an input for each direction separately. Seismic responses are obtained directly from these analyses using modal superposition per NRC Reg. Guide 1.92.

d. Response Spectra Analysis

The stress analysis for cable tray hangers shall be performed on the 3-D finite element model using the "g" value obtained in Step c. The load cases which shall be considered are the same as those listed in Section IV.1.d. R6

e. Analysis Results

See Section IV.1.e

f. Seismic Design Acceptance of Cable Tray Hangers and their Anchorages

See Section IV.1.f.

V. Recommendation of Successive Methods to be Used for Design of Cable Tray Hangers

The cable tray hangers may be designed or design verified by a static analysis method first (IV.1). If the cable tray hangers fail to meet the seismic requirement under this most conservative method, a refined analysis method of equivalent static method (IV.2) shall be used. If the cable tray hangers still fail to meet the design criteria, then the response spectrum method (IV.3), may be used. The response spectrum method approach simulates better the dynamic behavior of the cable tray system under the effect of the postulated seismic event and thus may produce seismic responses of the structural system closer to reality. Therefore, by response spectrum method, the conservatism associated with the seismic response obtained from static analysis and equivalent static method can be reduced to a minimum. In conclusion, if the cable tray hangers still fail to pass the acceptance criteria by a spectral response analysis, a much more refined analysis such as a time history analysis method can be used. A procedure for such analyses will be given, should the need arise.

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APPENDIX 1

Peak Acceleration Tables

1. Reactor Building Internal Structure

Floor Elevation (Ft)	Peak "g" Value			
	OBE 4%		SSE 7%	
	H	V	H	V
905.75	2.95	1.54	2.99	1.94
885.50	2.41	1.45	2.45	1.82
860.00	1.73	1.34	1.78	1.68
832.50	0.99	1.23	1.08	1.53
808.00	0.54	1.14	0.67	1.41
783.58	0.47	1.06	0.54	1.31

2. Safeguard Building

Floor Elevation (Ft)	Peak "g" Value			
	OBE 4%		SSE 7%	
	H	V	H	V
896.5	2.283	1.5	2.45	2.01
873.5	2.079	1.637	2.26	2.212
852.5	1.605	1.458	1.75	2.041
831.5	1.141	1.30	1.16	1.809
810.5	0.701	1.26	0.86	1.747
790.5	0.429	1.049	0.62	1.456
785.5	0.392	1.017	0.57	1.410
773.5	0.327	0.949	0.48	1.314

Note: Safeguard Building Peak "g" values are applicable to the Diesel Generator Area of that building.

3. Electrical Building

Floor Elevation	Peak "g" Value			
	OBE 4%		SSE 7%	
(Ft)	H	V	H	V
873.33	1.79	1.32	1.85	1.77
854.33	1.57	1.31	1.62	1.77
830.00	1.11	1.22	1.16	1.65
807.0	0.72	1.26	0.87	1.70
778.0	0.51	1.26	0.63	1.69

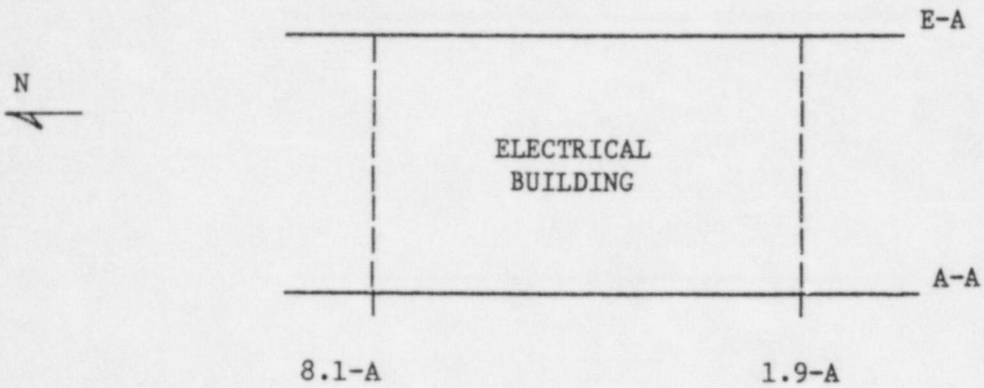
NOTE: See sheet 4.1 of Appendix 1 for clarification of column lines defining the Electrical Building.

4. Auxiliary Building

Floor Elevation (Ft)	Peak "g" Value			
	OBE 4%		SSE 7%	
	H	V	H	V
899.50	2.66	1.71	2.72	2.11
886.50	2.32	1.63	2.36	2.16
873.50	1.98	1.66	2.02	2.22
852.50	1.66	1.64	1.72	2.13
831.50	1.22	1.58	1.36	2.02
810.50	0.71	1.48	0.82	1.88
790.50	0.53	1.34	0.68	1.84

Note: See sheet 4.1 of Appendix 1 for clarification of column lines defining the Electrical Building.

The Electrical Building response spectra shall be utilized for all CTHs located within the building area boundaries defined by column rows 8.1-A, 1.9-A, A-A, and E-A.



The specific building room numbers are as follows:

FLOOR ELEVATION	UNIT 2 ARCH ROOM NO.
778'-0	113
	115
	115B
792'-0(Part Plan)	118
	120
	122
	126
807'-0	134
840'-6	148A
	148C
854'-4	150
	150B
	151
	151B

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Note: Even though the "As-Built Drawings" identify these Room Nos. as Auxiliary Building, review of the Architectural drawings indicates that these rooms are physically located in the Electrical Building.

5. Fuel Handling Building

Floor Elevation (Ft)	Peak "g" Value			
	OBE 4%		SSE 7%	
	H	V	H	V
918.0	2.30	0.98	2.70	1.29
899.50	2.02	1.02	2.47	1.34
860.0	1.14	0.94	1.46	1.26
841.0	0.97	0.88	1.24	1.19
825.0	0.84	0.84	1.08	1.17
810.50	0.72	0.76	0.92	1.12

6. Containment Bldg

Floor Elevation	Peak "g" Value			
	OBE 4%		SSE 7%	
(Ft)	H	V	H	V
1000.50	2.39	2.23	2.49	2.51
950.58	1.85	1.76	1.98	2.09
905.75	1.36	1.39	1.52	1.83
860.00	0.87	1.28	1.05	1.58
805.50	0.59	1.14	0.72	1.42
783.58	0.53	1.06	0.65	1.31

APPENDIX 2

STRUCTURAL EMBEDMENTS

Appendices from Specification No. 2323-SS-30 Rev 2

Notes:

1. This is a Gibbs & Hill document incorporated in the Design Criteria without any changes except that typographical errors in Appendix 2, page 4 of 9 were corrected. R6

2. When a redline drawing does not identify the bolt/threaded rod material in a Richmond Insert, A-36 material shall be assumed in the cable tray hanger design verification.

TEXAS UTILITIES SERVICES INC.
AGENT FOR
TEXAS UTILITIES GENERATING COMPANY
ACTING FOR
DALLAS POWER & LIGHT COMPANY
TEXAS ELECTRIC SERVICE COMPANY
TEXAS POWER AND LIGHT COMPANY

COMANCHE PEAK STEAM ELECTRIC STATION
UNITS NO. 1 & 2

STRUCTURAL EMBEDMENTS
SPECIFICATION NO. 2323-SS-30
REVISION 1 - FEBRUARY 10, 1984
REVISION 2 - JUNE 13, 1986

6/16/86

P. E. DATE

GIBBS & HILL, INC.
ENGINEERS, DESIGNERS, CONSTRUCTORS
NEW YORK, NEW YORK

STRUCTURAL EMBEDMENTS

CONTENTS

The following DCA's have been incorporated into Revision 1 of Specification 2323-SS-30 as follows:

DCA No.	Rev. No.	Section No.
12411	0	Appendix 4
13194	0	Appendix 3
13215	0	Appendix 4
15338	1	Appendix 6
15883	0	Sect. 2.5

STRUCTURAL EMBEDMENTS

1.0 SCOPE

1.1 DRILLED-IN EXPANSION BOLTS

This Specification covers the design criteria for the use of drilled-in expansion bolts and the requirements for furnishing all equipment, labor and materials necessary for the installation of drilled-in expansion bolts in existing structural concrete. The drilled-in expansion bolts shall be Hilti Kwik-Bolt and Super Kwik-Bolt Anchors (including nuts and washers) as furnished by Hilti Fastening Systems.

1.2 SCREW ANCHORS AND EMBEDDED PLATES

This Specification covers the design criteria for the use of screw anchors and steel plates embedded in concrete to which miscellaneous hangers and other structural supports are attached. Screw anchors are Richmond Structural Connection Inserts as furnished by Richmond Screw Anchor Co., Inc. Installation of screw anchors and fabrication and installation of embedded plates are as shown on the engineering drawings.

2.0 INSTALLATION OF HILTI EXPANSION BOLTS

2.1 GENERAL REQUIREMENTS

The expansion bolts shall be installed in strict accordance with the installation instructions and procedures as developed and recommended by Hilti Fastening Systems and the requirements of this Specification. Where Hilti requirements conflict with requirements of this Specification, the Specification shall govern.

2.2 EXPANSION BOLT SPACING

Unless otherwise specified on design documents, expansion anchors shall not be spaced closer than 10 anchor diameters. The minimum anchor spacing between two (2) unequal sized bolts shall be the sum of (5) respective bolt diameters as shown in Attachment 1 of Appendix 1 of this Specification. For expansion bolt spacing less than that required by Attachments 1, 2 and 3 to Appendix 1 of this Specification Engineering approval shall be obtained prior to installation of the expansion bolt.

2.3 INTERFERENCE WITH STRUCTURAL REINFORCING STEEL

Where interference between the expansion bolt and reinforcement is encountered, the bolt location shall be adjusted within tolerances as noted on design drawings to avoid such interference. In no case shall reinforcement steel be cut without prior approval of the Engineer.

2.4 CUTTING STRUCTURAL REINFORCING STEEL

Rebar cutting procedure, where permitted by the Engineer, shall be in accordance with CEI-20 (Appendix 1 of this Specification).

2.5 SETTING EXPANSION BOLTS

Expansion bolts shall be set by tightening the nut to the required torque value as given in CEI-20 (Appendix 1 of this Specification.) These torques are the minimum values required to obtain, without slippage, a minimum static tensile test load capacity of 115 percent of the allowable tensile working load given in Tables 1 and 2 of Appendix 2 of this Specification for a factor of safety of 5.

Torque values for other allowable tensile working loads shall be established by on-site testing

2.5.1 Setting (torque) verification of expansion anchors, if not at time of installation of the expansion anchor, shall be as follows:

Setting verification shall be by application of the torque as specified in 3.1.4.1 of CEI-20 (Appendix 1) during the verification process. Nut may turn additionally due to the initial relaxation. Torque must be obtained prior to nut bottoming out in the threads. Frequency of verification shall be per applicable site QA/QC procedures and instructions.

3.0 INSPECTION

3.1 INSPECTION OF EXPANSION BOLTS

All installed expansion bolts shall be visually inspected for proper size, embedment length, and thread projection above top of nut, and for possible cracks, distortions and damaged concrete.

4.0 REPAIR OF EXPANSION BOLT FAILURES

All expansion bolts that, during installation or after inspection fail to meet the requirements of this Specification shall be repaired as follows by the Contractor, unless otherwise directed by the Engineer.

4.1 EXPANSION BOLT SLIPPAGE, LOOSENING, PULLOUT OR FAILURE (RUPTURE, DISTORTION, DEFORMATION)

4.1.1 For expansion bolts that slip, loosen, pull out, or fail, using appropriate equipment, the existing anchor bolt hole shall be redrilled in accordance with Appendix 1 of this Specification.

4.1.2 For cases in which the bolt can not be removed, the bolt shall be cut flush with the concrete surface driven back into the hole and the surface of the concrete patched as required by this Specification.

4.2 CONCRETE SHEAR CONE FAILURE

For concrete shear cone failure, using appropriate equipment, the existing anchor bolt hole shall be redrilled so that the new embedment depth is 4-1/2 anchor diameters for Kwik bolts and 6-1/2 anchor diameters for Super Kwik bolts greater than the previous embedded depth. As an alternate the expansion bolt may be relocated, however the damaged concrete shall still be repaired.

5.0 REPAIR OF DAMAGED CONCRETE

Repair of damaged concrete shall be in accordance with Specification 2323-SS-9 and Appendix 1 of this Specification.

6.0 DESIGN

6.1 DESIGN CRITERIA FOR EXPANSION BOLTS

Design criteria for use of Hilti Kwik- and Hilti Super Kwik-Bolts are provided in Appendix 2 of this Specification.

6.2 DESIGN CRITERIA FOR SCREW ANCHORS

Design criteria for use of Richmond structural connection inserts are provided in Appendix 3 of this Specification.

6.3 DESIGN CRITERIA FOR EMBEDDED STEEL PLATE STRIPS

6.3.1 Design criteria for the use of embedded steel plate strips are provided in Appendix 4 of this Specification.

6.3.2 Alternative design criteria for the use of embedded steel plate strips are provided in Appendix 4W of this Specification. Appendix 4W is a Westinghouse generated document. The design methodology, assumptions, procedures and summary of results are provided in Westinghouse document WCAP 10923 dated 8/30/85. REV 2

6.4 DESIGN CRITERIA FOR EMBEDDED LARGE STEEL PLATES

6.4.1 Design criteria for the use of embedded large steel plates are provided in Appendix 5 of this Specification.

6.4.2 Alternative design criteria for the use of embedded large steel plates are provided in Appendix 5W of this Specification. Appendix 5W is a Westinghouse generated document. The design methodology, assumptions, procedures and results are provided in Westinghouse document WCAP 10923 dated 8/30/85. REV 2

7.0 QUALITY ASSURANCE

7.1 SCREW ANCHORS AND EMBEDDED PLATES

Quality assurance requirements for use of Richmond structural connection inserts and embedded plates shall be in accordance with site engineering procedures. REV 2

7.2 DRILLED-IN EXPANSION BOLTS

7.2.1 MANUFACTURER'S REQUIREMENTS

Hilti Kwik-Bolts Super Kwik-Bolts shall be supplied by the manufacturer with a certification of compliance signed and dated by a responsible person within the manufacturer's organization. This certification shall state that the Hilti Kwik-Bolts and Super Kwik-Bolts furnished under the purchase order are manufactured in accordance with Hilti Catalog Supplement #H-390B, dated 4/77. In addition, the certifiation shall state the grade of material used, part numbers, and number of each part number covered by the certification.

All materials furnished may be subject to confirmatory testing by the Contractor to assure that the quality of the material is consistent with the specifications listed in the above mentioned catalog.

7.2.2 INSTALLATION REQUIREMENTS

Quality Assurance Installation Requirements shall be in accordance with Appendix 1 of this Specification.

7.2.3 DESIGN

Quality assurance requirements for use of drilled-in expansion bolts shall be in accordance with site engineering procedures. REV 2

SS-30

APPENDIX 1

INSTALLATION OF "HILTI"
DRILLED-IN BOLTS
CEI-20 REVISION 9

BROWN & ROOT, INC. CPSES	PROCEDURE NUMBER	REVISION	EFFECTIVE DATE	PAGE
	CEI-20	9	12/16/83	1 of 12

TITLE: INSTALLATION OF "HILTI" DRILLED-IN BOLTS	
ORIGINATOR	[signature] 12-16-83 Date
REVIEWED BY:	N/A [signature] B&R QA 12-16-83 Date
	[signature] TUGCO QA 12/16/83 Date
APPROVED BY:	[signature] CONSTRUCTION PROJECT MGR 12-16-83 Date

o.i TABLE OF CONTENTS

0.ii ATTACHMENTS

1.0 REFERENCES

1.1 B&R Construction Procedure 35-1195-CCP-12, Concrete Patching, Finishing and Preparation of Construction Joints"

1.2 DM-13966, "Hilti Kwik-Bolt Testing Program".

1.3 TUF-4593, (May 22, 1978)

1.4 B&R Quality Assurance Procedure CP-QAP-16.1, "Control of Nonconforming Items".

1.5 TUSI Procedure No. CPP-EP-1, "Procedure for Preparation of Design Changes".

1.6 35-1195-IEI-13, B&R Instruction "Calibration of Micrometer Torque Wrenches".

1.7 CP-QP-11.2, TUGCO Procedure, "Surveillance and Inspection of Concrete Anchor Bolt Installations".

1.8 QI-QP-11.2-1, TUGCO Instruction, "Concrete Anchor Bolt Installation".

1.9 QI-QP-11.2-3, TUGCO Instruction, "Torquing of Concrete Anchor Bolts".

1.10 QI-QP-11.2-4, TUGCO Instruction, "Inspection of "Hilti" Super Kwik Bolts".

2.0 GENERAL

2.1 PURPOSE

2.1.1 The purpose of this instruction is to describe the methods to be followed in the field installation of Hilti drilled-in expansion anchors.

2.2 SCOPE

2.2.1 This instruction covers the location, repair and preparation of expansion bolt holes, installation of the expansion bolts, and the permanent marking of bolts for identification both prior to and after their installation. The provisions of this instruction apply to both Hilti Kwik-Bolts and Hilti Super Kwik-Bolts that are used for installation of safety related equipment, and for the installation of non-safety related equipment located in safety related structures. Deviations from this instruction are permitted provided they are properly approved by the Engineer. Post nut caps may not be substituted for hex head nuts without prior Engineering approval.

2.3 RESPONSIBILITY

2.3.1 Establishment of control points and lines for use in layout of bolt locations shall be the responsibility of the B&R Field Engineering Superintendent. Determination and marking of bolt hole location shall be performed by the craft which prepares the holes and installs the bolts; and the superintendent of that craft shall be responsible for this layout work and for preparation of holes and bolt installation.

2.4 DEFINITIONS

2.4.1 "Drilled-in Expansion Bolts" are bolts having expansion wedges so arranged that, when placed in a drilled hole and the nut tightened, the wedges are expanded and the bolt is securely anchored, all as manufactured by Hilti Fastening Systems, Inc.

2.4.2 "Hilti" is Hilti Fastening Systems, Inc., supplier of the expansion bolts.

2.4.3 "Bolt Length" is the total overall length of the bolt. This is the length dimension shown in the Bill of Material on the appropriate drawings.

2.4.4 "Setting" a bolt means positioning the bolt and tigntening the nut or post nut to the extent required to complete the expansion of the wedges.

2.4.5 "Embedment Length" is the length of bolt extending below the surface of the 4000 psi (28-day strength) structural concrete prior to setting (tightening). Where not shown on the pipe/instrument support design drawings, the minimum embedment length shall be as follows:

BOLT DIAMETER	MINIMUM EMBEDMENT	
	Kwik-Bolts	Super Kwik-Bolts
1/4	1 1/8	—
3/8	1 5/8	—
1/2	2 1/4	3 1/4
5/8	2 3/4	—
3/4	3 1/4	—
1	4 1/2	6 1/2
1 1/4	5 1/2	8 1/8

Dimensions are in inches, they are according to recommendations by Hilti and correspond to the minimums shown in Abbot A. Hanks, Inc. Test Report No. 8783R on Kwik-Bolts and Test Report No. 8786 on Super Kwik-Bolts, as published in Hilti "Architects and Engineers Anchor and Fastener Design Manual.

The above minimum embedment lengths are into structural concrete. On floors where 2-inch thick concrete topping (and thicker on roof slabs built up to slope to drain) has been placed separately, bolts shall be of sufficient length to provide embedment length or overall length at least equal to the thickness of the topping in addition to the length shown on the drawings. For floor mounted pipe supports only, the engineer shall evaluate and approve the support for sufficient embedment length on a case-by-case basis. The areas where this topping occurs are shown on the following drawings:

Drawing No.	Sheet No.	Building
FSC-00421	1	Fuel
FSC-00421	2	Fuel
FSC-00422	1	Reactor #1
FSC-00422	2	Reactor #1
FSC-00422	3	Reactor #1
FSC-00422	4	Reactor #1
FSC-00422	5	Reactor #1
FSC-00423	1	Auxiliary
FSC-00423	2	Auxiliary
FSC-00423	3	Auxiliary
FSC-00423	4	Auxiliary
FSC-00423	5	Auxiliary
FSC-00423	6	Auxiliary
FSC-00423	7	Auxiliary
FSC-00423	8	Auxiliary
FSC-00423	9	Auxiliary
FSC-00424	1	Safeguard #1
FSC-00424	2	Safeguard #1
FSC-00424	3	Safeguard #1
FSC-00424	4	Safeguard #1
FSC-00424	5	Safeguard #1
FSC-00424	6	Safeguard #1
FSC-00426	1	Service Water Intake
FSC-00425	1	Safeguard #2
FSC-00425	2	Safeguard #2
FSC-00425	3	Safeguard #2

3.0 PROCEDURE

3.1 INSTALLATION

3.1.1 Locating Bolts

3.1.1.1 As required by authorized engineering documents, bolt locations shall be determined by the installing craftsmen using the control points and lines established by the Field Engineering Department; and, as an aid in locations where reinforcing steel integrity is considered to be critical, utilization of reinforcing steel placement drawings and suitable reinforcement detection equipment may be used. The minimum spacing and/or clearance for expansion bolts shall be provided as indicated in Attachments 1, 2 and 3 unless specifically approved otherwise by the Engineer using appropriate design documents.

3.1.2 Drilled Holes

3.1.2.1 Expansion bolt holes shall not be drilled into structural reinforcing steel unless approved by the design engineer or his representative. Holes for the expansion bolts shall be drilled into concrete by the use of suitable power drills using "hilti" carbide masonry bits of the same nominal size as the bolt and which are designed and recommended by the Hilti Corp. specifically for this purpose, or an approved equal. The holes shall be drilled to depths at least one-half (½) inch greater than the embedment length of the bolt. This is in order that any accessible/usable abandoned bolt can be cut off and driven deeper into the hole and top covered with grout or other suitable filler to close the hole. Abandoned bolts that are not usable or accessible may be left in place without further rework or approval.

3.1.2.2 Holes shall normally be drilled as near the perpendicular to the concrete surface as feasible. In no case shall the long axis of installed bolts be more than 6° from this perpendicular direction. Excess dust should be cleaned from the hole after drilling.

3.1.2.3 Where cutting of structural reinforcing steel is permitted by the Engineer, Drillco water cooled carbide/diamond bits or equal shall be used. Once the structural reinforcing steel is cut, the remainder of the hole shall be drilled with a "hilti" carbide masonry bit per 3.1.2.1. Both bits shall be of the same nominal diameter as the bolt to be installed.

3.1.2.4 In limited access areas it may be difficult to drill holes for expansion bolts using equipment as required by 3.1.2.1. For this situation, a flexible drive drill with drill press/vacuum base and Drillco water cooled carbide/diamond bit or approved equal may be used. Caution shall be used when drilling to avoid the cutting of structural reinforcing steel. In no case shall structural reinforcing steel be cut without prior approval of the Engineer.

3.1.3 Marking Bolts

3.1.3.1 The threaded end of bolts shall bear permanent markings which indicate the bolt length.

3.1.3.2 These markings shall be made by the manufacturer by die-stamping a letter or a number on the top end of the bolt. This stamping shall indicate the bolt length in accordance with the "Length Identification System" (Attachment 4). Bolts may also be marked on-site by the same system if verified and documented by B&R QC. For Post Nut Series Hilti Bolts, the letter or number designation shall correspond to the overall length of the assembly with the Post Nut Cap completely installed (threads bottomed out).

3.1.3.3 Hilti Super Kwik Bolts shall be additionally marked with a "star" on the end which will remain exposed upon installation. This marking will be performed by the craft in a manner which does not obliterate the length marking. The stamp shall be controlled by the cognizant QC Inspector.

3.1.4 Setting Bolts

3.1.4.1 In no case shall bolts be set in concrete having strength less than the 28-day old design strength. Inserting bolts may be accomplished either by use of a mandrel or double nuts. In using double nuts, they shall be placed on the bolt so as to protect the bolt end and threads. The bolt shall be driven into the hole the embedment length by blows on the mandrel or nut. Projection of the bolt should be such that, after final tightening, the end of the bolt is not lower than flush with the top of the nut. Its projection above the top of the nut is not limited although its change in projection during tightening shall be within the limit specified below. The mandrel, if used, is then replaced by a nut, or the top double nut is removed and the bolt is "set". The setting will be accomplished by tightening the nut against the fixture being installed. At that time, the nut will be drawn down and the bolt pulled to set the wedges by the use of a torque wrench, attaining at least the respective final values shown in the following table unless otherwise shown on the drawings. During tightening the nut, the change in bolt projection shall not exceed one nut height unless otherwise approved by the engineer. Where 5/8" diameter bolts are used in erecting Uni-Strut members for instrument or conduit supports in such a way that the bearing surface under the nut, used with a flat washer, bears against the open side of the Uni-Strut, the nut shall be tightened to 80-foot-pounds torque.

BOLT SIZE	TORQUE (Ft.-Lbs) (Hilti Kwik or Super Kwik, all embedment depths)
1/4	8
3/8	17
1/2	70
5/8	120
3/4	150
1	230
1 1/4	400

These values were determined by field tests conducted by Hilti at the CPSES site which yielded a minimum static tensil load capacity equal to or greater than 115% of the tensile working loads given in Tables 1 and 2 of Appendix 2 of Specification 2323-SS-30.
The complete report on those tests is filed in the B&R QC Department. (Ref. CPPA-7240 or B&R IM-13966).

Bolts which cannot be torqued to the above minimum values shall be cut off, driven deeper into the hole, and patched per Reference 1.1 or shall be removed and replaced in accordance with 3.1.4.2 below. Torque wrenches used in this operation shall be calibrated and periodically recalibrated in accordance with Engineering Instruction 35-1195-IEI-13, "Calibration of Micrometer Torque Wrenches", Reference 1.6.

3.1.4.1.1 For post nut series Hilti bolts, setting the bolts shall be done in accordance with Section 3.1.4 with the following exceptions applying to Section 3.1.4.1.

Inserting bolts may be accomplished by the use of a post nut, placed on the bolt so as to protect the bolt end and threads. The bolt shall be driven into the hole the embedment length by blows on the post nut. Projection of the bolt should be such that, after final tightening, the end of the bolt has a minimum thread engagement of 3/16" for 1/4" dia. and 5/16" for 3/8" dia. bolts. The projection should also be limited such that, after final tightening, the threads on the post nut have not bottomed out on the bolt. The post nut used to insert the bolt should then be removed and the bolt is "set". The setting will be accomplished by tightening a new post nut against the fixture being installed. At that time, the nut will be drawn down and the bolt pulled to set the wedges by the use of a torque wrench and 3/8" drive screwdriver adapter attaining at least the respective final values shown in the above table unless otherwise shown on the drawings. During tightening the post nut, the change in bolt projection shall not exceed ½" for a ½" dia. and 3/8" for 3/8" dia. bolts, unless otherwise approved by the engineer.

3.1.4.2 Replacement of expansion bolts that slip, loosen, pull out or fail to achieve the specified torque may be accomplished by one of the following methods:

3.1.4.2.1 The bolt shall be removed and replaced with a bolt that has an embedment depth increased by at least 4½ bolt diameters for Hilti Kwik-Bolts and 6½ bolt diameters for Hilti Super Kwik-Bolts unless otherwise directed by the Engineer. QC shall be notified prior to commencing work.

3.1.4.2.2 The re-installation of an expansion bolt in an empty but "pre-used" hole is acceptable provided the following requirements are met:

a. The existing hole has not experienced structural damage as may be exhibited if the previous bolt had been displaced through tension or shear causing severe concrete spalling. Severe concrete spalling are depths that exceed the dimensions provided in 3.1.5.1 below.

b. New "Replacement" expansion anchors are at least one diameter size larger.

c. New embedment depth is equal to or greater than the previous bolt but in no case less than the minimum embedment required per 2.4.5 above based on the "replacement" bolt size.

d. Bolts that cannot be replaced per the above may be replaced by a bolt meeting the requirements of 3.1.4.2.1 or may be cut off, driven into the hole and patched per Reference 1.1.

e. QC shall be notified prior to commencing work and after the bolt has been removed so that QC may inspect the "pre-used" hole in accordance with the applicable QC procedures.

f. QC shall be notified prior to commencing work.

3.1.4.2.3 The re-installation of an expansion bolt in an empty but "pre-used" hole is acceptable provided the following requirements are met.

a. The bolt being replaced has been removed from the concrete using a Diamond core bit of the same nominal outside diameter as the replacement expansion bolt. The replacement bolt shall be one diameter size larger than the bolt being removed.

b. The existing hole after bolt removal should not show evidence of structural change as in the form of severe concrete spalling. Severe concrete spalling are depths that exceed the dimensions provided in 3.1.5.1 below.

c. New embedment depth is equal to or greater than the previous bolt but in no case less than the minimum embedment required per 2.4.5 above based on the "replacement" bolt size.

d. Bolts that cannot be replaced per this method may be replaced by a bolt meeting the requirements of 3.1.4.2.1, 3.1.4.2.2 or may be cut off, driven into the hole and patched per Reference 1.1.

e. QC shall be notified prior to commencing work, and after the bolt has been removed so that QC may inspect the "pre-used" hole in accordance with the applicable QC procedures.

3.1.5 Repair of Broken Concrete and Abandoned Holes

3.1.5.1 Structural concrete that is broken or spalled as a result of bolt installation but is structurally sound shall be cleaned up and may be cosmetically repaired either in accordance with Construction Procedure CCP-12, or by the use of "NUTEC" #11S as manufactured by and according to the recommendations of Southern Imperial Coating, Inc. Spalling of structural concrete to depths greater than those listed below shall be cause for rejection of the hole and redrilling will be necessary.

Hole Size	Max. Acceptable Spall Depth
5/8" and under	1/2"
3/4" to 1 1/4" (incl.)	3/4"

Spalling of the 2" topping in areas described in Section 2.4.5 shall be cleaned up and repaired in accordance with Construction Procedure CCP-12 using material described in Section 4.1.2.7 of CCP-12. Maximum spall depth is not to exceed depth of topping.

3.1.5.2 Abandoned holes shall be filled and patched prior to coating the concrete. This repair shall be in accordance with provisions of B&R Construction Procedure 35-1195-CCP-12 for filling "Tie Holes" by the use of patching mortar prepared as described in paragraph 4.1.1.3 of that procedure. However, abandoned OVERHEAD holes, orginally drilled for Hilti expansion bolts, which will be completely covered by the base plates or angles of attached fixtures and which are farther than four bolt diameters (center-to-center) from an active Hilti bolt, may be filled with "Silpruf" waterproofing sealant or "GE 1300", both as manufactured by General Electric, Inc. Holes located at a distance of four bolt diameters and closer, measured center-to-center, from Hilti bolts shall be filled and patched according to Procedure 35-1195-CCP-12 described above prior to torquing.

3.1.5.3 Unused Richmond Screw Anchors which have been plugged by Richmond screw-in plugs may be used for permanent anchorage only after specific approval by the Engineer.

3.1.6 <u>Modification</u>

3.1.6.1 When it is necessary, as the result of reinforcing steel interference or on-site unavailability of correct lenght bolts or for other reason, Hilti bolts may be modified, with proper QC witnessing, on-site shortening, rethreading, and stamping the new length designation. This shall be done only on a case by case basis upon approval of the design engineer responsible for the fixture or item involved and upon completion of appropriate permanent plant documentation (i.e., DCA, CMC, FSE, Operational Traveler, Design Drawing, etc.) by the design engineer. Final bolt length shall be sufficient to satisfy the design requirement.

3.1.6.2 Substitution of a Hilti bolt of the next larger size is acceptable, provided all spacing and embedment requirements are met or exceeded for size Hilti bolt substituted.

3.1.7 <u>Rework of Bolts in 2-inch Concrete Topping Areas</u>

3.1.7.1 For areas in which the requirements of Section 2.4.5 cannot be met, the following action shall be taken:

3.1.7.1.1 Expansion bolts which after setting have less than below indicated embedment length into the structural concrete shall be reworked by one of the methods provided in section 3.1.4.2 or as follows:

Bolt Type	Embedment After Setting
Kwik-Bolts	3½ bolt diameters
Super Kwik-Bolts	5½ bolt diameters

a. Existing Location

1. Bolt removal - The removal of in-place expansion bolts shall be completed with care so as not to damage the concrete, thereby impairing its integrity. A hollow core hydraulic ram placed directly over an appropriately sized baseplate which is centered on the bolt may be used to apply direct tension to pull the bolt through the expansion wedges. The baseplate should be a ½ inch thick square plate of a minimum of 16 expansion bolt diameters in width, bearing directly against the concrete surface.

2. Once the bolt is removed, use a high speed drill and bit to drill through the wedges remaining in the side of the hole. Remove any loose wedges in the hole.

3. Using appropriate equipment, re-drill existing expansion bolt hole so that the new embedment depth is a minimum of 4½ bolt diameters for Hilti Kwik-Bolts greater than the previous existing embedment depth or to the specified embedment depth, whichever is greater unless otherwise directed by the Engineer by appropriate design documents.

4. Reinstall the appropriate sized expansion anchor to meet the required embedment length.

b. Relocation - Abandon existing expansion anchor bolts and relocate support structure. Abandoned bolts should be cut off, driven deeper into the hole, and patched per Reference 1.1.

3.1.7.1.2 Expansion bolts which have less than the specified designed embedment length into structural concrete but greater than the values indicated above in 3.1.7.1.1 shall be evaluated by the responsible design engineer. If found to be acceptable "as-is", appropriate design change documents shall be issued. If found to be unacceptable, the expansion bolt shall be reworked in accordance with 3.1.7.1.1 a or b.

3.2 INSPECTION

3.2.1 Inspection of Hilti bolt installation shall be performed in accordance with References 1.6, 1.7, 1.8, 1.9, and 1.10 and other applicable site QA/QC procedures and instructions.

3.2.2 Removal of an inspected Hilti bolt shall be documented on an IRN in accordance with CP-CPM 6.10. Removal and replacement of non-Q Hilti bolts in Catagory I structures shall be documented on an IRN and submitted to QC for subsequent processing.

Note: An IRN is not required if a non-Q Hilti is only going to be removed and not replaced.

ATTACHMENT 1

* MINIMUM SPACING BETWEEN HILTI EXPANSION BOLTS

Hilti Bolt Size	CENTER TO CENTER SPACING TO: 1/4"Hilti	3/8"Hilti	1/2"Hilti	5/8"Hilti	3/4"Hilti	1"Hilti	1 1/4"Hilti
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

Dimensions in inches.

* The minimum spacing outlined in the above chart applies to Hilti bolts detailed on separate adjacent fixtures. Violation of minimum spacing by the installation of two separate adjacent fixtures will be approved only by issuance of an Engineering Evaluation of Separation Violation Form by the CPPE design groups (Ref. CP-EP-4.3).

Hilti bolts detailed on an individual fixture drawing may have less than the minimum spacing tabulated above. Such fixtures have been derated by engineering justification and are the responsibility of the organization issuing the respective fixture drawing. Installation in this case shall proceed in accordance with the fixture drawing.

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ATTACHMENT 2

MINIMUM BOLT CLEARANCES *
(INCHES)

Hilti Bolt Size	MINIMUM DISTANCE TO: Richmond Screw Anchors* 1-inch	Richmond Screw Anchors* 1¼-inch	Concrete Edge*	Abandoned Hilti Bolts or Holes and Embedded Anchor Bolts that are Cut Off**
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.)

Locations closer than the above distances shall be used only upon approval of the engineer.

Hilti bolts may be installed as close as practical to unused Richmond Screw Anchors which have been plugged (i.e., grouted, Richmond Screw-in plug or snap-in plug, etc.).

Unused Richmond Screw Anchors located nearer to Hilti bolts than the respective distances shown above may be used temporarily for construction purposes when the applied load is:

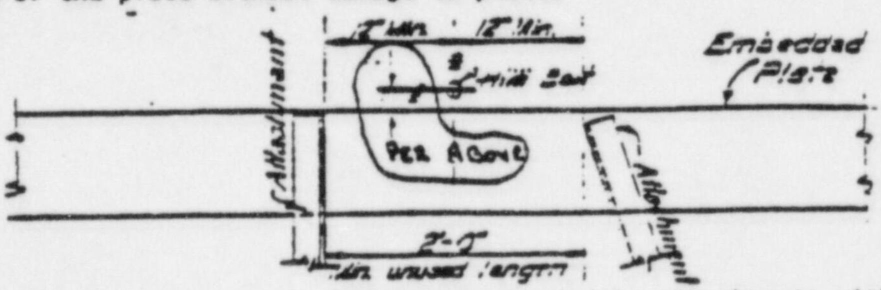
(a) For 1" Richmond Anchors, less than 8,000 pounds minus the actual load supported by the Hilti bolt; or

(b) For 1¼" Richmond Anchors, less than 20,000 pounds minus the actual load supported by the Hilti bolt.

ATTACHMENT 3

MINIMUM CLEARANCES TO EMBEDDED PLATES

1. Where embedded steel plates are unoccupied by attachments for a minimum distance of 12 inches on both sides of a proposed Hilti Bolt location as shown below, 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

Dimensions are in inches.
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.

ATTACHMENT 4

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
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ATTACHMENT 4 (cont'd)

LENGTH IDENTIFICATION SYSTEM

Stamp On Anchor	Length of Anchor (Inches) From	Up to (Not including)
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
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.

SS-30
APPENDIX 2

DESIGN CRITERIA FOR
HILTI KWIK- AND SUPER
KWIK-BOLTS

APPENDIX 2

DESIGN CRITERIA FOR HILTI KWIK- AND SUPER KWIK-BOLTS

1.0 REFERENCES

1.1 "Architects and Engineers Anchor and Fastener Design Manual" by Hilti Fastening Systems, 3.6/Hi-1, No. H-427A 10/78.

1.2 TUSI correspondence CPPA-7419 - Reduced Design Allowables for 1" diameter Hilti Kwik-Bolts, dated, 11-18-80.

2.0 MINIMUM SEPARATION REQUIREMENTS

2.1 To attain the design capacity of a Hilti Kwik or Super Kwik bolt for a specified embedment the minimum spacings provided by Appendix 1 of this Specification must be maintained.

2.2 For installations not conforming to the provisions of Section 2.1 above, the capacity of both anchors shall be reduced on a straight-line basis to 50 percent at half the minimum distance between embedments given in Appendix 1 of this Specification. In no case shall embedments be spaced closer than half this minimum distance.

Methods for evaluation of this reduced capacity are given at the end of this Appendix and are controlled by concrete stresses.

3.0 DESIGN ALLOWABLE LOADS

3.1 Design allowable tensile and shear loads are provided in Tables 1 and 2. These design allowables are based on the average ultimate tensile and shear loads published in Reference 1.1 and 1.2 of this Appendix . Factor of safety of less than 4 is not acceptable.

3.2 Prior to the utilization of the allowable tensile loads in Tables 1 and 2 of this Appendix (except for the 1-inch diameter Kwik-bolts) the manufacturer shall certify the validity of the ultimate capacities of the Kwik and Super Kwik bolts as published in reference 1.1 of this Appendix.

4.0 COMBINED LOADING

When the Hilti expansion anchor is subjected to a combination of tension and shear loading the following interaction requirement shall be met:

$$\frac{T}{T^1} + \frac{S}{S^1} \leq 1$$

T = Actual applied tension load
T^1 = Allowable design tension load
S = Actual applied shear load
S^1 = Allowable design shear load

5.0 REQUIRED EMBEDMENT

For the required minimum anchor embedments see Appendix 1 of this Specification.

TABLE 1

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

FACTOR OF SAFETY		FS=4.0		FS=5.0	
DIAMETER	EMBEDMENT	TENSION	SHEAR	TENSION	SHEAR
1/4	1 1/8"	364	653	291	522
	1 1/2"	556	653	445	522
	1 3/4"	675	653	540	522
	2"	781	653	625	522
	2 1/4"	827	653	662	522
	2 1/2"	837	653	670	522
3/8"	1 5/8"	588	1276	471	1021
	2"	756	1276	605	1021
	2 1/2"	975	1276	780	1021
	3"	1075	1354	860	1083
	3 1/2"	1150	1354	920	1083
	4"	1187	1354	950	1083
	4 1/2"	1200	1354	960	1083
1/2"	2 1/4"	1377	2079	1102	1663
	2 3/4"	1800	2079	1440	1663
	3 1/2"	2362	2079	1890	1663
	4 1/2"	2806	2558	2245	2046
	5 1/2"	3012	2558	2410	2046
	6"	3075	2558	2460	2046
5/8"	2 3/4"	1650	2890	1320	2312 R6
	3 1/2"	2275	2890	1820	2312
	4 1/2"	3000	2890	2400	2312
	5 1/2"	3575	3859	2860	3087 R6
	6 1/2"	4000	3859	3200	3087
	7 1/2"	4250	3859	3400	3087
3/4"	3 1/4"	2537	4283	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

TABLE 1 (Cont'd)

FACTOR OF SAFETY		FS=4.0		FS=5.0	
DIAMETER	EMBEDMENT	TENSION	SHEAR	TENSION	SHEAR
	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 average ultimate tensile and shear loads published in "HILTI - Architects and Engineers Anchor and Fastener Design Manual" 3.6/Hi-1, Reference 1.1 and 1.2 of this Appendix.

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

**Values per Reference 1.2 of this Appendix.

TABLE 2

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

FACTOR OF SAFETY		FS=4.0		FS=5.0	
DIAMETER	EMBEDMENT	TENSION	SHEAR	TENSION	SHEAR
1/2"	3 1/4"	2496	2860	1997	2288
	4 1/4"	3695	2860	2956	2288
	5 1/4"	3641	2860	2913	2288
	6 1/4"	3786	2860	3029	2288
1"	6 1/2"	8741	6884	6993	5507
	8 1/2"	12452	6884	9962	5507
	10 1/2"	12439	6884	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 average ultimate tensile and shear loads published in "HILTI - Architects and Engineers Anchor and Fastener Design Manual" 3.6/Hi-1. Reference 1.1 of this Appendix.

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

EVALUATION METHOD I:

PROBLEM: Calculation of the reduced allowable capacities for Hilti expansion anchors spaced at less than minimum separation requirement indicated in Appendix 1 of this Specification.

EVALUATION:

STEP 1: Determine actual loading conditions on the Hilti expansion anchors in question.

STEP 2: Calculate the separation ratio. (S.R.)

Separation ratio is defined as the ratio of the separation provided to the minimum separation required by Appendix 1 of this Specification. This ratio must be equal to or greater than .500.

$$S.R. = \frac{\text{SEPARATION PROVIDED}}{\text{MINIMUM SEPARATION REQUIRED}} \quad (1)$$

$$\text{and } S.R. \geq .500 \quad (2)$$

STEP 3: Once the separation ratio is computed and actual loads are determined, the following relation shall be satisfied for acceptability of the anchor design.

$$\frac{T}{T^1(S.R.)} + \frac{S}{S^1(S.R.)} \leq 1 \quad (3)$$

WHERE: T = Actual Tension; S = Actual Shear; T^1 = Allowable Design Tension; S^1 = Allowable Design Shear; S.R. = separation ratio.

STEP 4: If the requirement of Formula (3) is satisfied, Hilti expansion anchors for the support in question are acceptable.

If the relationship in Formula (3) is not satisfied, Hilti expansion anchors are not acceptable and an appropriate action shall be taken by adjustment of separation to meet the requirement in Formula (3)

EVALUATION METHOD 2:

PROBLEM: Calculation of the reduced allowable capacities for both the Hilti expansion anchor and the Richmond screw anchor when minimum separation is not provided as required in Appendix 1 of this Specification.

EVALUATION:

STEP 1: Determine actual loading condition on the Hilti expansion anchor and/or Richmond screw anchor in question.

STEP 2: Calculate the separation ratio (S.R.).

$$S.R. = \frac{\text{SEPARATION PROVIDED}}{\text{MINIMUM SEPARATION REQUIRED}} \quad (4)$$

$$\text{and } S.R. \geq .500 \quad (5)$$

STEP 3: Once the separation ratio and the actual loads are computed, the following relations shall be satisfied for acceptability of the anchor and insert design:

For Hilti expansion anchor:

$$\frac{T}{T^1(S.R.)} \quad \frac{S}{S^1(S.R.)} \leq 1 \quad (6)$$

For Richmond insert:

$$\left[\frac{T}{T^1(S.R.)}\right]^{4/3} + \left[\frac{S}{S^1(S.R.)}\right]^{4/3} \leq 1 \quad (7)$$

For Richmond insert design allowable values see Appendix 3 of this Specification.

STEP 4: If the requirements of both Formula (6) and (7) are satisfied the Hilti expansion anchor and Richmond screw anchor for the support in question are acceptable.

If any of the relations in Formula (6) and (7) is not satisfied, the corresponding Hilti expansion anchor and Richmond screw anchor for the support in question are not acceptable, and an appropriate action shall be taken by adjustment of the separation to meet the requirements of Formulas (6) and (7).

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APPENDIX 3

DESIGN CRITERIA FOR
SCREW ANCHORS

APPENDIX 3

DESIGN CRITERIA FOR SCREW ANCHORS

1.0 GENERAL

1.1 Screw anchors are Richmond structural connection inserts (Types EC-2, EC-6, EC-2W or EC-6W) and are prefabricated steel anchors embedded in concrete to which structural supports are attached.

1.2 ASTM A325N A490 or A449 bolts (suitable washers optional) shall be used for the Richmond insert bolt connections. ASME SA-193 threaded rods with ASME SA-194 double nuts may be used for the Richmond insert bolt connections as a substitute for ASTM A325N bolts.

1.2.1 Thread engagement into the Richmond insert shall be at least 2 x bolt diameter + 1/8 inch

1.3 In no case shall these inserts be loaded before concrete attains its 28-day design strength.

2.0 APPLICABLE REFERENCES:

1. "Richmond Inserts for Concrete Constuction" Bulletin No. 6 Richmond Screw Anchor Co.,Inc., catalog.
2. Manual of Steel Construction AISC 7th Edition.

3.0 DESIGN CRITERIA

3.1 Design allowable tension and shear loads (under working stress condition) for respective center-to-center spacing of inserts and respective concrete thicknesses, are provided in the following Table 1.

3.2 Inserts and A307, A325, A490 or A449 bolts or A36 threaded rods subjected to combined tension and shear loads should satisfy the following interaction formulas.

FOR INSERTS:

$$* \left(\frac{T}{T^1}\right)^{4/3} + \left(\frac{S}{S^1}\right)^{4/3} \leq 1$$

FOR BOLTS: (Verified for specific type bolt materials.)

T^1; S^1 different for each grade.

$$\left(\frac{T}{T^1}\right)^{2} + \left(\frac{S}{S^1}\right)^{2} \leq 1$$

WHERE: T - APPLIED TENSION

S - APPLIED SHEAR

T^1 - DESIGN ALLOWABLE TENSION

S^1 - DESIGN ALLOWABLE SHEAR

3.3 Minimum distances between Richmond screw anchors and Hilti bolts for 100 percent performance of each are provided in Appendix 1 of this Specification.

For those situations where minimum distances cannot be met, evaluation method 2 shown in Appendix 2 of this Specification shall be used to calculate the reduced capacity of Richmond inserts.

* PCI MANUAL O! DESIGN OF CONNECTIONS FOR PRECAST PRESTRESSED CONCRETE

NOTES FOR TABLE I

1.- INSERT CAPACITIES ARE BASED ON INSERT EMBEDDED IN 4000 PSI COMPRESSION STRENGTH CONCRETE.

2.- ALL ALLOWABLE LOADS SHOWN IN TABLE I OF THIS APPENDIX ARE IN KIPS.

3.- TO DEVELOP THE FULL TENSION CAPACITY OF INSERT (EXCEPT AT BEAM SIDES) THE MINIMUM DISTANCE FROM CONCRETE EDGE TO CENTER OF INSERT SHALL BE 11" FOR 1½"φ INSERTS AND 7" FOR 1"φ INSERTS.

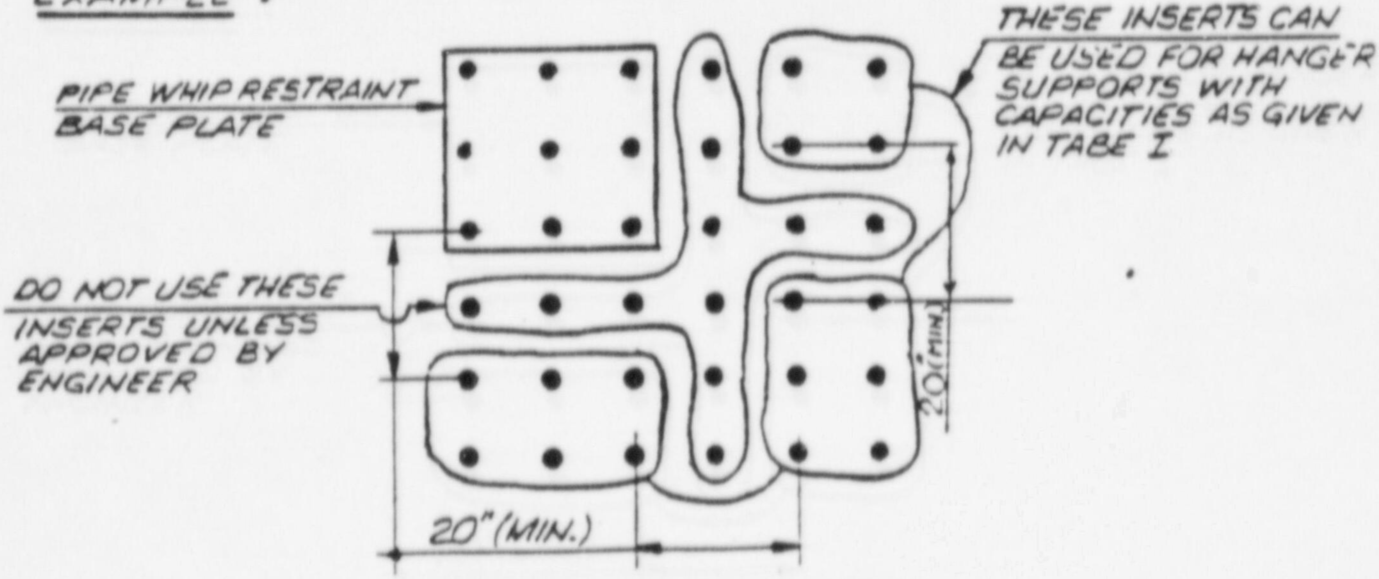
4.- TO DEVELOP THE FULL SHEAR CAPACITY OF INSERT (EXCEPT AT BEAM SIDES) THE MINIMUM DISTANCE FROM CONCRETE EDGE TO CENTER OF INSERT SHALL BE 14" FOR 1½"φ INSERTS AND 9.5" FOR 1"φ INSERTS.

5.- FOR BEAM SIDES THIS DISTANCE SHALL BE A MINIMUM 8" FOR 1½"φ INSERTS AND 7" FOR 1"φ INSERTS. (FOR TENSION AND SHEAR)

6.- CENTER TO CENTER (C/C) DISTANCES SHOWN IN TABLE I OF THIS APPENDIX ARE MINIMUM FOR THE ALLOWABLE LOADS.

7.- WHEN PART OF THE 1½"φ INSERT CLUSTER (INSERT CLUSTERS WERE ORIGINALLY PROVIDED FOR PIPE WHIP RESTRAINTS) IS USED FOR HANGER SUPPORTS THE OUTERMOST ROW OF INSERTS USED FOR THESE SUPPORTS SHALL BE AT LEAST 20" AWAY FROM THE NEAREST INSERTS USED FOR ANY OTHER SUPPORTS OR RESTRAINTS BASE PLATE.

EXAMPLE:



TYPICAL INSERT CLUSTER

NOTES FOR TABLE I (Contd.)

8.- TO FIND THE CAPACITIES OF INSERTS WHERE SPACING AND CONCRETE THICKNESS ARE NOT SHOWN IN TABLE I OF THIS APPENDIX USE THE INSERT CAPACITY OF THE NEAREST CORRESPONDING LOWER INSERT SPACING OR THINNER CONCRETE WALL, SLAB OR COLUMN INDICATED IN TABLE I OF THIS APPENDIX.

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" ϕ 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" ϕ 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 I

OBE ALLOWABLES

AND

SSE ALLOWABLES

R6

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

I: INSERT CAPACITY B: BOLT CAPACITY T: TENSION S: SHEAR

CONCRETE THICKNESS		INSERT SPACING ON 10"C/C BOTH WAYS				INSERT SPACING ON 12"C/C BOTH WAYS				INSERT SPACING ON 14"C/C OR MORE BOTH WAYS (FULL CAPACITY)			
		A-307 BOLTS OR A-36 THD. RODS USED W/INSERT		A-325 OR BETTER BOLTS USED W/INSERT		A-307 BOLTS OR A-36 THD. RODS USED W/INSERT		A-325 OR BETTER BOLTS USED W/INSERT		A-307 BOLTS OR A-36 THD. RODS USED W/INSERT		A-325 OR BETTER BOLTS USED W/INSERT	
		T	S	T	S	T	S	T	S	T	S	T	S
10" OR THICKER	I	6	6	6	6	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 I (Contd.)

OBE ALLOWABLES
AND
SSE ALLOWABLES

R6

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

I : INSERT CAPACITY B : BOLT CAPACITY T : TENSION S : SHEAR

CONCRETE THICKNESS		INSERT SPACING ON 20"C/C 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 I (Contd.)

OBE ALLOWABLES
AND
SSE ALLOWABLES

R6

ALLOWABLE LOADS OF 1 1/2" ø RICHMOND INSERTS (EC6W) AND BOLTS, (IN CLUSTER) TO BE USED IN INTERACTION FORMULAS FOR WALLS, SLABS & COLUMNS IN 12" THICK CONCRETE

I: INSERT CAPACITY B: BOLT CAPACITY T: TENSION S: SHEAR

INSERT PATTERN		INSERT SPACING ON 12" C/C BOTH WAYS				INSERT SPACING ON 18" C/C BOTH WAYS			
		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
TWO INSERTS ●●	I	22.1	22.1	22.1	22.1	25	25	25	25
	B	28.11	17.67	56.21	26.51	28.11	17.67	56.21	26.51
FOUR INSERTS ●●/●●	I	17.29	17.29	17.29	17.29	23.21	23.21	23.21	23.21
	B	28.11	17.67	56.21	26.51	28.11	17.67	56.21	26.51
SIX INSERTS ●●/●●/●●	I	15.24	15.24	15.24	15.24	21.16	21.16	21.16	21.16
	B	28.11	17.67	56.21	26.51	28.11	17.67	56.21	26.51
NINE INSERTS ●●●/●●●/●●●	I	12.57	12.57	12.57	12.57	17.83	17.83	17.83	17.83
	B	28.11	17.67	56.21	26.51	28.11	17.67	56.21	26.51

TABLE I (Contd.)

~~SSE ALLOWABLES~~ AND OBE ALLOWABLES

1R

ALLOWABLE LOADS OF 1/2"ø RICHMOND INSERTS (EC6W) AND BOLTS, (IN CLUSTER) TO BE USED IN INTERACTION FORMULAS FOR WALLS, SLABS & COLUMNS IN 16" THICK CONCRETE

I: INSERT CAPACITY B: BOLT CAPACITY T: TENSION S: SHEAR

INSERT PATTERN		INSERT SPACING ON 10"C/C BOTH WAYS				INSERT SPACING ON 12"C/C BOTH WAYS			
		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
TWO INSERTS	I	20.45	20.45	20.45	20.45	22.1	22.1	22.1	22.1
	B	28.11	17.67	56.21	26.51	28.11	17.67	56.21	26.51
FOUR INSERTS	I	16.05	16.05	16.05	16.05	18.6	18.6	18.6	18.6
	B	28.11	17.67	56.21	26.51	28.11	17.67	56.21	26.51
SIX INSERTS	I	14.59	14.59	14.59	14.59	17.44	17.44	17.44	17.44
	B	28.11	17.67	56.21	26.51	28.11	17.67	56.21	26.51
NINE INSERTS	I	12.57	12.57	12.57	12.57	14.9	14.9	14.9	14.9
	B	28.11	17.67	56.21	26.51	28.11	17.67	56.21	26.51
SIXTEEN INSERTS	I	10.06	10.06	10.06	10.06	12.03	12.03	12.03	12.03
	B	28.11	17.67	56.21	26.51	28.11	17.67	56.21	26.51

SSE ALLOWABLES AND OBE ALLOWABLE

TABLE I (Contd.)

R6

ALLOWABLE LOADS OF 1½"ø RICHMOND INSERTS (EC6W) AND BOLTS (IN CLUSTER) TO BE USED IN INTERACTION FORMULAS FOR WALLS, SLABS & COLUMNS IN 22" THICK OR GREATER CONCRETE

I: INSERT CAPACITY   B: BOLT CAPACITY   T: TENSION   S: SHEAR

INSERT PATTERN		INSERT SPACING ON 10" C/C BOTH WAYS				INSERT SPACING ON 12" C/C BOTH WAYS			
		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
TWO INSERTS	I	20.45	20.45	20.45	20.45	22.1	22.1	22.1	22.1
	B	28.11	17.67	56.21	26.51	28.11	17.67	56.21	26.51
FOUR INSERTS	I	16.05	16.05	16.05	16.05	18.6	18.6	18.6	18.6
	B	28.11	17.67	56.21	26.51	28.11	17.67	56.21	26.51
SIX INSERTS	I	14.59	14.59	14.59	14.59	17.44	17.44	17.44	17.44
	B	28.11	17.67	56.21	26.51	28.11	17.67	56.21	26.51
NINE INSERTS	I	13.15	13.15	13.15	13.15	16.22	16.22	16.22	16.22
	B	28.11	17.67	56.21	26.51	28.11	17.67	56.21	26.51
SIXTEEN INSERTS	I	11.54	11.54	11.54	11.54	14.25	14.25	14.25	14.25
	B	28.11	17.67	56.21	26.51	28.11	17.67	56.21	26.51

TABLE I (Contd.)

SSE ALLOWABLES AND OBE ALLOWABLES

R6

SS-30

APPENDIX 4

DESIGN CRITERIA FOR
EMBEDDED PLATE STRIPS

APPENDIX 4

DESIGN CRITERIA FOR EMBEDDED PLATE STRIPS

1.0 DESCRIPTION

Embedded plate strips are ASTM A36 steel plates, 3/4" thick and 8" or 10" wide, embedded in concrete walls, columns, sides of beams and the underside of floor or roof slabs and attached to the concrete by means of Nelson studs welded to the plate. They are used to support hangers and other structural supports which are connected to the embedded plate by welding or by threaded Nelson studs. The design of the threaded Nelson studs and the weld at the connection to the embedded plate is the responsibility of the designer of the hanger or the structural support.

2.0 APPLICABLE REFERENCES

1. Manual of Steel Construction AISC 7th edition
2. Design Data 10 - Embedment Properties of Headed Studs-TRW Nelson Division 2-77

3.0 CAPACITY OF EMBEDDED PLATE STRIPS FOR CONCENTRIC LOADING

3.1 Allowable loads on embedded plate strips are shown on sheet A4.1 and A4.2 for loadings acting at mid-spans between studs and sheet A4.3 and A4.4 for loadings acting at stud locations. However, as shown on sheet A4.3, the maximum allowable tensile load at the extreme stud location on both ends of the embedded plate strip is reduced by 40 percent. For loadings acting between mid-span and stud location the allowable load shall be determined by linear interpolation.

3.2 Loading is not permitted on the cantilever portions of the embedded plate strips beyond the last pair of studs.

3.3 Allowable loads as shown on sheet A4.2 and A4.4 are valid only when loadings are placed within ±3/4" of the centerline of the embedded plate and only if the Nelson studs of the embedded plate are located at least 8" from a concrete free edge (i.e., openings, face of beam, etc.) in any direction.

3.4 "Pin Connections" shall be assumed for load transfer to the embedded plates. Only forces normal to the embedment (P) and forces in the plane of the embedment (S) may be transferred to the embedment. Moments due to cantilever action or from any other source may be transferred to the embedment only when the embedment is stiffened for the calculated moment.

3.5 The loading pattern on sheet A4-1 and A4-3 assumes that the embedment is loaded at the midpoint of every span between pairs of studs for A4-1 and at every pair of studs for A4-3. In cases in which the load is distributed on more than one pair of studs, the full normal load (P) and only half of the plane load (S) should be considered when using the figures on sheet A4-2 and A4-4.

3.6 For capacity of embedded plate strips for loads acting on stud line see Cases 3 and 4 (A4-5 through A4-9)

4.0 CAPACITY OF EMBEDDED PLATE STRIPS FOR LARGE ECCENTRIC LOADING

4.1 Tension and shear forces generated on the stud anchors by loads applied eccentric to the supporting stud group should be calculated to insure no failure of the stud anchors.

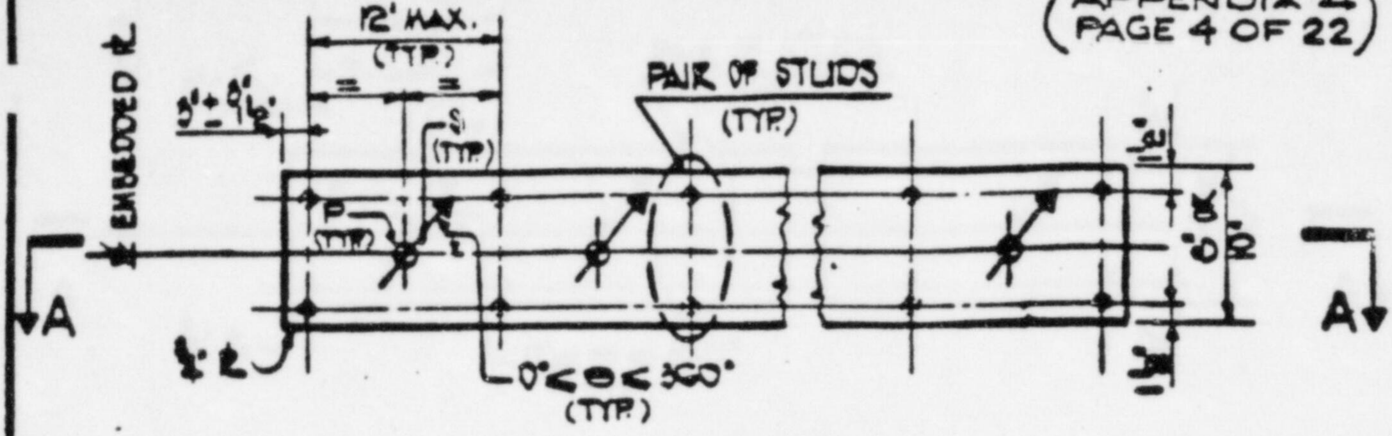
4.2 Ultimate tension and shear capacities of the stud anchors shall be taken from reference 2 of this Appendix.

4.3 The number of participating stud anchors may be increased by welding stiffeners to the embedded plate strips and to the support structure to ensure that the loading is spread to all the selected stud anchors. The embedded plate strip shall be checked for bending and shear.

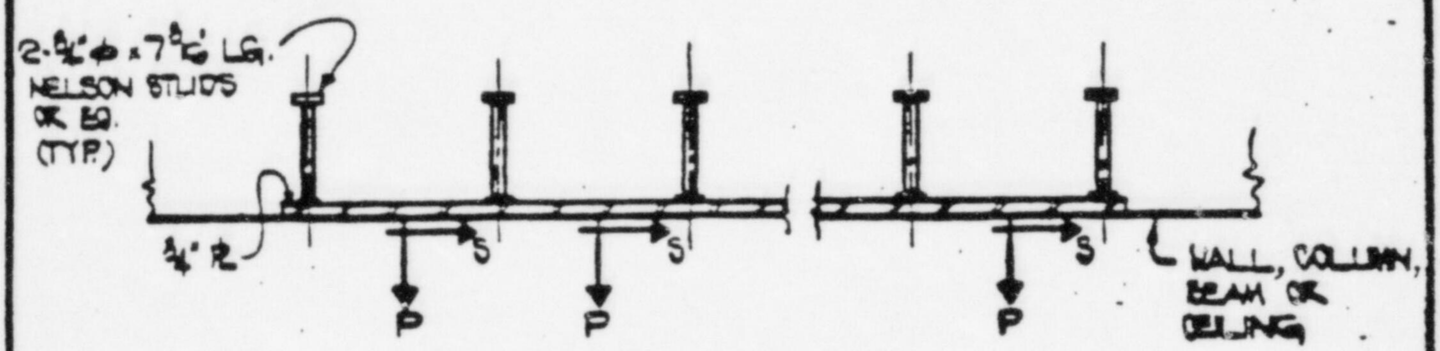
4.4 Steel plate material is A-36 Nuclear Safety Related as defined on Drawing 2323-S-0786 for embedded plate details.

5.0 REDUCED CAPACITIES OF HILTI EXPANSION BOLT - STRIP PLATE VIOLATING MINIMUM SEPARATION REQUIREMENT

5.1 For calculational procedures see final pages of this Appendix.



PLAN OR ELEVATION



SECTION A-A

CASE 1

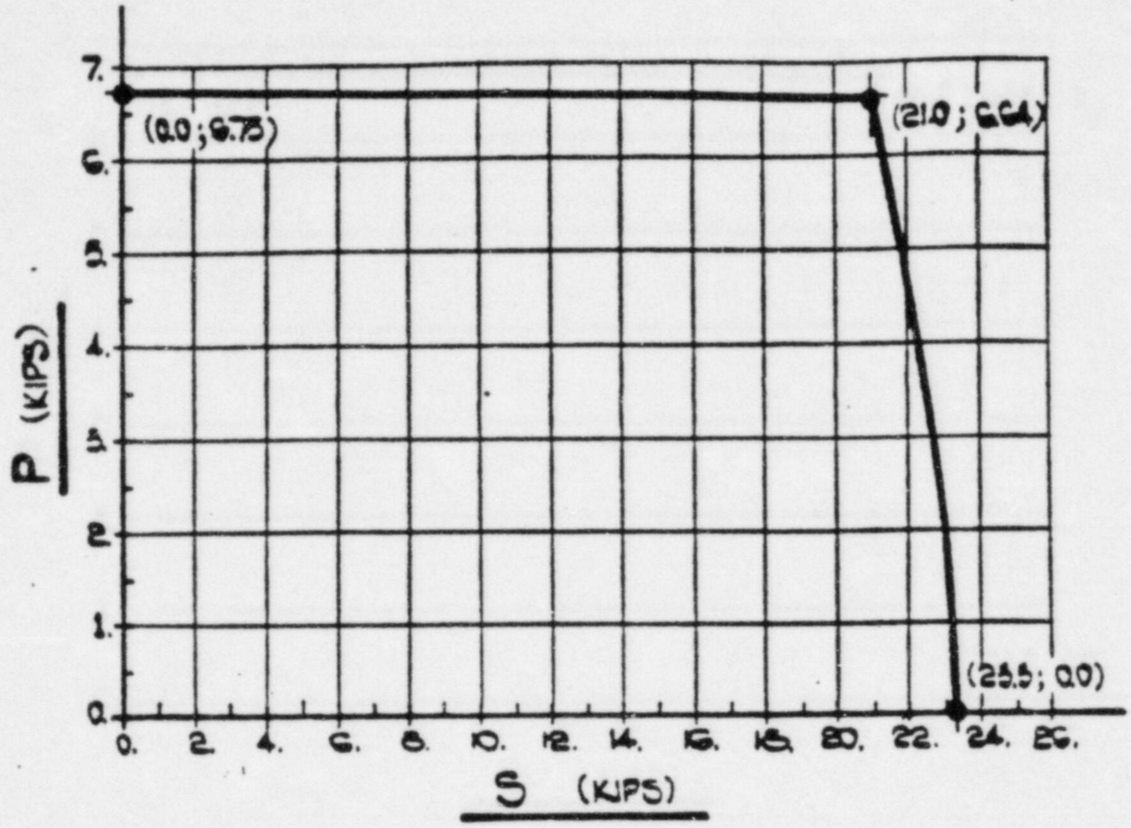
LOADINGS AT MID SPAN BETWEEN STUDS

NOTATION:

P--- ACTUAL APPLIED TENSION LOAD.

S--- ACTUAL APPLIED SHEAR LOAD.

TUSI COMANCHE PEAK	
ALLOWABLE LOADS ON EMBEDDED PLATES	
Gibbs & Hill Inc.	—
2323	SH. A4-1



ALLOWABLE EMBEDDED PLATE CAPACITY FOR COMBINED TENSION AND SHEAR LOADS

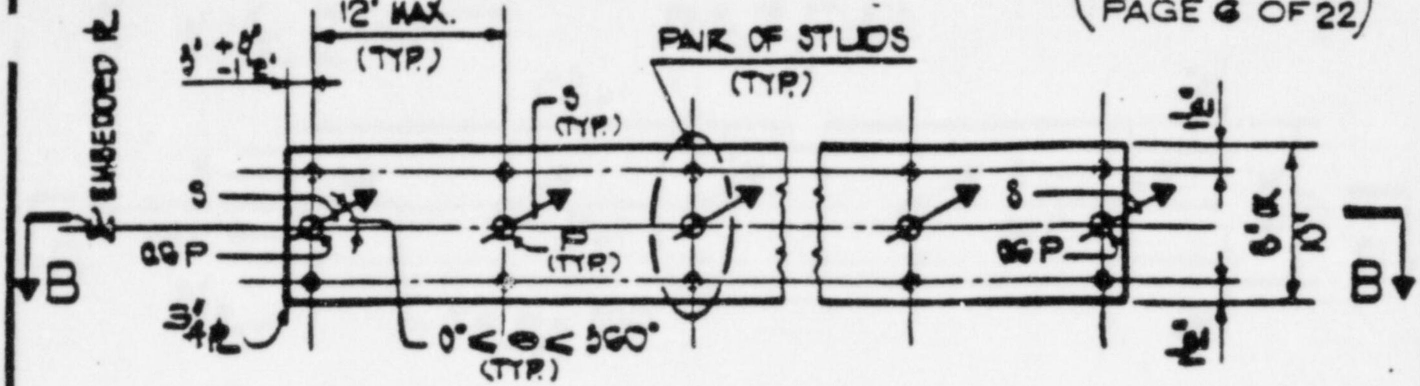
INTERACTION DESIGN CURVE FOR LOADINGS ACTING AT MID-SPAN BETWEEN STUDS

CASE 1

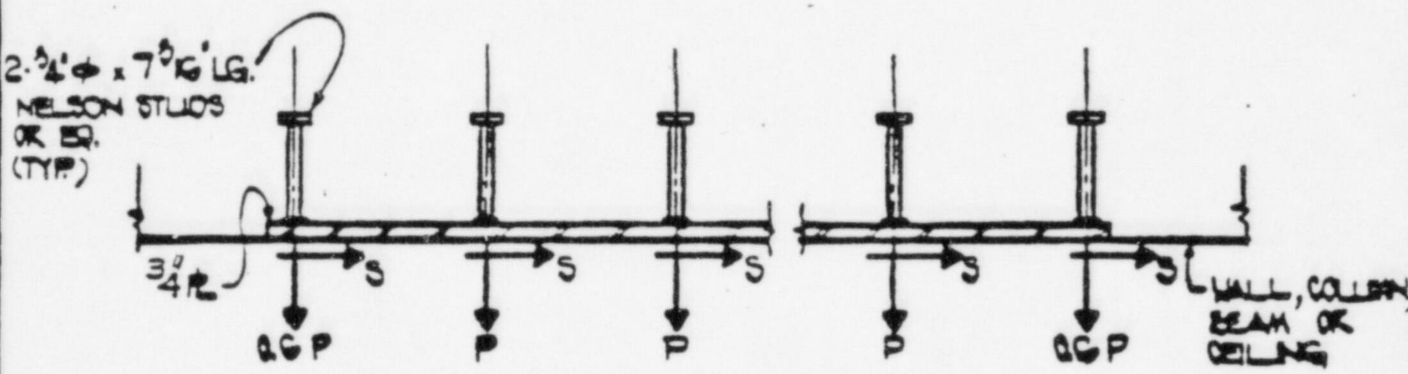
NOTATION:

P---ACTUAL APPLIED TENSION LOAD.
S---ACTUAL APPLIED SHEAR LOAD.

TUSI COMANCHE PEAK	
ALLOWABLE LOADS ON EMBEDDED PLATES	
Gibbs & Hill, Inc.	—
2323	SH. A4-2



PLAN OR ELEVATION



SECTION B–B

CASE 2
LOADINGS AT STUD LOCATIONS

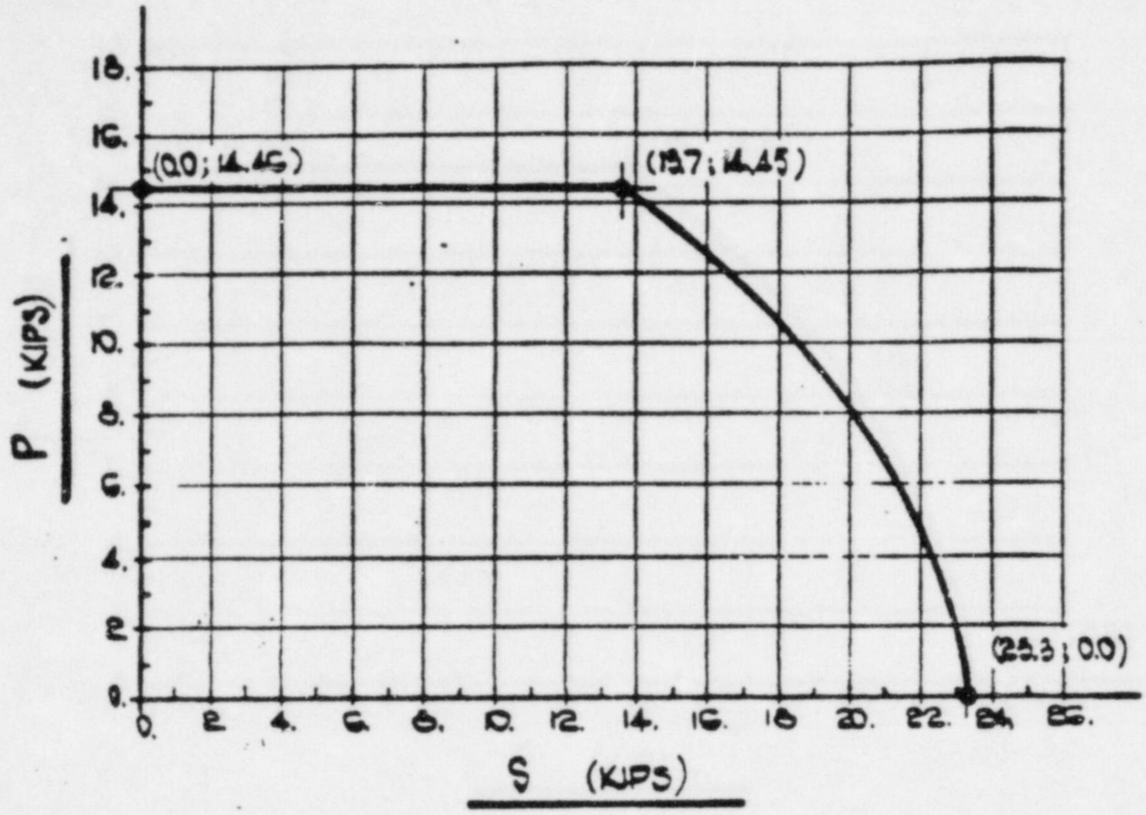
NOTATION:

P--- ACTUAL APPLIED TENSION LOAD.
S--- ACTUAL APPLIED SHEAR LOAD.

TUSI
COMANCHE PEAK

ALLOWABLE LOADS ON
EMBEDDED PLATES

2323 | SH. A4-3



ALLOWABLE EMBEDDED PLATE CAPACITY FOR COMBINED TENSION AND SHEAR LOADS

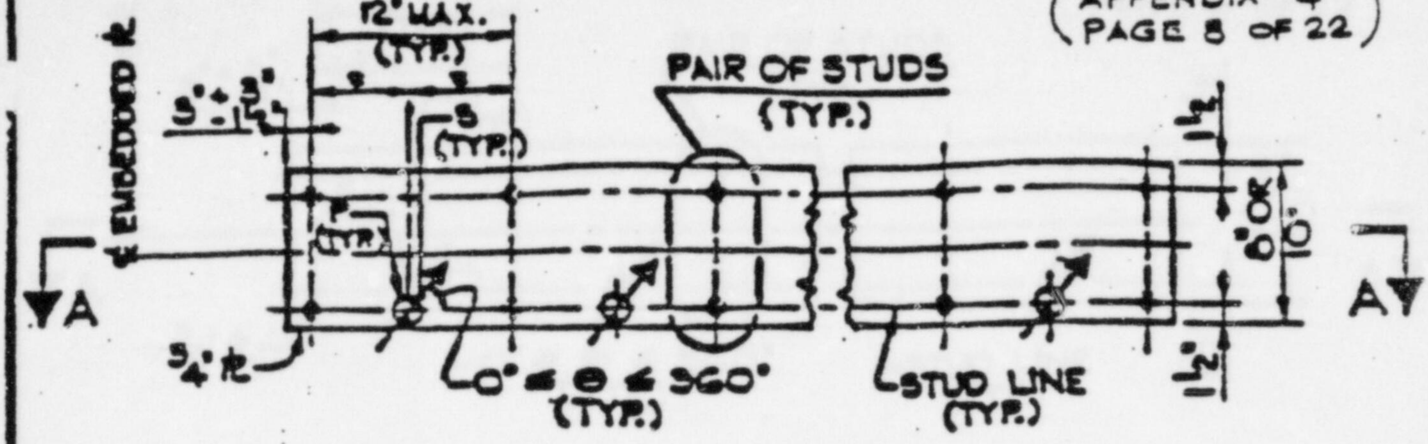
INTERACTION DESIGN CURVE FOR LOADINGS ACTING AT STUD LOCATIONS

CASE 2

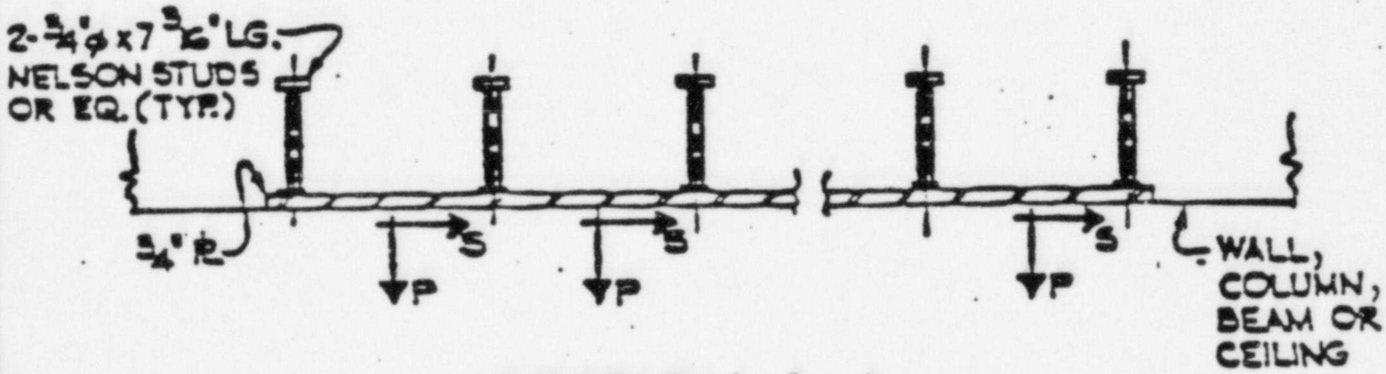
NOTATION:

P--- ACTUAL APPLIED TENSION LOAD.
S--- ACTUAL APPLIED SHEAR LOAD.

TUSI COMANCHE PEAK	
ALLOWABLE LOAD ON EMBEDDED PLATES	
Gibbs & Hill, Inc.	—
2323	SH. A4-4



PLAN OR ELEVATION



SECTION A-A

CASE-3
LOADINGS AT MID SPAN BETWEEN STUDS ON STUD LINE

NOTATION:

P... ACTUAL APPLIED TENSION LOAD
S... ACTUAL APPLIED SHEAR LOAD

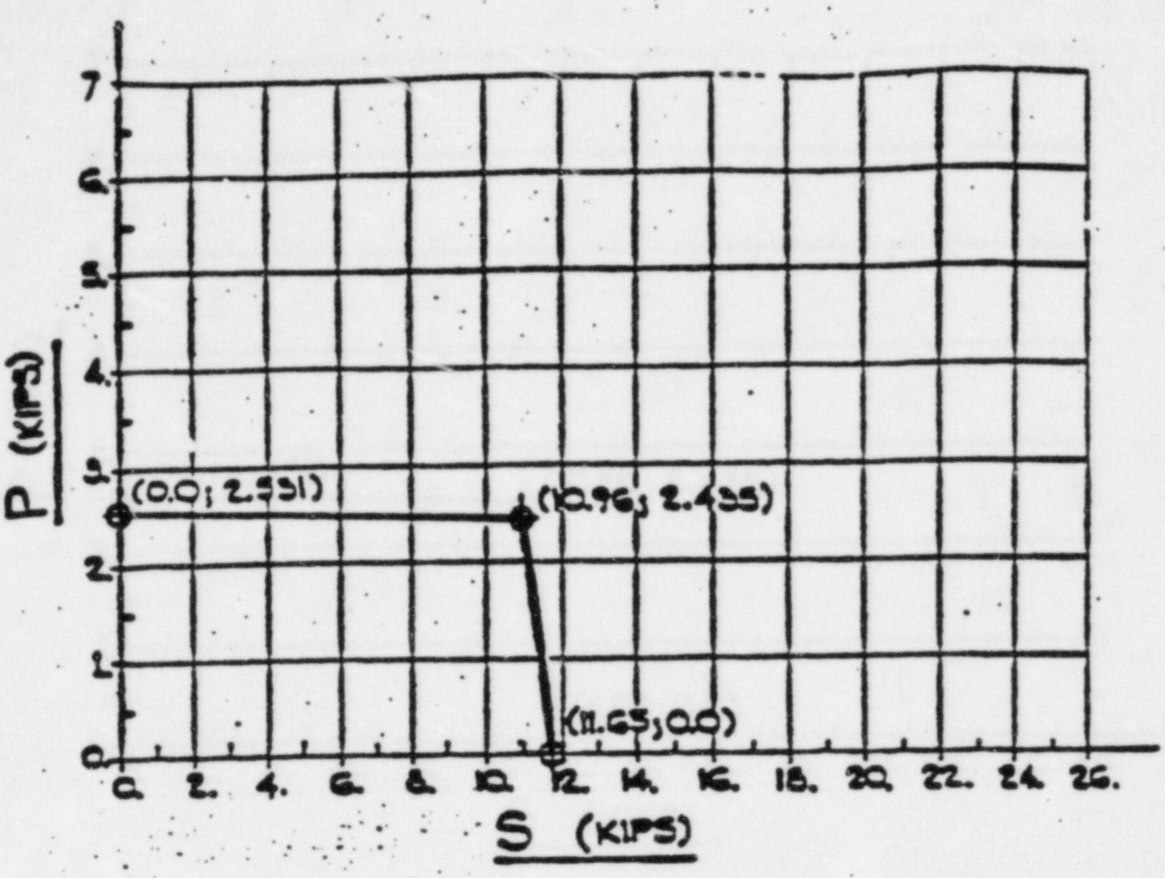
TUSI
COMANCHE PEAK

ALLOWABLE LOADS ON
EMBEDDED PLATES

GIBBS & HILL INC.

2323

SH. A4-5



ALLOWABLE EMBEDDED PLATE CAPACITY FOR COMBINED TENSION AND SHEAR LOADS.

INTERACTION DESIGN CURVE FOR LOADINGS ACTING AT MID-SPAN BETWEEN STUDS ON STUD LINE

CASE-3

NOTATION:

P... ACTUAL APPLIED TENSION LOAD
S... ACTUAL APPLIED SHEAR LOAD

TUSI
COMANCHE PEAK

ALLOWABLE LOADS ON EMBEDDED PLATES

GIBBS & HILL INC.

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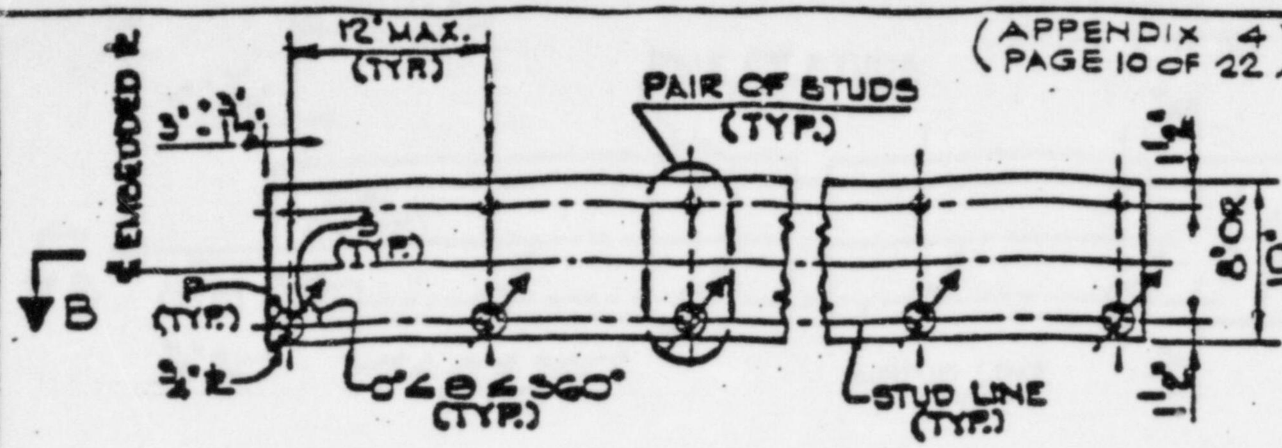
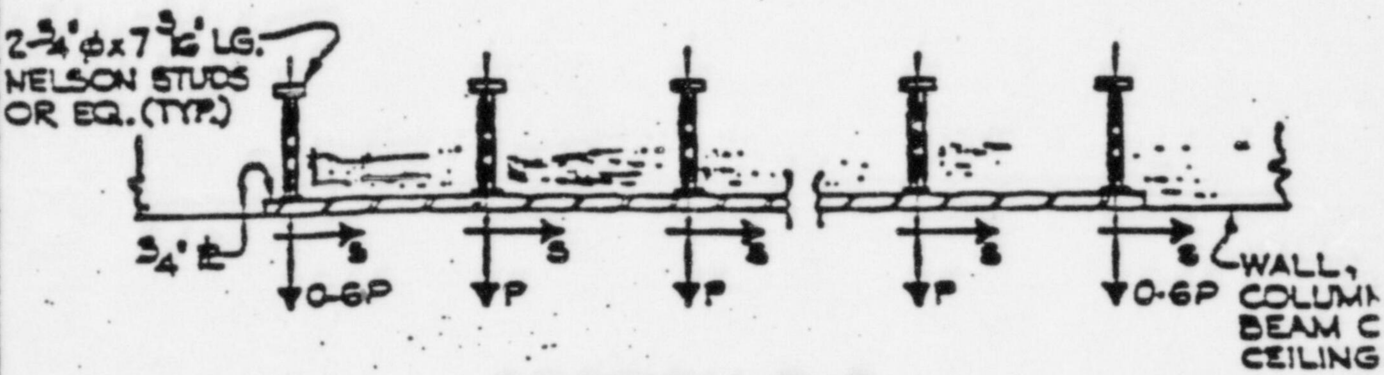
SH. A4-6



PLAN OR ELEVATION



SECTION B-B

CASE-4
LOADINGS AT STUD LOCATION ON STUD LINE

NOTATION:

P - ACTUAL APPLIED TENSION LOAD
S - ACTUAL APPLIED SHEAR LOAD

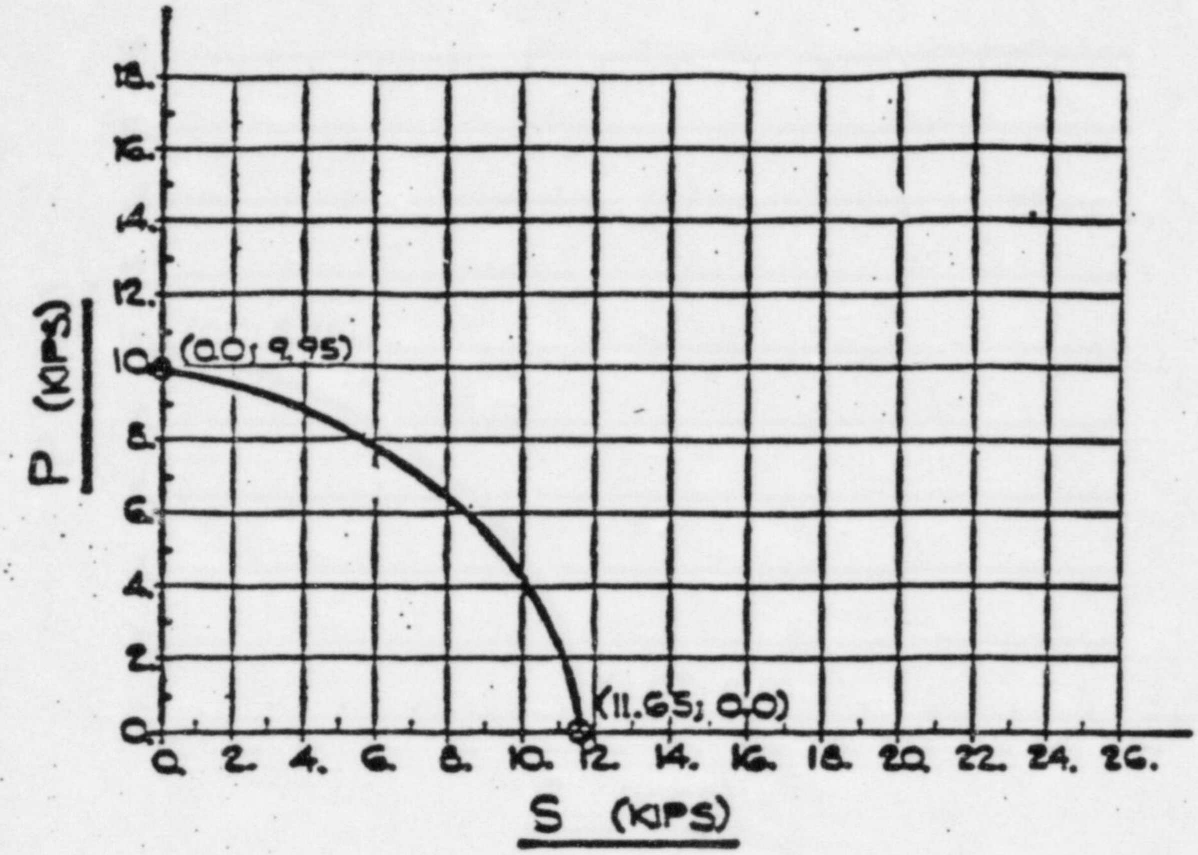
TUSI
COMANCHE PEAK

ALLOWABLE LOADS ON EMBEDDED PLATES

Gibbs & Hill, Inc.

2323

SH. A4-7



ALLOWABLE EMBEDDED PLATE CAPACITY FOR COMBINED TENSION AND SHEAR LOADS

INTERACTION DESIGN CURVE FOR LOADINGS ACTING AT STUD LOCATIONS ON STUD LINE

CASE - 4

NOTATION:

P - ACTUAL APPLIED TENSION LOAD

S - ACTUAL APPLIED SHEAR LOAD

TUSI
COMANCHE PEAK

ALLOWABLE LOAD ON EMBEDDED PLATES

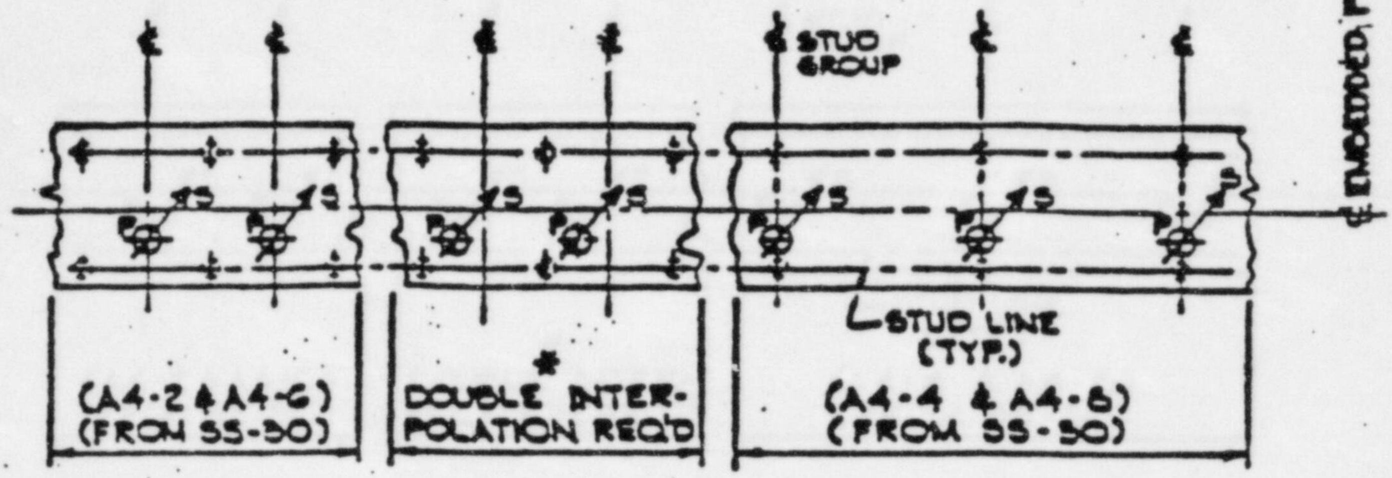
GIBBS & HILL INC.

2323

SH. A4-

NOTE

(1) IF THE LOAD LIES IN BETWEEN THE CENTER LINE OF PLATE AND THE STUD LINE, INTERPOLATION MAY BE MADE BETWEEN CORRESPONDING GRAPHS AND/OR FORMULAE.



* IF THE VARIATION OF LOAD LOCATION IS IN TWO DIRECTIONS DOUBLE INTERPOLATION IS REQUIRED.

(2) LOADS (P & S) SHALL NOT BE APPLIED BEYOND STUD LINE TOWARDS EDGE OF THE PLATE.

(3) FOR ADDITIONAL APPLICABLE NOTES SEE

TUSI COMANCHE PEAK	
ALLOWABLE LOADS ON EMBEDDED PLATES	
GIBBS & HILL, INC.	
2323	SH. A4-9

Reduced Capacities of Hilti Expansion Bolt-Strip Plate, Violating Minimum Separation Requirement

Calculation of the reduced allowable capacities for Hilti expansion anchors and embedded strip plates spaced at less than minimum separation requirement indicated in attachment 3 item 2 in Appendix 1 of G&H specification SS-30 (CE I-20 Rev. 8)

Notation

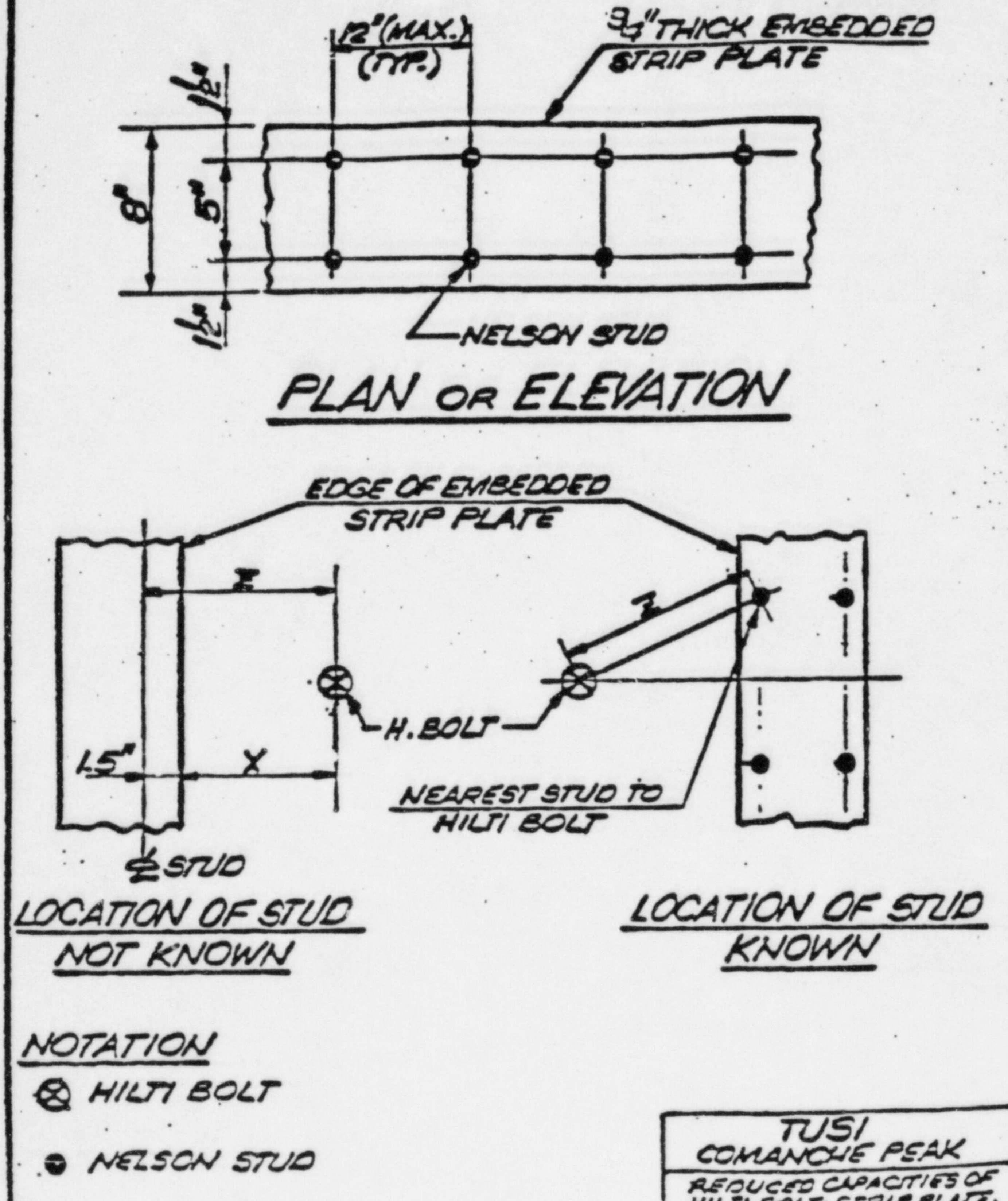
- d — Diameter of Hilti bolt (in)
- X — Distance between Hilti bolt and nearest edge of embedded strip plate (in)
- Z — Actual or estimated minimum distance between Hilti bolt and nearest Nelson stud of embedded strip plate (in)
  $Z=X+1.5$
- Z_1 — Minimum distance between Nelson stud of embedded strip plate and Hilti bolt for each to have 50% capacity (in)
  $Z_1=1.5+2.5d$
- Z_2 — Minimum distance between Nelson stud of embedded strip plate and Hilti bolt for each to have 100% capacity (in)
  $Z_2=4.0+5.0d$
- R — Allocation ratio for distance "Z"
  $R = \frac{d}{d+1.0}$
- a — Distance allocated to Hilti bolt (in)
  $a=R\ (Z-Z_1) + 2.5d$
- b — Distance allocated to Nelson stud of embedded stip plate (in)
  $b=Z-a \qquad b\geq 1.5"$
- S.R. — Separation ratio for Hilti bolt
  $S.R. = \frac{a}{5d}$
- R_{TU} — Tensile capacity reduction of Nelson stud due to separation requirement violation (kips)
- T' — Allowable (working) capacity of Nelson stud in tension (kips per stud)
- S' — Allowable (working) capacity of Nelson stud in shear (kips per stud)
- T_A — Allowable design tension load for Hilti bolt, see Tables 1 and 2, Appendix 2 of G&H specification SS-30
- S_A — Allowable design shear load for Hilti bolt, see Tables 1 and 2, Appendix 2 of G&H Specification SS-30

T_R Reduced allowable tension capacity for Hilti bolt (kips)

S_R Reduced allowable shear capacity for Hilti bolt (kips)

P Actual applied tension load on embedded strip plate (kips)

S Actual applied shear load on embedded strip plate (kips)



TUSI
COMANCHE PEAK

REDUCED CAPACITIES OF HILTI BOLT-STRIP PLATE, VIOLATING MINIMUM SEPARATION REQUIREMENT

Gibbs & Hill, Inc.

2323 | SH. A4 - 10

PROCEDURES

Step 1

Find out the distance 'Z' between the nearest Nelson stud of embedded strip plate and Hilti bolt.

a) If location of stud is known, measure 'Z'.

b) If location of stud is not known, measure 'X' where 'X' = distance between Hilti bolt and nearest edge of embedded strip plate
$Z = X + 1.5"$

Step 2

Determine whether spacing violation exists:

Min. Z req'd = $Z_1 = 1.5 + 2.5d$
-if $Z < Z_1$, not acceptable, relocate Hilti bolt
-if $Z \geq Z_2 = 4.0 + 5d$, both stud and Hilti bolt are fully developed therefore no spacing violation exists and no reduction is req'd.
-if $\underbrace{1.5 + 2.5d}_{Z_1} \leq Z < \underbrace{4.0 + 5d}_{Z_2}$,

a spacing violation exists, proceed to step 3

Step 3

Calculate the reduced allowable capacities of the Hilti bolt.

$$R = \frac{d}{d+1.0}$$

$$a = R(Z-Z_1) + 2.5d$$

$$b = Z-a$$

$$S.R. = \frac{a}{5d}$$

Reduced allowable (working) capacity of Hilti bolt in tension and shear

$T_R = T_A$ (S.R.)

$S_R = S_A$ (S.R.)

Step 4

Calculate the reduced allowable capacities of Nelson stud.

$R_{TU} = 12.4 - 2.5b$ (for $1.5 \le b \le 3.5$)

$R_{TU} = 28.9 - 7.2b$ (for $3.5 < b \le 4.0$)

Reduced allowable (working) capacity of Nelson stud in tension,

$T' = 9.95 - \frac{R_{TU}}{2}$ (kips/stud)

Step 5

Verification of embedded strip plate adequacy.

A) Location of Nelson studs of the embedded strip plate is known.

Case 1: Loadings (P&S) acting at midspan between studs; embedded strip plate is adequate when equations 1 and 2 are both satisfied.

$$\left[\left(\frac{S}{107.16}\right)^2 + \left(\frac{P}{6.75}\right)^2\right]^{1/2} \le 1.0 \qquad (1)$$

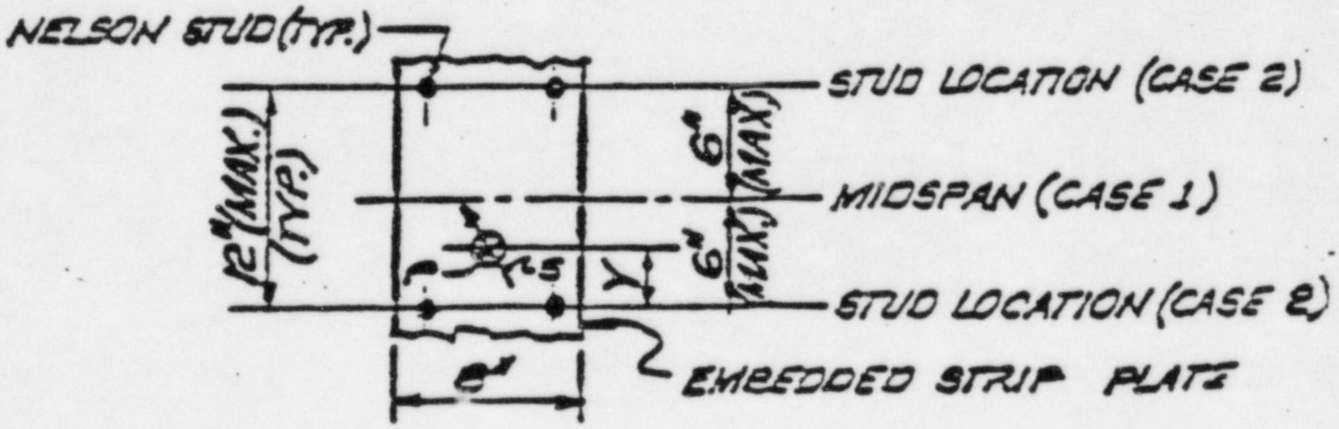
$$\left(\frac{S}{17.92}\right)^{5/3} + \left(\frac{P}{1.54(T')}\right)^{5/3} \leq 1.0 \qquad (2)$$

Case 2: Loadings (P&S) acting at stud location; embedded strip plate is adequate when equations 3 and 2 are both satisfied.

$$\left[\left(\frac{S}{306.00}\right)^{2} + \left(\frac{P}{14.46}\right)^{2}\right]^{1/2} \leq 1.0 \qquad (3)$$

$$\left(\frac{S}{17.92}\right)^{5/3} + \left(\frac{P}{1.54(T')}\right)^{5/3} \leq 1.0 \qquad (2)$$

Case 3: Loadings (P&S) acting somewhere between case 1 and case 2



Calculation procedure:

(1) Measure distance 'Y' (from nearest stud location to applied load P&S).

(2) With known S (or P) calculate allowable P (or S) for both case 1 and case 2 as per equations 1, 2, and 3, 2 respectively.

(3) Interpolate by the use of either of the two following equations 4 or 5.

$$P\ case_3 = P\ case_1 + \left(\frac{6 - Y}{6}\right)(P\ case_2 - P\ case_1)\ \text{Kips} \quad (4)$$

(Allowable)

OR

$$S\ case_3 = S\ case_1 + \left(\frac{6 - Y}{6}\right)(S\ case_2 - S\ case_1)\ \text{Kips} \quad (5)$$

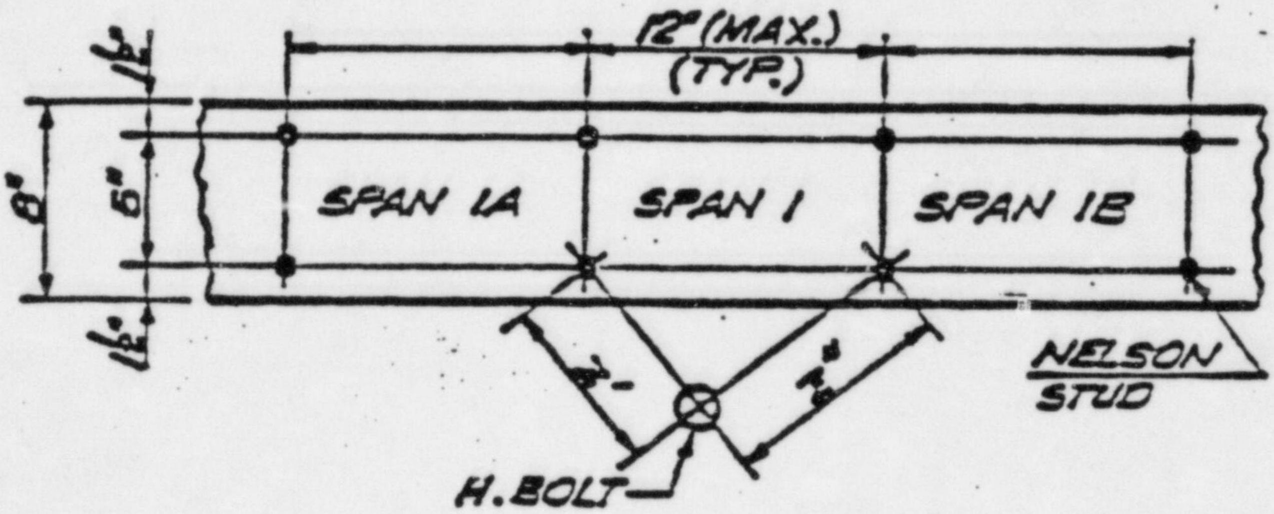
(Allowable)

(4) Compare P (allow) of case 3 (or S (allow) of case 3) with actual P (or S).

B) Location of Nelson studs of the embedded strip plate not known:

Embedded strip plate is adequate when equations 1 and 2 are both satisfied.

ALLOWABLE LOADS FOR ADJACENT SPANS



a) Z' & Z" are used here only for illustrative purposes.

b) Z' < Z" and both Z' & Z" are bolt violations.

c) The maximum capacity of the embedded strip plate, in particular span 1, is determined by calculating the allowable (working) capacity of the Nelson stud nearest to the Hilti bolt, (Z' in this case since Z'<Z").

d) If a load is to be placed on span 1A, the maximum capacity determined for span 1 may be used for span 1A provided that no other spacing violation exists for any other Nelson stud supporting span 1A. If another spacing violation does exist, then choose the smallest Z dimension for any one of the

4 studs of span 1A to determine the load capacity by using the procedures cutlined on the preceeding pages.

e) If a load is to be placed on span 1B, the maximum capacity of strip plate is determined by calculating the capacity of the Nelson stud located at 2" distance from Hilti Bolt, as illustrated above, provided that no other spacing violation exists for any other Nelson stud supporting span 1B. Follow the procedure as mentioned above in note d if another spacing violation exists.

SS-30

APPENDIX 4W

DESIGN CRITERIA FOR EMBEDDED PLATE STRIPS (ALTERNATE)

(Attachment to Westinghouse Document No. 10923
Transmitted with WPT-8031 and SD-433 Dated 5/3/86

APPENDIX 4W

DESIGN CRITERIA FOR EMBEDDED PLATE STRIPS

JUNE 3, 1986
Revision 1

AUTHORS: Richard Orr
R. S. Orr

R Condrac
R. Condrac

VERIFIER: H P Bonnet
H. P. Bonnet

APPROVED: M. Mahlab
M. Mahlab

1.0 DESCRIPTION

Embedded plate strips are ASTM A36 steel plates, 3/4" thick and 8" or 10" wide, embedded in concrete walls, columns, sides of beams and the underside of floor or roof slabs, and attached to the concrete by means of Nelson studs welded to the plate. The design of the threaded Nelson studs and the weld at the connection to the embedded plate is the responsibility of the designer of the hanger or the structural support.

2.0 APPLICABLE REFERENCES

2.1 Manual of Steel Construction AISC 7th edition

2.2 Design Data 10 - Embedment Properties of Headed Studs-TRW Nelson Division 2-77

3.0 LOCATION OF ATTACHMENTS

3.1 Attachments may be welded to the strip plate at any location provided that the centroid of the weld configuration is inside the stud area. For attachments that are welded on 2 opposite sides the centroid of each weld shall lie within the stud area.

3.2 Attachments should be located to meet a minimum spacing of 12" between the center lines of attachments measured along the center line of the plate strip (see Figure A4W-1). If this minimum spacing requirement is satisfied each attachment may be evaluated individually. If the spacing is less than twelve inches the attachments must be evaluated concurrently as specified in paragraph 4.4.

3.3 Allowable loads given in this section are only valid if the Nelson studs of the embedded plate are located at least eight inches from a concrete free edge (i.e. openings, face of beam, etc) in any direction.

3.4 Construction tolerance shall be considered during the design phase. Allowable loads are based on the eccentricity of the attachment center line from the centerline of the plate strip. The design eccentricity shall be increased by 3/4" to provide location tolerance during construction unless increased eccentricity is prevented by the one inch minimum edge distance of paragraph 3.1.

4.0 CAPACITY OF EMBEDDED PLATE STRIPS

4.1 The embedded plate shall be evaluated for loads from all attachments or both stud capacity as specified in Paragraph 4.2 and for plate bending as specified in Paragraph 4.3. Where attachments are located closer to each other than twelve inches attachments shall be evaluated concurrently as specified in Paragrah 4.4.

If the attachment is located less than 4 inches from the end of the strip plate, the allowable stud tension loads shall be reduced by a factor $\frac{S'}{8}$, if there is an adjacent strip plate. S' equals the distance between the end studs of the 2 plate strips, see Figure A-4W-3. Note that the shear capacity does not require reduction as long as a 3 inch spacing is maintained between studs.

If the end stud locations are not known the attachment weld centroid or centroids must be located at least six inches from the plate end, otherwise S must be assumed to equal the minimum possible spacing of 3".

Loads applied to the attachment are designated as F_x F_y F_z, M_x, M_y, M_z, (kips or inch kips) with z normal to the plate and y parallel to the plate center line. 'A' is the minimum dimension of the attachment cross-section. When a base plate is used, 'A' is the dimension from the compression face of the attachment member to the tension weld between the base plate and the embedment. 'E_x' is the eccentricity of the attachment center line from the center line of the plate.

4.2 Studs shall satisfy the allowables defined by the following equations:

Stud Tension: $T_s = (0.5F_z + 0.16M_x)(1 + 0.4E_x) + 0.2M_y\ (1 + 0.2E_x)$

Stud Shear: $V_s = 1/2\ [(F_x + \frac{M_z}{12})^2 + F_y^2]^{1/2}$

Interaction: $(\frac{T_s}{9.95})^{5/3} + (\frac{V_s}{11.65})^{5/3} \leq 1$

4.3 Plate stress shall be evaluated for attachments with minimum dimension less than four inches and shall satisfy the allowables defined by the following equations.

$f_1 = 0.375\ Vs$
$f_2 = 2.4\ (1 - .10A)\ (1 + .2E_x)\ F_z$
$f_3 = 1.1\ (1 - .15A)\ (1 + .07E_x)\ M_x$
$f_4 = 0.9\ (1 - .15A)\ (1 + .2Ex)\ M_y$
$f = f_1 + f_2 + f_3 + f_4$
$f \leq 27$ ksi

4.4 For attachments A and B located closer than twelve inches apart at spacing 's', calculate the stud loads (T_{sa}, T_{sb}, V_{sa}, V_{sb}) and maximum plate stress (f_a, f_b) using the equations given in 4.2 and 4.3. The combined stud loads (T_s, V_s) and plate stress (f) calculated from the following equation should then be checked in the stud interaction equation and against the allowable plate stress.

T_s = Greater of (T_{sa}, T_{sb}) $+(\frac{12 - S}{12})$ x lesser of (T_{sa}, T_{sb})

V_s = Greater of (V_{sa}, V_{sb}) $+(\frac{12 - S}{12})$ x lesser of (V_{sa}, V_{sb})

f = Greater of (f_a, f_b) $+(\frac{12 - S}{12})$ x lesser of (f_a, f_b)

These expressions assume that the higher loaded attachment is located at the most critical location. The influence of the other attachment is then obtained using linear interpolation between zero influence at twelve inch spacing and absolute summation at zero spacing.

4.5 The number of participating stud anchors may be increased by welding stiffeners to the embedded plate strips and to the support structure to ensure that the loading is spread to all the selected stud anchors. The embedded plate strip shall be checked for bending and shear.

4.6 Steel plate material is A-36 Nuclear Safety Related as defined on Drawing 2323-S—0786 for embedded plate details.

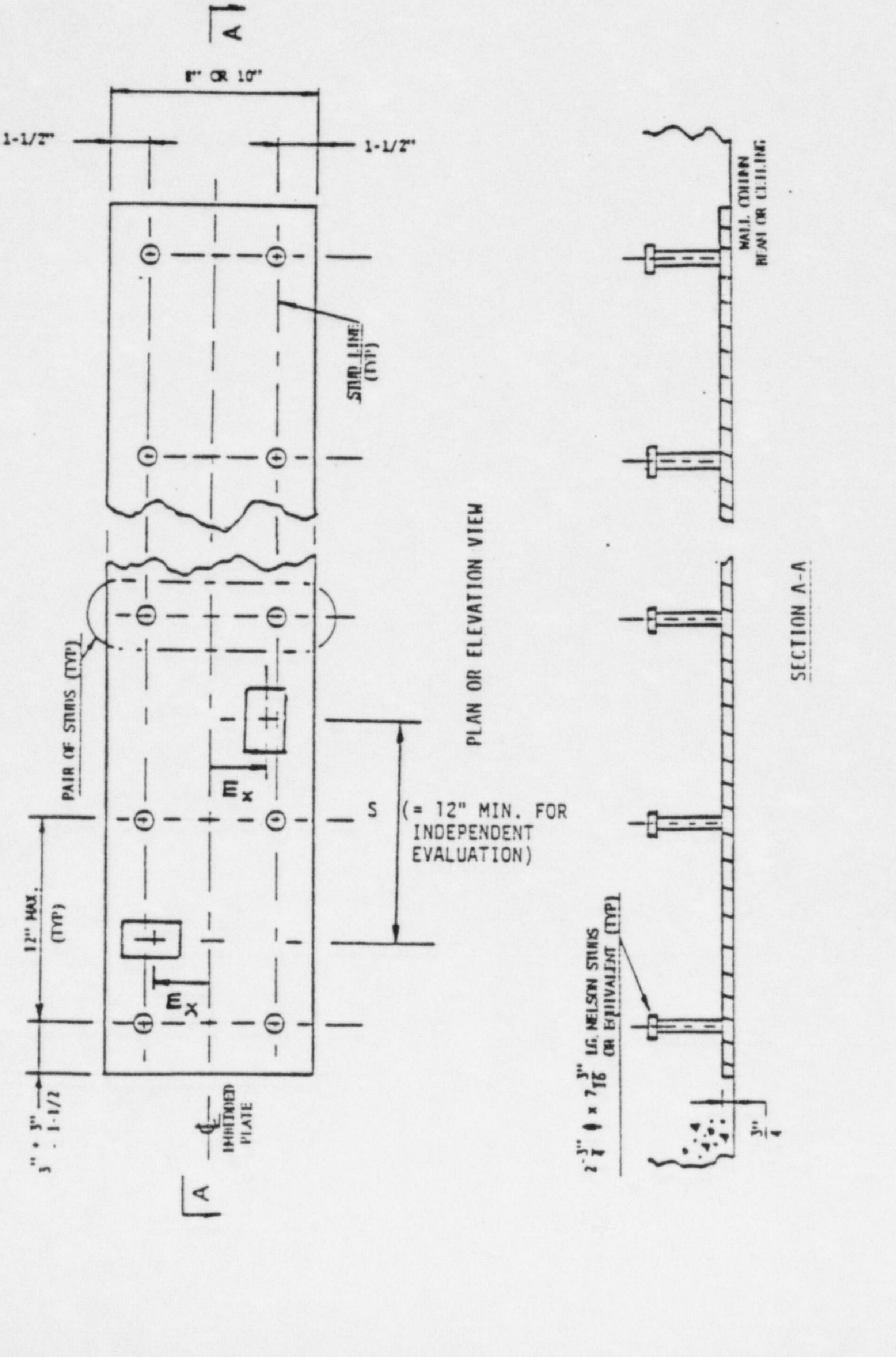

A
8" OR 10"
1-1/2"
1-1/2"
STUD LINE (TYP)
PAIR OF STUDS (TYP)
12" MAX. (TYP)
S
(= 12" MIN. FOR INDEPENDENT EVALUATION)
PLAN OR ELEVATION VIEW
WALL COLUMN BEAM OR CEILING
SECTION A-A
EMBEDDED PLATE
A

FIGURE A4W-1

5.0 REDUCED CAPACITIES OF HILTI EXPANSION BOLT - STRIP PLATE VIOLATING MINIMUM SEPARATION REQUIREMENT

5.1 The reduced allowable capacities for Hilti expansion anchors and embedded strip plates spaced at less than minimum separation requirement indicated in Attachment 3 item 2 in Appendix 1 of G&H specification SS-30 (CE I-20 Rev. 8) shall be calculated using the following procedure:

5.1.1 Notation (see Figure A4W-2)

d — Diameter of Hilti bolt (in)

X — Distance between Hilti bolt and nearest edge of embedded strip plate (in)

Z — Actual or estimated minimum distance between Hilti bolt and nearest Nelson stud of embedded strip plate (in) $Z = X + 1.5$.

Z_1 — Minimum distance between Nelson stud of embedded strip plate and Hilti bolt for each to have 50% capacity (in) $Z_1 = 1.5 + 2.5d$.

Z_2 — Minimum distance between Nelson stud of embedded strip plate and Hilti bolt for each to have 100% capacity (in) $Z_2 = 4.0 + 5.0d$.

R — Allocation ratio for distance "Z"

$$R = \frac{d}{d+1.0}$$

a — Distance allocated to Hilti bolt (in)

$$a = R\,(Z - Z_1) + 2.5d$$

b — Distance allocated to Nelson stud of embedded strip plate (in)

$$b = Z - a \qquad b \geq 01.5"$$

S.R. — Separation ratio for Hilti bolt

$$S.R. = \frac{a}{5d}$$

R_{TU} — Tensile capacity reduction of Nelson stud due to separation requirement violation (kips)

T' — Allowable (working) capacity of Nelson stud in tension (kips per stud)

S' — Allowable (working) capacity of Nelson stud in shear (kips per stud)

T_A — Allowable design tension load for Hilti bolt, see Tables 1 and 2, Appendix 2 of of G&H specification SS-30

S_A	Allowable design shear load for Hilti bolt, see Tables 1 and 2, Appendix 2 of G&H Specification SS-30
T_R	Reduced allowable tension capacity for Hilti bolt (kips)
S_R	Reduced allowable shear capacity for Hilti bolt (kips)
P	Actual applied tension load on embedded strip plate (kips)
S	Actual applied shear load on embedded strip plate (kips)

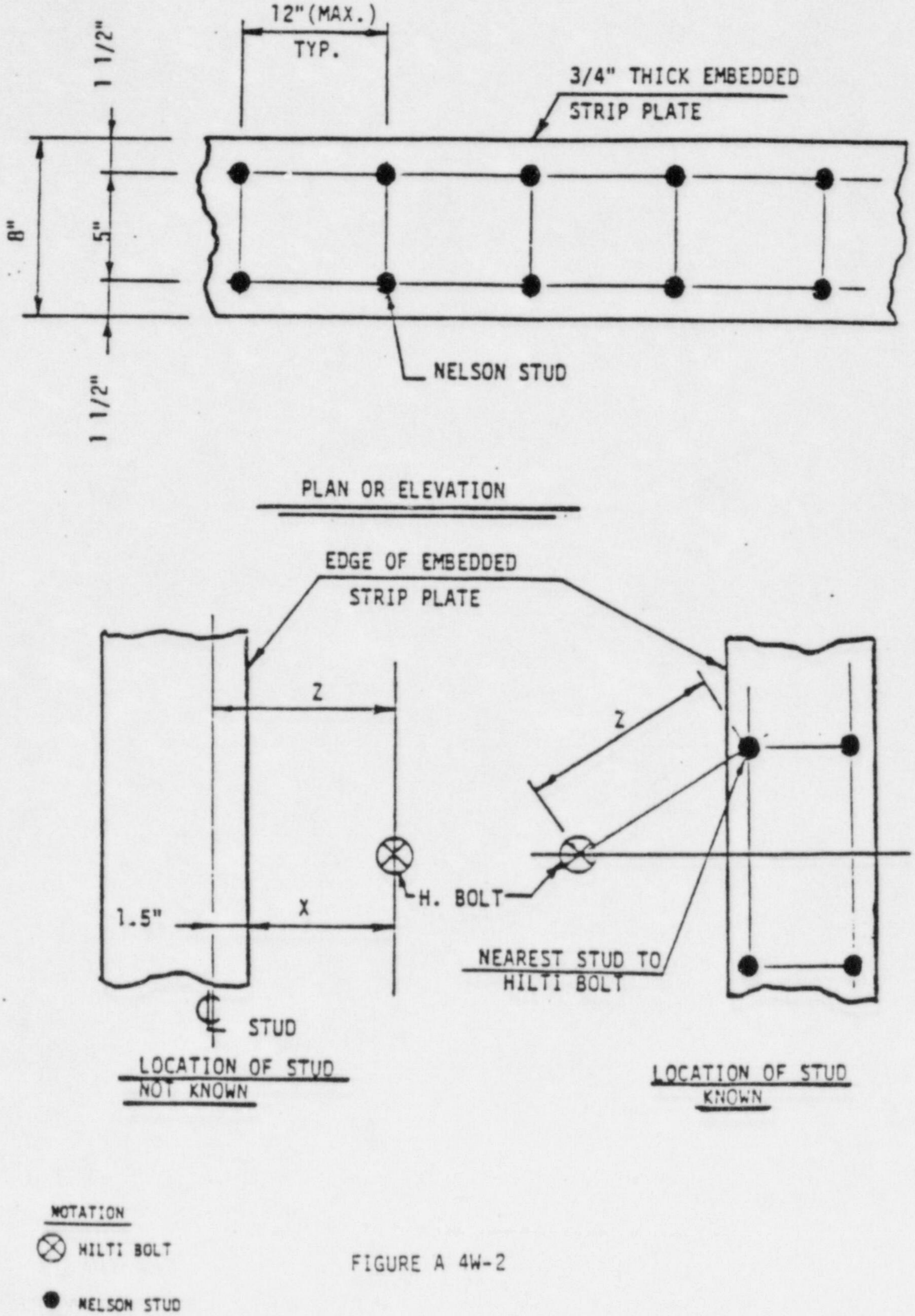

12" (MAX.)
TYP.
1 1/2"
3/4" THICK EMBEDDED
STRIP PLATE
8"
5"
NELSON STUD
1 1/2"
PLAN OR ELEVATION
EDGE OF EMBEDDED
STRIP PLATE
Z
Z
H. BOLT
1.5"
X
NEAREST STUD TO
HILTI BOLT
STUD
LOCATION OF STUD
NOT KNOWN
LOCATION OF STUD
KNOWN
NOTATION
HILTI BOLT
NELSON STUD

FIGURE A 4W-2

5.1.2 Calculation Procedure

Step 1

Determine the distance 'Z' between the nearest Nelson stud of embedded strip plate and Hilti bolt.

a) If location of stud is known, measure 'Z'.

b) If location of stud is not known, measure 'X' where 'X' = distance between Hilti bolt and nearest edge of embedded strip plate
$Z = X + 1.5"$

Step 2

Determine whether spacing violation exists:

Min. Z required = $Z_1 = 1.5 + 2.5d$
-if $Z < Z_1$, not acceptable, relocate Hilti bolt
-if $Z \geq Z_2 = 4.0 + 5d$, both stud and Hilti bolt are fully developed therefore no spacing violation exists and no reduction is required.

-if $\underbrace{1.5 + 2.5d}_{Z_1} \leq Z < \underbrace{4.0 + 5d}_{Z_2}$
a spacing violation exists, proceed to step 3

Step 3

Calculate the reduced allowable capacities of the Hilti bolt.

$R = \frac{d}{d+1.0}$

$a = R(Z-Z_1) + 2.5d$

$b = Z-a$

$S.R. = \frac{a}{5d}$

Reduced allowable (working) capacity of Hilti bolt in tension and shear

$$T_R = T_A \text{ (S.R.)}$$
$$S_R = S_A \text{ (S.R.)}$$

Step 4

Calculate the reduced allowable capacities of Nelson stud.

$$R_{TU} = 12.4-2.5b \text{ (for } 1.5 \leq b \leq 3.5)$$
$$R_{TU} = 28.9-7.2b \text{ (for } 3.5 \leq b \leq 4.0)$$

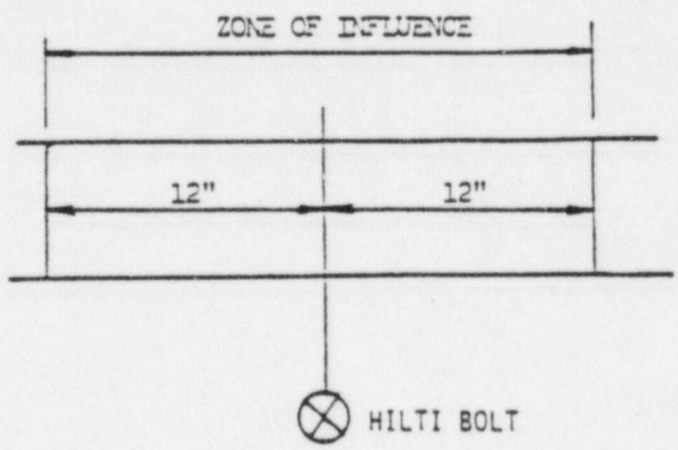
Reduced allowable (working) capacity of Nelson stud in tension.

$$T' = 9.95 - \frac{R_{TU}}{2} \qquad \text{(kips/stud)}$$

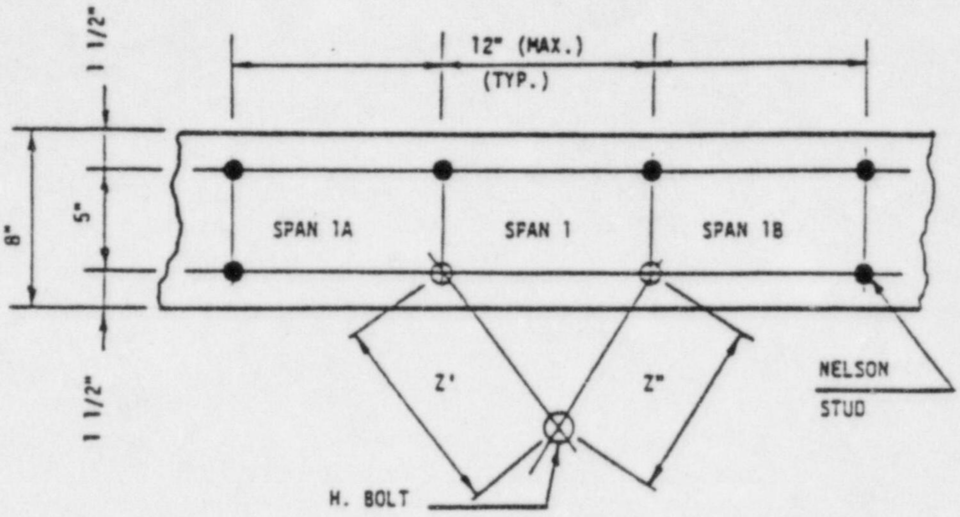
Step 5 Verification of embedded strip plate adequacy.

A) Location of Nelson studs of the embedded strip plate is known. Use the reduced allowable (working) capacity of the stud in tension in the interaction equation of paragraph 4.2 for all attachments within spans 1, 1A and 1B (see allowable loads on adjacent spans, PAGE 12).

B) Location of Nelson studs of the embedded strip plate is not known. Use the reduced allowable (working) capacity of the stud in tension in the interaction equation of paragraph 4.2 for all attachments located less than twelve inches along the plate strip from the Hilti bolt.



ALLOWABLE LOADS FOR ADJACENT SPANS



a) Z' & Z" are used here only for illustrative purposes

b) Z' < Z" and both Z' & Z" are bolt violations

c) The maximum capacity of the embedded strip plate, in particular span 1, is determined by calculating the allowable (working) capacity of the Nelson stud nearest to the Hilti bolt, (Z' in this case since Z' < Z").

d) If a load is to be placed on span 1A, the maximum capacity determined for span 1 may be used for span 1A provided that no other spacing violation exists for any other Nelson stud supporting span 1A. If another spacing violation does exist then choose the smallest Z dimension for any one of the four studs of span 1A to determine the load capacity by using the procedures outlines on the preceeding pages.

e) If a load is to be placed on span 1B, the maximum capacity of strip plate is determined by calculating the capacity of the Nelson stud located at 2" distance from Hilti Bolt, as illustrated above, provided that no other spacing violation exists for an other Nelson stud supporting span 1B. Follow the procedure as mentioned above in note d if another spacing violation exists.

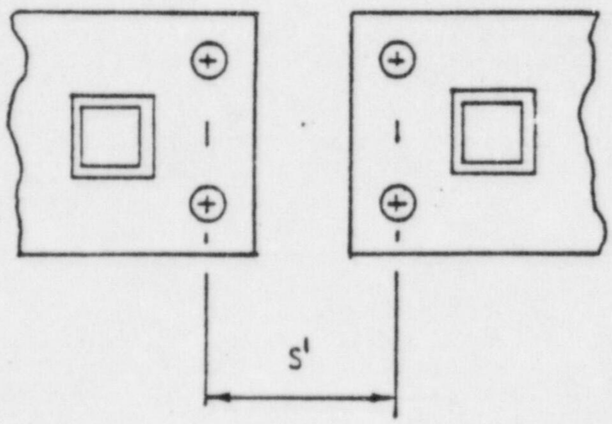


FIGURE A-4W-3

55-30

APPENDIX 5

DESIGN CRITERIA FOR
EMBEDDED LARGE STEEL PLATES

APPENDIX 5

DESIGN CRITERIA FOR EMBEDDED LARGE STEEL PLATES

1.0 DESCRIPTION

Embedded large steel plates are ASTM A36 steel plates, 3/4" thick connected to concrete walls and the under side of slabs by means of Nelson studs embedded in the concrete and welded to the plate. They are used to support hangers and other structural supports which are connected to the embedded plate by welding or by threaded Nelson studs. The design of the threaded Nelson studs and the welds at the connection to the embedded plate is the responsibility of the designer of the hanger or other structural support.

2.0 APPLICABLE REFERNECES

2.1 Manual of Steel Construction AISC 7th Edition

2.2 Design Data 10 - Embedment Properties of Headed Studs - TRW Nelson Division 2-77.

3.0 ALLOWABLE LOADS ON EMBEDDED LARGE STEEL PLATES

3.1 For design purposes each steel plate is divided into four different regions: Cantilever, Exterior, Exterior Corner, and Interior. There is an additional region called "Exterior Region Near Opening" if an opening in the steel plate exists. See Sheet A5-1. Designation of regions is as follows:

Area A; Interior Region

Area B; Exterior Region

Area C; Exterior Corner Region

Area D; Exterior Region Near Opening

Area E; Cantilever Region

3.2 Steel plate material is A-36 Nuclear Safety Related as defined on Drawing No. 2323-S-0786 for embedded plate details.

3.3 For allowable tension and shear loads at any location of each particular region of the steel plate see Sheet A5-2 through A5-4 and Sheet A5-8 through A5-10. No loading is permitted in the cantilever region except if special design is made for adequate load distribution.

3.4 Stiffners may be used between the attachment and the plate for load distribution in order to stay within the allowable loads defined on Sheet A5-2 through A5-4 and Sheet A5-8 through A5-10.

3.5 When moment is transmitted to the plate, the moment may be converted into a couple acting on the plate; the couple is calculated as the resultant tension and compression force of the distributed pressure acting on the plate due to the moment. The tension component of the couple and the direct tension load should be combined numerically. The resulting tension force and the simultaneous shear force should be used in conjuction with Sheets A5-2, A5-3, A5-4, A5-8, A5-9 and A5-10 in order to ensure that the plate is not overloaded. Other design methods may be used if proven by analysis.

3.6 Weld contours of adjacent attachments, including auxiliary steel, shall be separated by 12 inches minimum. See Sheet A5-5.

For examples, see Sheet A5-6 and Sheet A5-7 for pin and moment connections to the large steel plate, respectively.

3.7 a) Allowable load capacities for attachments smaller than 6"x 6" are shown on Sheets A5-2 through A5-4.

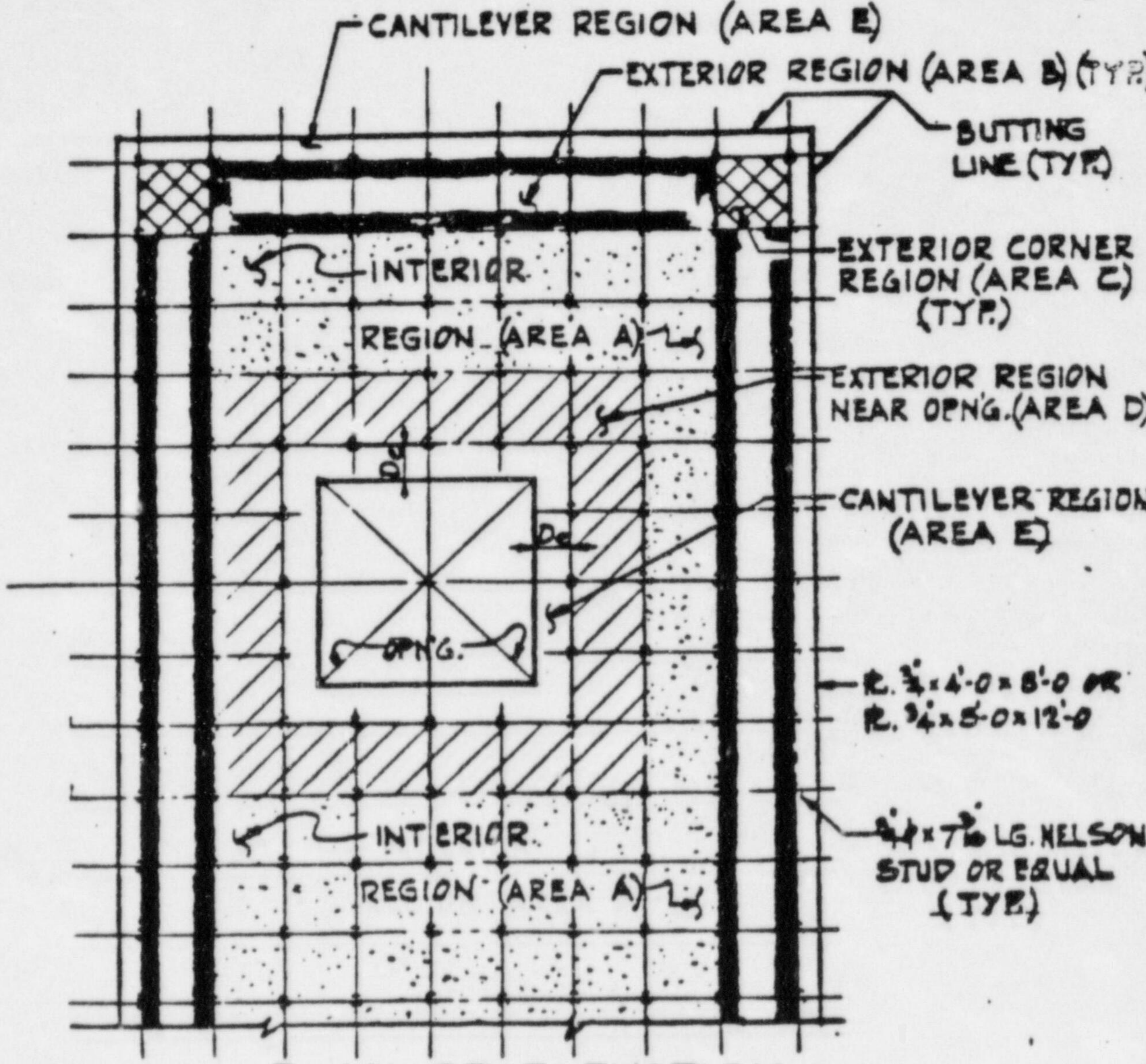
b) Allowable load capacities for attachments 6"x 6" and larger are shown on Sheets A5-8 through A5-10. Attachments should be welded all around.

Note: If the attachment is not square the smaller dimension of the attachment shall be at least 6".

3.8 For plate attachments larger than 16" x 16" the use of Sheets A5-8 through A5-10 may be too conservative. In these cases, the total tension and shear forces may be distributed to a few lumped force points along the tension welds. Each lumped force point should maintain a minimum of 12" from any adjacent lumped force point. The allowable load capacity shown on Sheets A5-2 through A5-4 may then be used to check each individual lumped force.

3.9 If the attachment is connected to more than one region of the large steel plate the smallest allowable load capacity of these regions should be used.

3.10 Attachment dimension refers to the dimension of the attachment at the interface with the large steel plate.



PLAN OR ELEVATION

FIG. 1

NOTES:

1. FOR ALLOWABLE LOAD CAPACITY AT ANY LOCATION OF:
   AREA A; INTERIOR REGION, SEE FIG. 2 AND FIG. 8.
   AREA B; EXTERIOR REGION, SEE FIG. 3 AND FIG. 9.
   AREA C; EXTERIOR CORNER REGION, SEE FIG. 3 AND FIG. 9.
   AREA D; EXTERIOR REGION NEAR OPN'G., SEE FIG. 4 AND FIG. 10.
2. THE DIMENSION "De" IS THE FREE EDGE DISTANCE AS SHOWN.
3. FOR LOCATION OF STEEL PLATES SEE DE/CD S-1643.
4. FOR NELSON STUD PATTERN SEE DE/CD S-1582

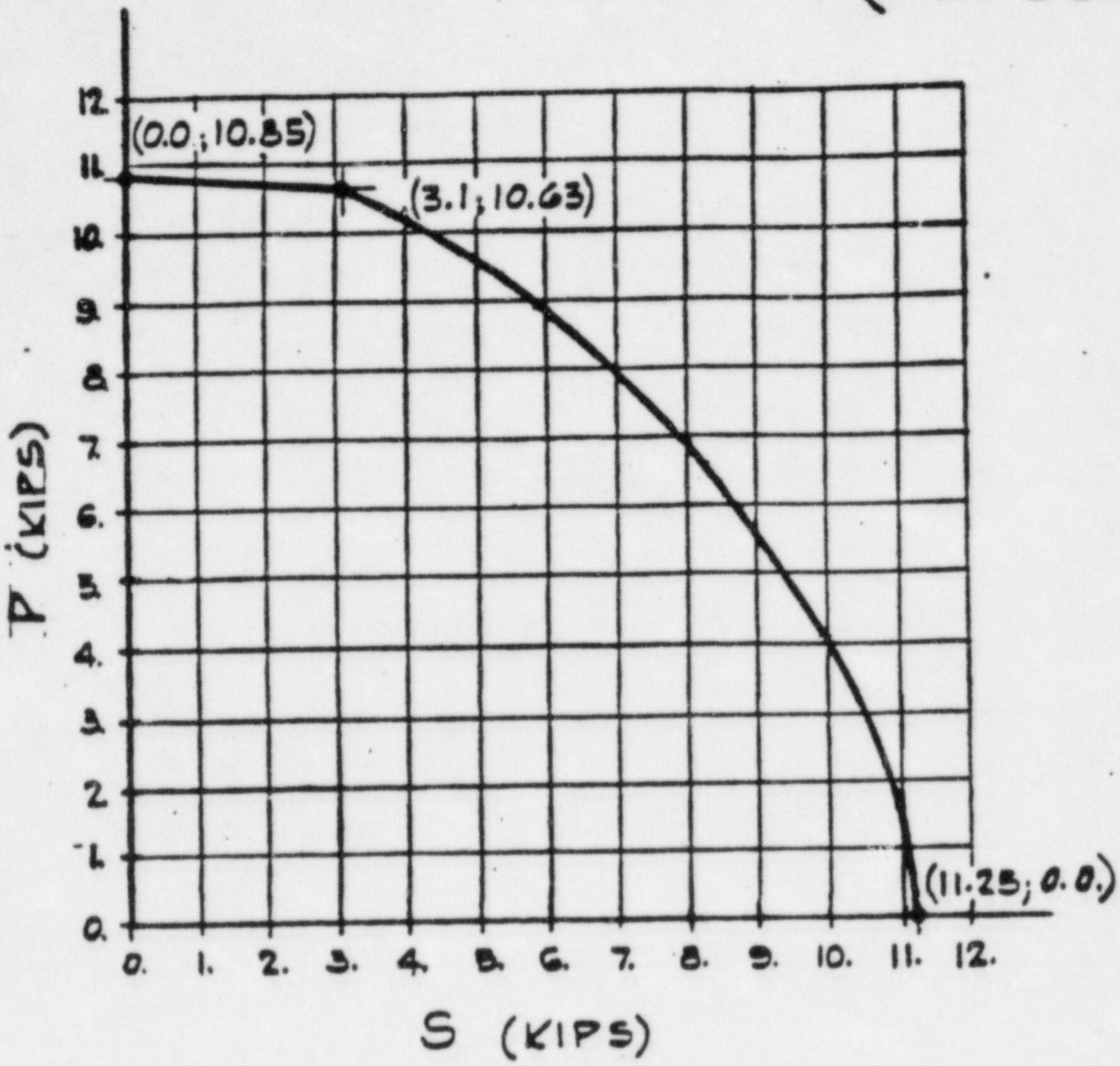
TUSI
COMANCHE PEAK

TYPICAL CONFIGURATION OF LARGE STEEL PLATES FOR LOADED REGION

Gibbs & Hill Inc.

2323

SH. A5-1



ALLOWABLE LOAD CAPACITY FOR COMBINED TENSION AND SHEAR - FOR ATTACHMENTS SMALLER THAN 6"x6"

INTERACTION DESIGN CURVE FOR LOADINGS ACTING AT INTERIOR REGION (AREA A)

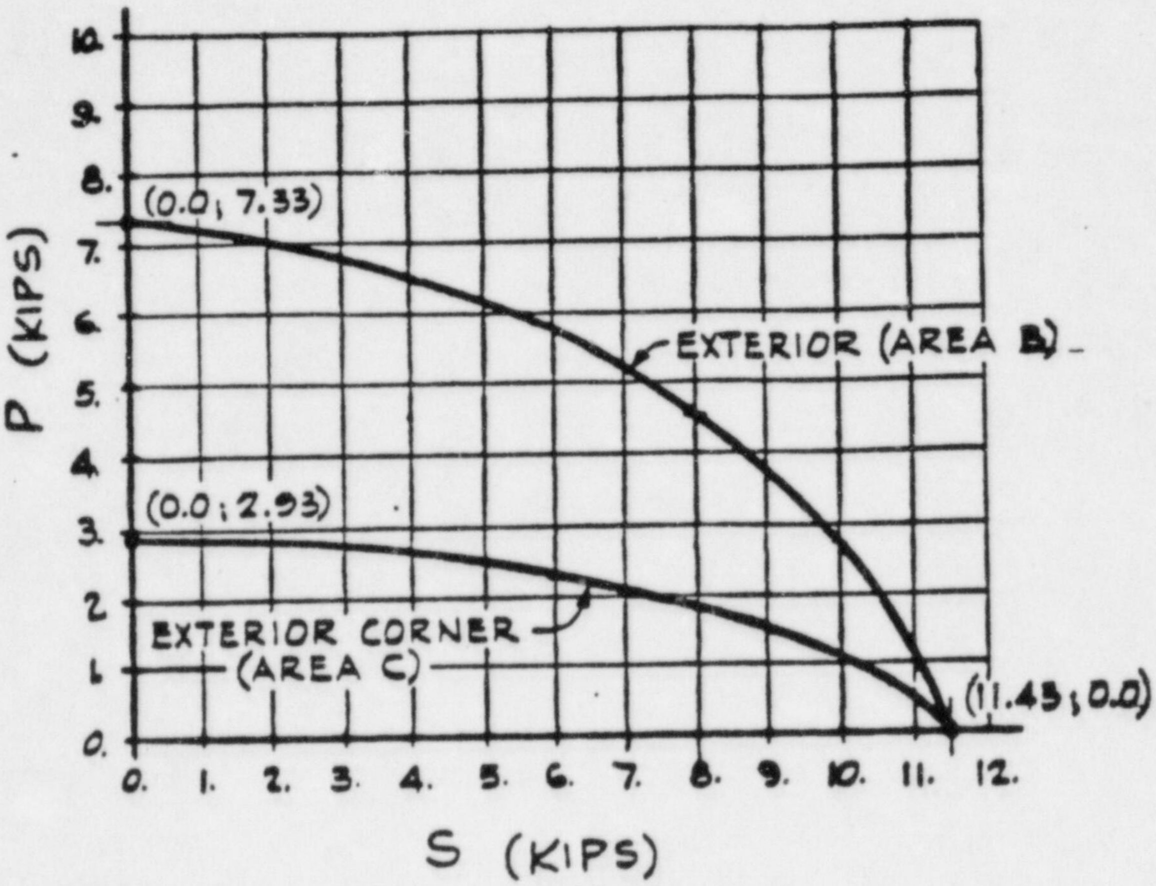
FIG. 2

NOTATION

P--- ACTUAL APPLIED TENSION LOAD*
S--- ACTUAL APPLIED SHEAR LOAD*

*SEE SECTION 3.5 OF THIS APPENDIX

TUSI COMANCHE PEAK	
ALLOWABLE LOADS ON LARGE STEEL PL FOR ATTACHMENTS SMALLER THAN 6"x6"	
Gibbs & Hill, Inc.	SCALE: —
2323	SH. A5-2



ALLOWABLE LOAD CAPACITY FOR COMBINED TENSION AND SHEAR - FOR ATTACHMENTS SMALLER THAN 6"x6"

INTERACTION DESIGN CURVE FOR LOADINGS ACTING AT EXTERIOR REGION (AREA B) & ACTING AT EXTERIOR CORNER REGION (AREA C)

FIG. 3

NOTATION

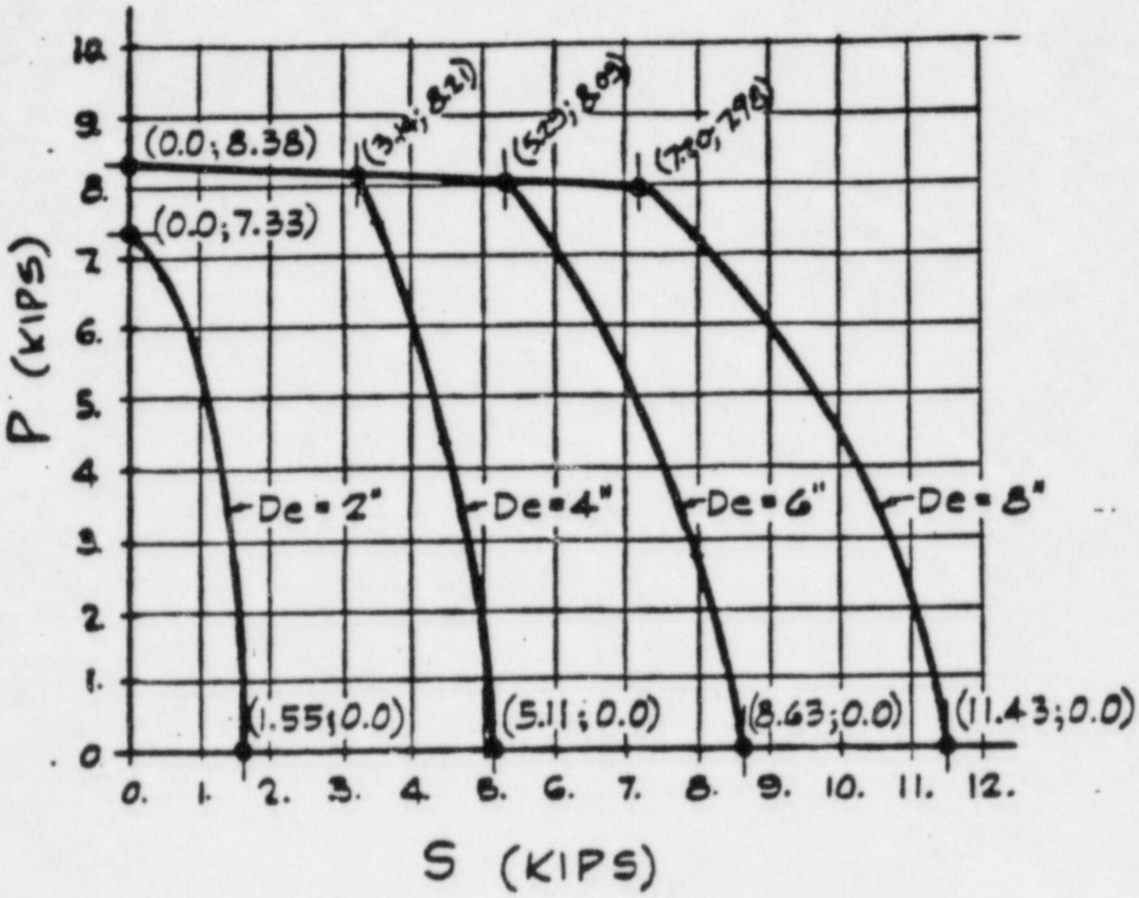
P---ACTUAL APPLIED TENSION LOAD*
S---ACTUAL APPLIED SHEAR LOAD*
* SEE SECTION 3.5 OF THIS APPENDIX.

TUSI
COMANCHE PEAK

ALLOWABLE LOADS ON LARGE STEEL PL FOR ATTACHMENTS SMALLER THAN 6"x6"

Gibbs & Hill, Inc.

--2323 | SH. A5-3



ALLOWABLE LOAD CAPACITY FOR COMBINED TENSION AND SHEAR - FOR ATTACHMENTS SMALLER THAN 6"x6"

INTERACTION DESIGN CURVE FOR LOADINGS ACTING AT EXTERIOR REGION NEAR OPNG. (AREA D) WITH D_e = 2", 4", 6" & 8"

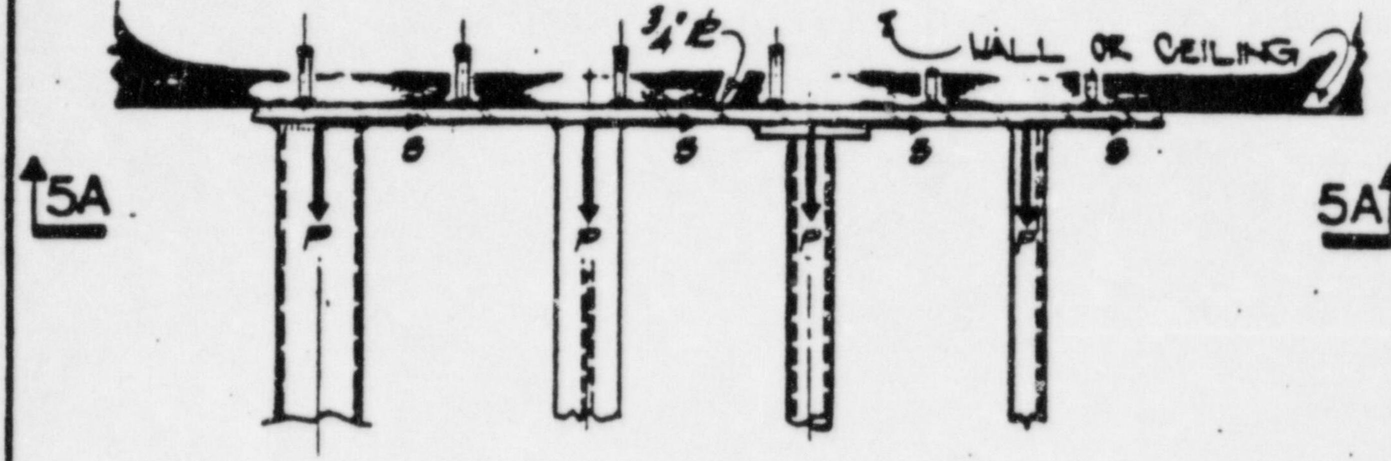
FIG. 4

NOTATION

P--- ACTUAL APPLIED TENSION LOAD*
S--- ACTUAL APPLIED SHEAR LOAD*

* SEE SECTION 3.5 OF THIS APPENDIX

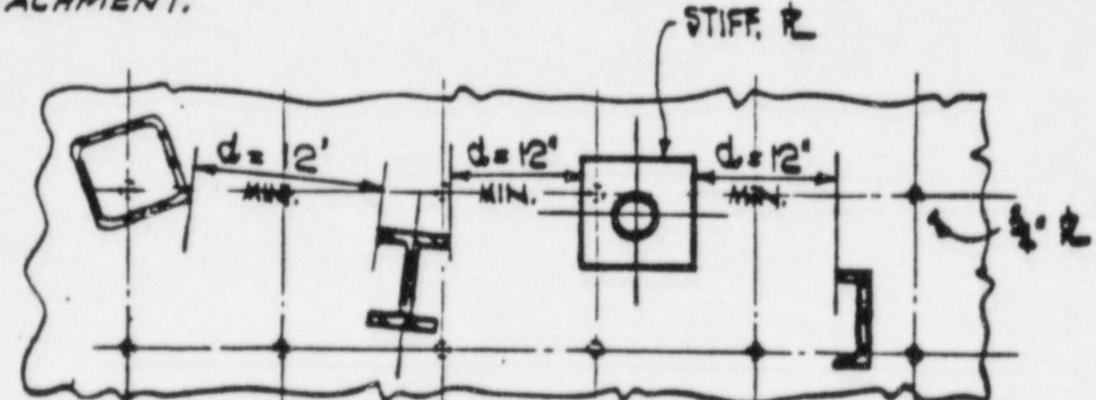
TUSI COMANCHE PEAK	
ALLOWABLE LOADS ON LARGE STEEL ℞ FOR ATTACHMENTS SMALLER THAN 6"x6"	
Gibbs & Hill, Inc.	
2323	SH. A5-4



PLAN OR ELEVATION

NOTE:

1. *FOR ANY SUPPORT WHERE ONE ATTACHMENT IS IN TENSION AND ONE ATTACHMENT IS IN COMPRESSION, THE MIN. 12" SEPARATION CRITERIA (d) REFERS TO THE DISTANCE BETWEEN THE WELD OF THE TENSION ATTACHMENT TO THE CENTER LINE OF THE COMPRESSION ATTACHMENT.*



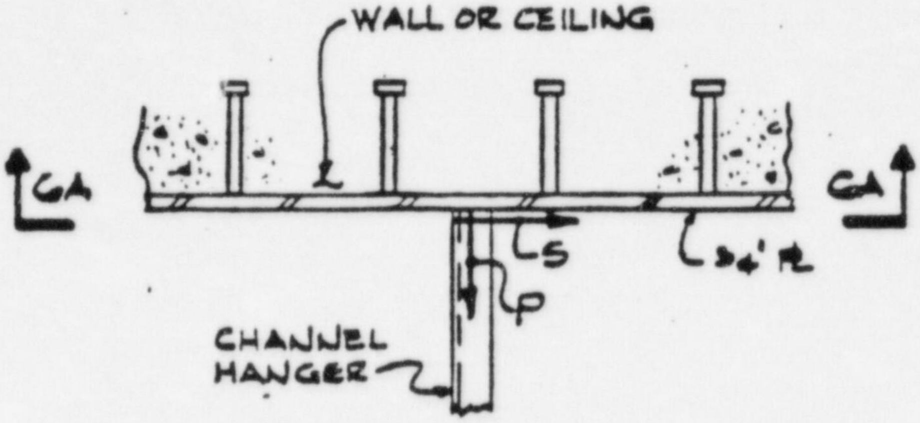
SECTION 5A-5A

FIGURE '5'

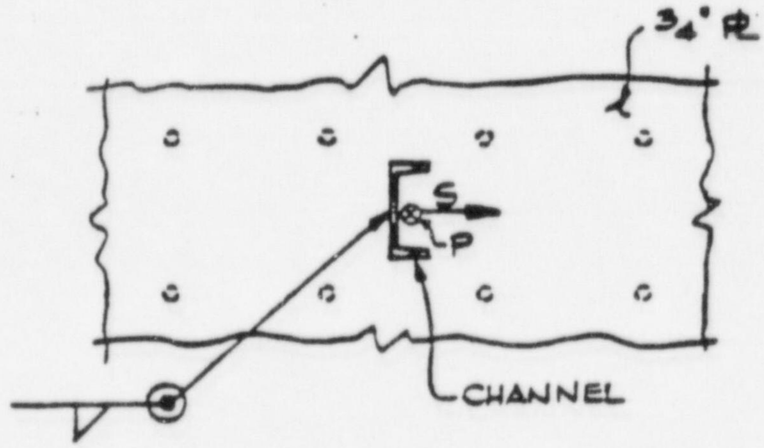
TUSI COMANCHE PEAK	
MINIMUM CLEAR DISTANCE (d) BETWEEN ADJACENT ATTACHMENTS	
GIBBS & HILL, INC.	SCALE: NONE
2323	SH. A5-5

EXAMPLE #1

HANGER AT EXTERIOR REGION (AREA B) OF LARGE STEEL PLATE



PLAN OR ELEVATION



SECTION 6A-6A

FIGURE 6

QUESTION : IF P = 6.0^K AND S = 3.0^K IS THE LARGE STEEL PL ADEQUATE?

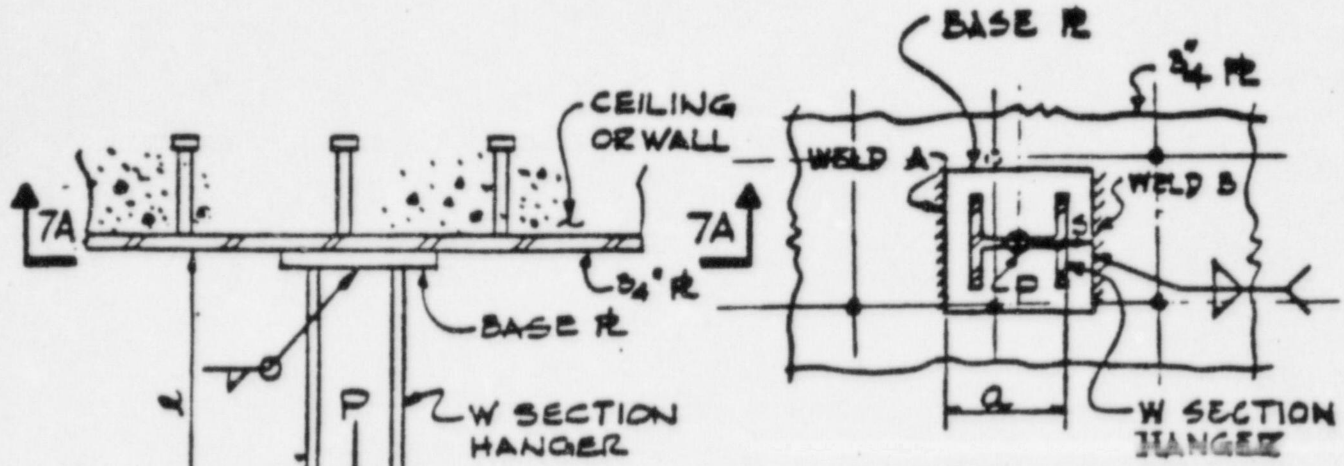
SOLUTION : FROM FIGURE 3 P=6.0^K
$S_{ALLOWABLE}$ = 5.5^K > 3.0^K.
O.K.

ANS : PL IS ADEQUATE.

TUSI COMANCHE PEAK	
PIN CONNECTION TO LARGE STEEL PLATE	
GIBBS & HILL INC.	SCALE - NONE
2323	SH. A5-6

EXAMPLE #2

HANGER AT INTERIOR REGION (AREA A) OF LARGE STEEL PLATE



PLAN OR ELEVATION

SECTION 7A-7A

FIGURE 7

QUESTION : IF $S = 2.0^K$, $P = 2.0^K$, $\ell = 3'\text{-}6"$ AND $a = 10"$ IS THE LARGE STEEL PLATE ADEQUATE ?

SOLUTION: AT WELD A:

SHEAR: $S_A = S/2 = 2.0/2 = 1.0^K$

TENSION DUE TO CANTILEVER MOMENT : P_A'

$$P_A' = \frac{S\ell}{a} = \frac{(2)(3.5)(12)}{10} = 8.4^K$$

TENSION DUE TO DIRECT TENSION LOAD : P_A''

$P_A'' = P/2 = 1.0^K$

RESULTANT TENSION, $\Sigma P_A^{*} = P_A' + P_A'' = 8.4 + 1 = 9.4^K$

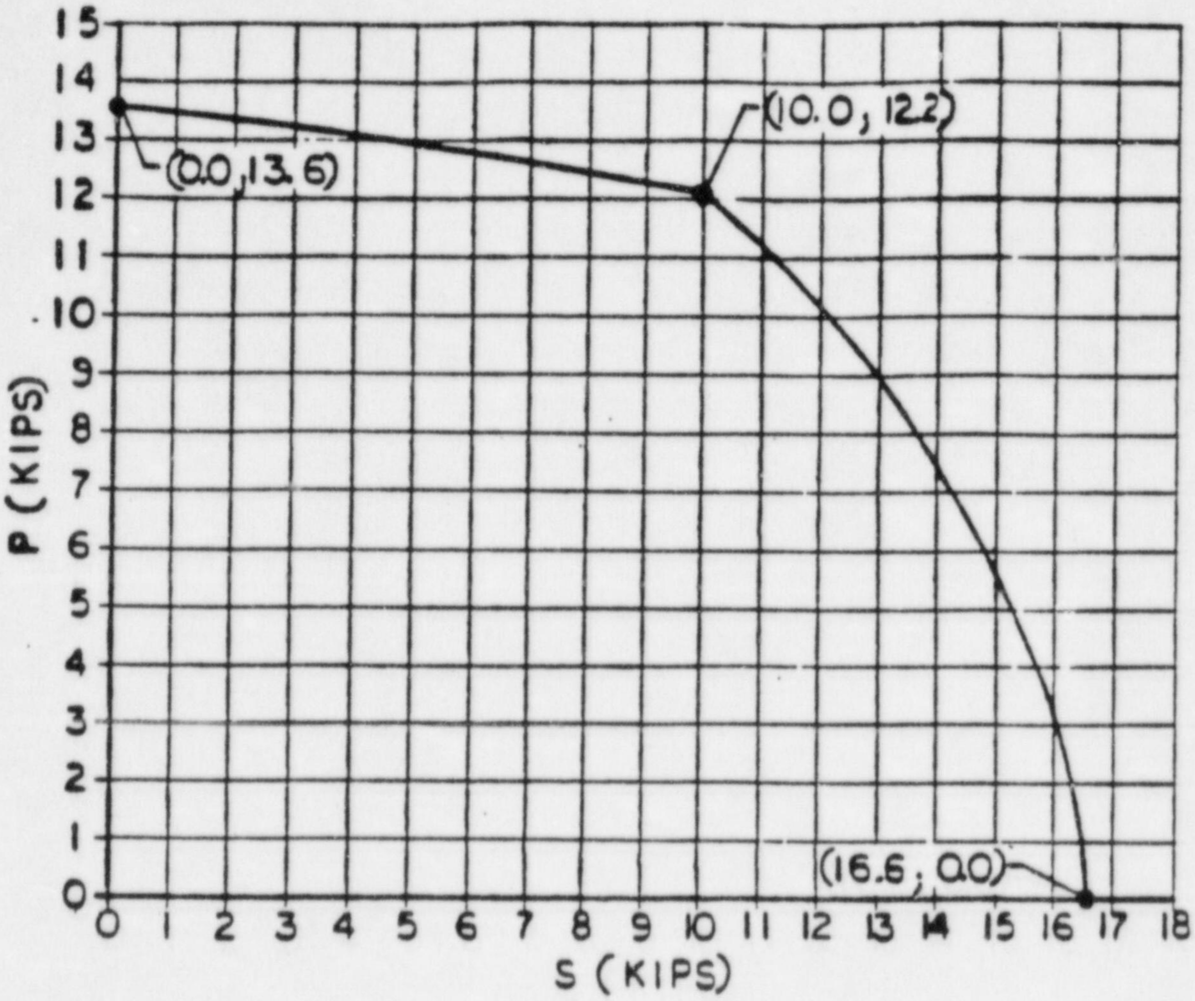
FROM FIGURE 2, SHEET 4

S *allowable* $= 5.4^K > 2.0^K$ O.K.

ANS: PL IS ADEQUATE

* : IF $P_A' < P_A''$ THEN THE RESULTANT TENSION, $\Sigma P_A = P + P_A'$

TUSI COMANCHE PEAK	
MOMENT CONNECTION TO LARGE STEEL PLATE	
GIBBS & HILL Inc.	SCALE - NONE
2323	SH. A5-7



ALLOWABLE LOAD CAPACITY FOR COMBINED TENSION AND SHEAR - FOR ATTACHMENTS 6"x 6" OR LARGER

INTERACTION DESIGN CURVE FOR LOADINGS ACTING AT INTERIOR REGION (AREA A)

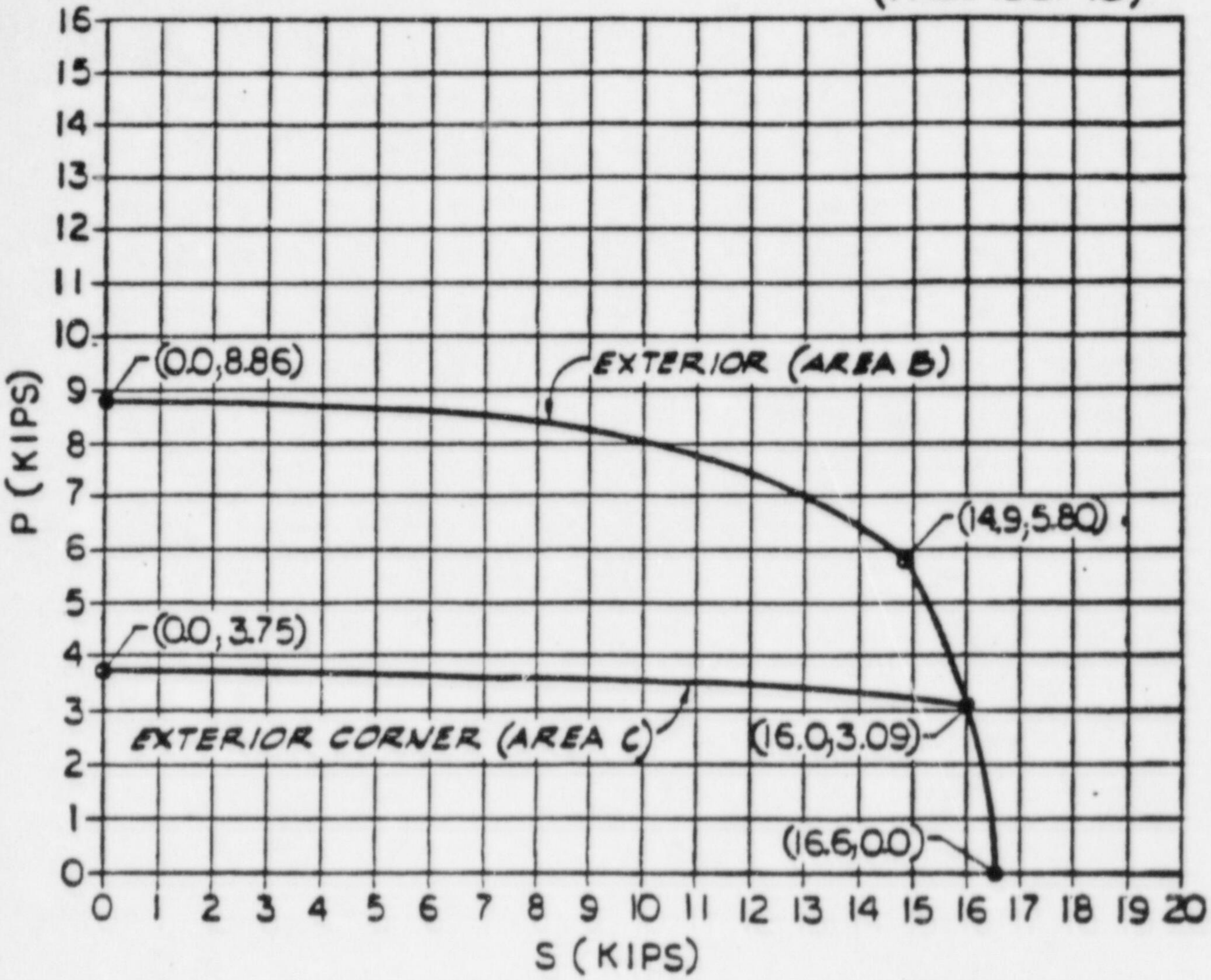
FIG. 8

NOTATION

*P___ACTUAL APPLIED TENSION LOAD**
*S___ACTUAL APPLIED SHEAR LOAD**

* *SEE SECTION 3.5 OF THIS APPENDIX*

TUSI COMANCHE PEAK	
ALLOWABLE LOADS ON LARGE STEEL PL FOR ATTACHMENTS OF 6"x6" AND LARGER	
Gibbs & Hill, Inc.	NONE
-- 2323	SH. A5-8



ALLOWABLE LOAD CAPACITY FOR COMBINED TENSION AND SHEAR - FOR ATTACHMENTS 6"x6" OR LARGER

INTERACTION DESIGN CURVE FOR LOADINGS ACTING AT EXTERIOR REGION (AREA B) AND ACTING AT EXTERIOR CORNER REGION (AREA C)

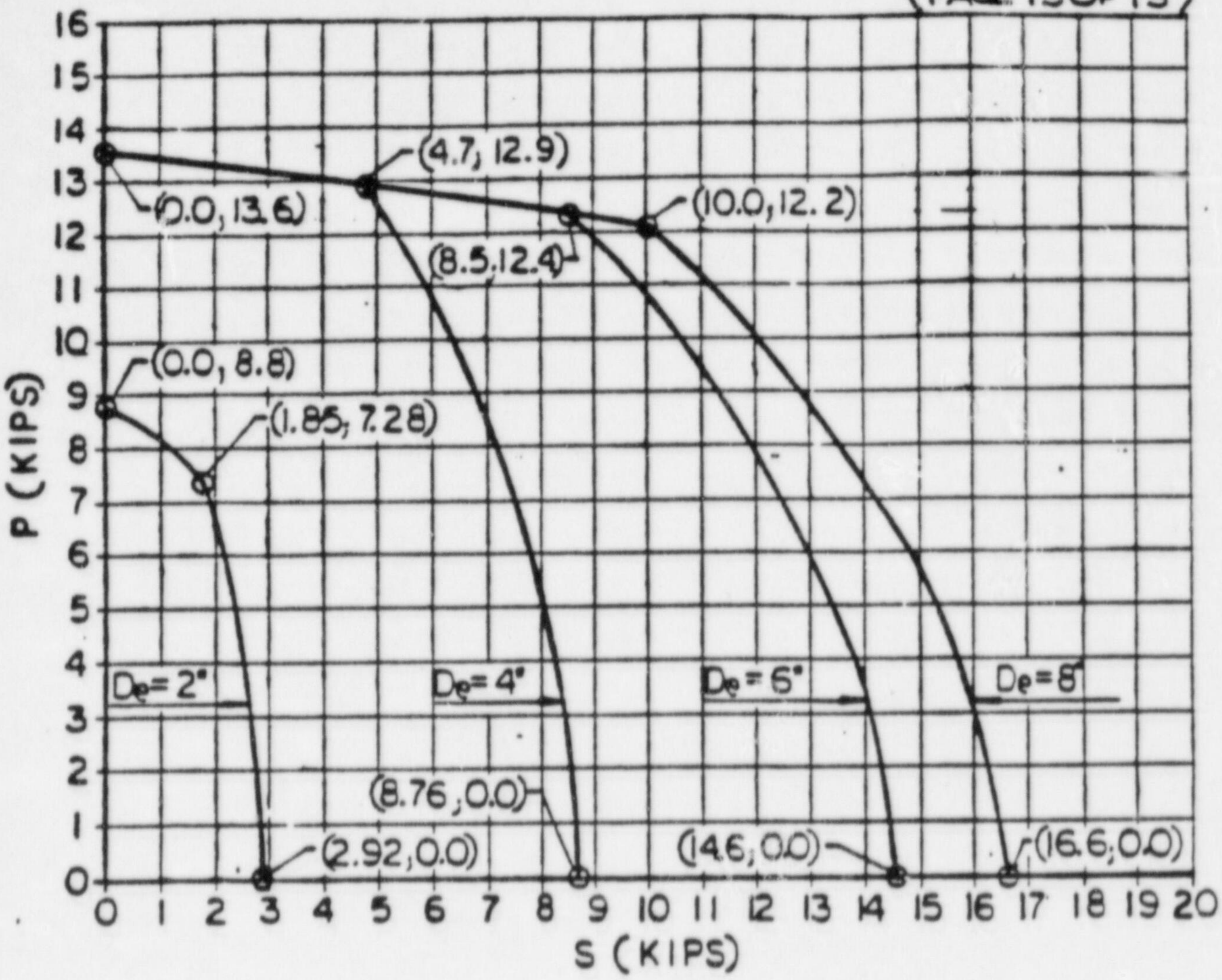
FIG.9

NOTATION

*P--- ACTUAL APPLIED TENSION LOAD**
*S--- ACTUAL APPLIED SHEAR LOAD**

** SEE SECTION 3.5 OF THIS APPENDIX*

TUSI COMANCHE PEAK	
ALLOWABLE LOADS ON LARGE STEEL PL FOR ATTACHMENTS OF 6"x6" AND LARGER	
Gibbs & Hill, Inc.	NONE
--2323	SH. A5-9



ALLOWABLE LOAD CAPACITY FOR COMBINED TENSION AND SHEAR - FOR ATTACHMENTS 6"x 6" OR LARGER

INTERACTION DESIGN CURVE FOR LOADINGS ACTING AT EXTERIOR REGION NEAR OPNG. (AREA D) WITH De = 2", 4", 6" & 8"

FIG. 10

NOTATION

*P--- ACTUAL APPLIED TENSION LOAD**
*S--- ACTUAL APPLIED SHEAR LOAD**

** SEE SECTION 3.5 OF THIS APPENDIX*

TUSI COMANCHE PEAK	
ALLOWABLE LOADS ON LARGE STEEL PL FOR ATTACHMENTS OF 6"x6" AND LARGER	
Gibbs & Hill Inc.	NONE
--2323	SH. A5-10

SS-30

APPENDIX 5W

DESIGN CRITERIA FOR EMBEDDED LARGE STEEL PLATES

(ALTERNATE)

(Attachment to Westinghouse Document No. 10923

Transmitted with WPT- 8031 Dated 9/10/85)

APPENDIX 5W

DESIGN CRITERIA FOR EMBEDDED LARGE STEEL PLATES

AUGUST 30, 1985

AUTHORS: [signature]
R. S. Orr

[signature]
R. Condrac

VERIFIER: [signature]
H. P. Bonnet

APPROVED: [signature]
M. Mahlab

1.0 DESCRIPTION

Embedded large steel plates are ASTM A36 steel plates, 3/4" thick connected to concrete walls and the under side of slabs by means of Nelson studs embedded in the concrete and welded to the plate. They are used to support hangers and other structural supports which are connected to the embedded plate by welding or by threaded Nelson studs. The design of the threaded Nelson studs and the welds at the connection to the embedded plate is the responsibility of the designer of the hanger or other structural support.

2.0 APPLICABLE REFERENCES

2.1 Manual of Steel Construction AISC 7th Edition.

2.2 Design Data 10 - Embedment Properties of Headed Studs - TRW Nelson Division 2-77.

3.0 ALLOWABLE LOADS ON EMBEDDED LARGE STEEL PLATES

3.1 For design purposes each steel plate is divided into different regions: Cantilever, Interior and "Exterior Region Near Opening", if an opening in the steel plate exists. (See Fig. A5W-1). Designation of regions is as follows:

Area A; Interior Region
Area D; Exterior Region Near Opening
Area E; Cantilever Region

3.2 Steel plate material is A-36 Nuclear Safety Related as defined on Drawing No. 2323-S-0786 for embedded plate details.

3.3 Loads on attachments on the interior region (Area A) shall be evaluated by calculating stud tension and shear loads using the following algorithms and evaluating these stud loads using the stud interaction equation given below. Loads on the attachment are defined as F_x, F_y, F_z, M_x, M_y, M_z with the z axis taken normal to the embedment plate. 'a' shall be taken as the smaller attachment dimension but shall not be taken greater than 6". The absolute value of the maximum load shall be used.

Stud tension: $T_s = \frac{20 - a}{20} F_z + [\frac{M_x^2 + M_y^2}{a + 2}]^{1/2}$

Stud shear: $V_s = \frac{10}{12 + a} [(F_x + 0.05 M_z)^2 + (F_y + .05 M_z)^2]^{1/2}$

Allowable stud loads: $(\frac{T_s}{11.9})^{5/3} + (\frac{V_s}{11.65})^{5/3} \leq 1$

3.4 No loading is permitted in the cantilever region except if special design is made for adequate load distribution.

3.5 Attachment to the exterior region near openings is only permitted when the edge distance, D_e, from the face of the opening to the first stud line is known such that the extent of the cantilever region is defined. If $D_e \geq 4"$ loads on attachments may be evaluated in accordance with paragraph 3.3. If $D_e < 4"$ stud tension and shear load shall be evaluated in accordance with paragraph 3.3 and these loads shall be evaluated using the following interaction equation.

$$(\frac{T_s}{2.975 D_e})^{5/3} + (\frac{V_s}{11.65})^{5/3} \leq 1$$

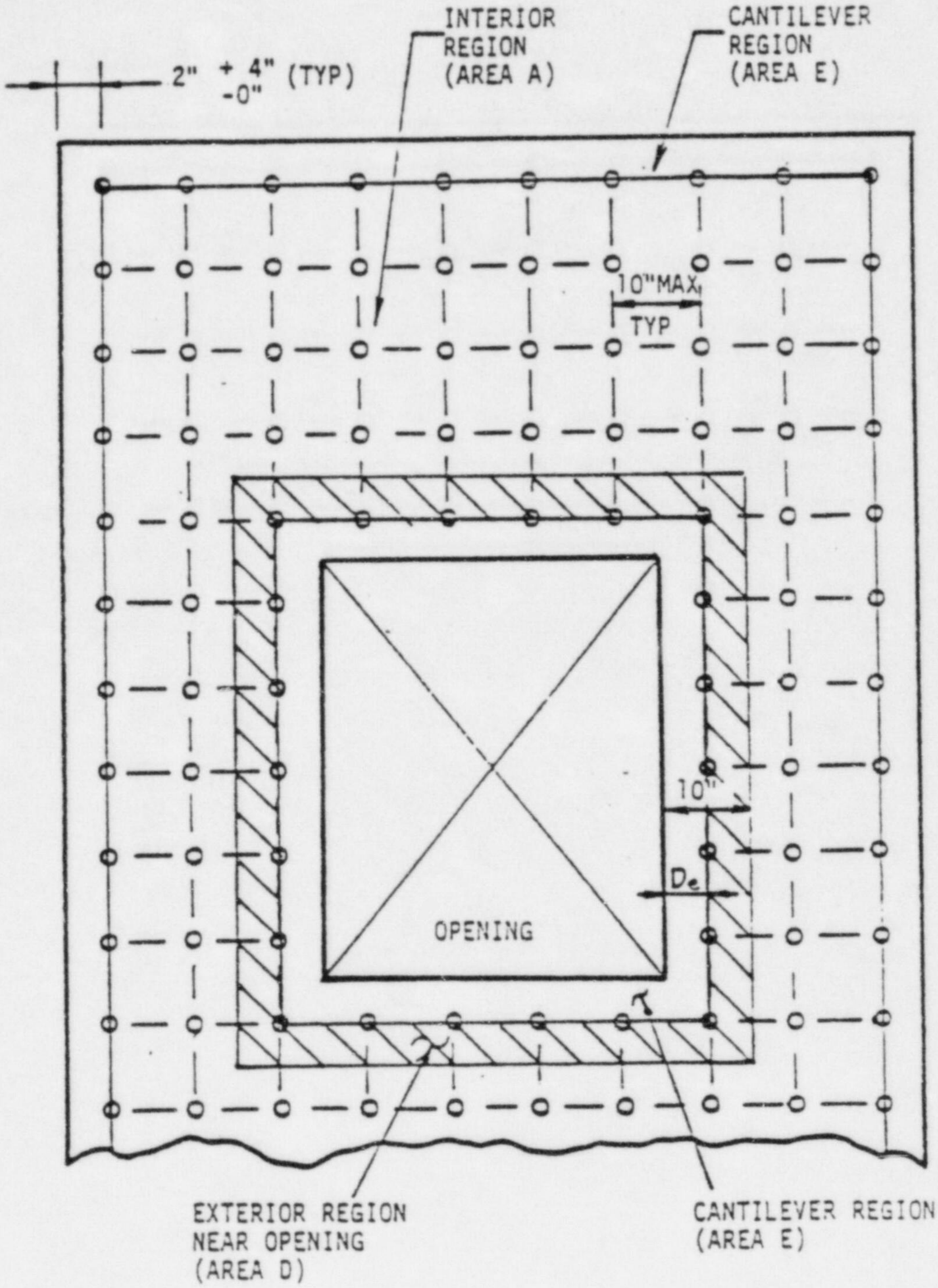
3.6 Stiffeners may be used between the attachment and the plate in order to increase the effective attachment size to stay within the allowable loads defined in paragraphs 3.3 and 3.5.

3.7 Weld contours of adjacent attachments, including auxiliary steel, shall be separated by 12 inches minimum. (See Fig. A5W-2). This minimum spacing is also applicable across butting lines between adjacent plates.

3.8 For plate attachments larger than 16" x 16" the use of paragraphs 3.3 and 3.5 may be too conservative. In these cases, the total tension and shear forces may be distributed to a few lumped force points along the tension welds. Each lumped force point should maintain a minimum of 12" from any adjacent lumped force point. The allowable load capacity of paragraphs 3.3 and 3.5 may then be used to check each individual lumped force.

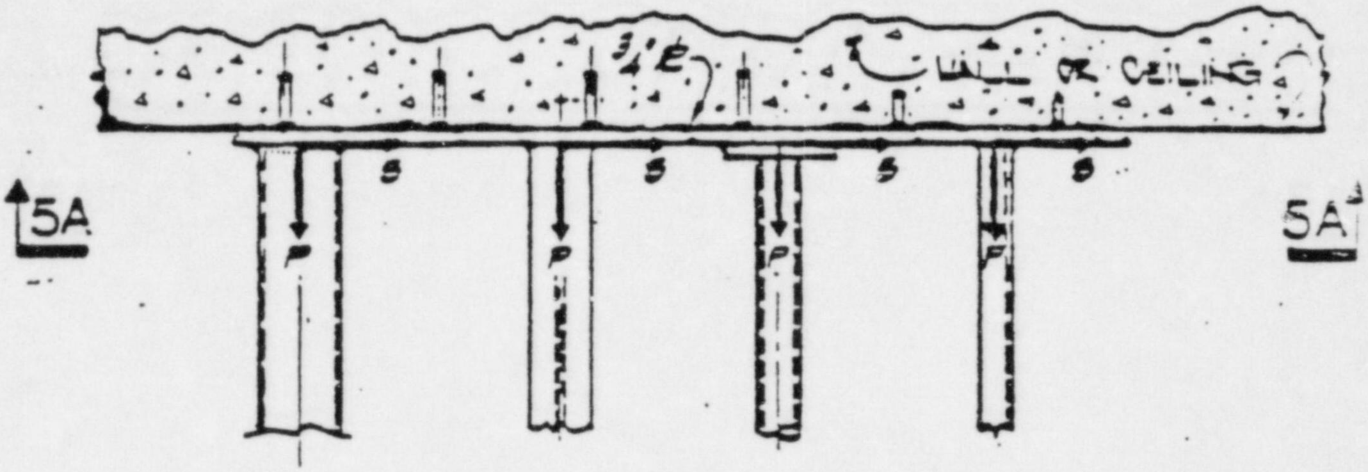
3.9 If the attachment is connected to more than one region of the large steel plate the smaller allowable load capacity of these regions should be used.

3.10 Attachment dimension refers to the dimension of the attachment at the interface with the large steel plate. If the attachment consists of a structural member and baseplate welded to the sheet plate, the dimension a shall be the distance from the compression flange of the structural member to the tension weld of the base plate to the sheet plate (see Fig. A5W-3)



PLAN OR ELEVATION VIEW OF SHEET EMBEDDED PLATE

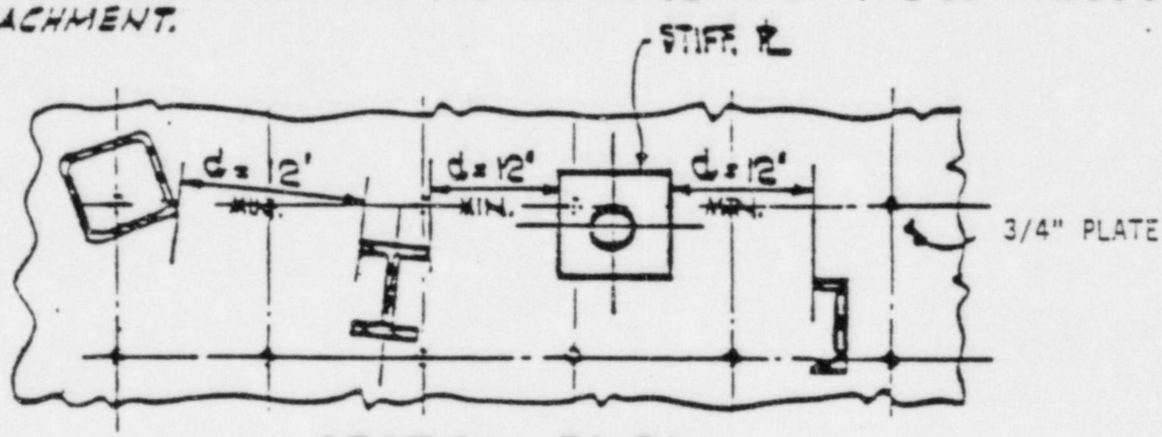
FIG. A5W-1



PLAN OR ELEVATION

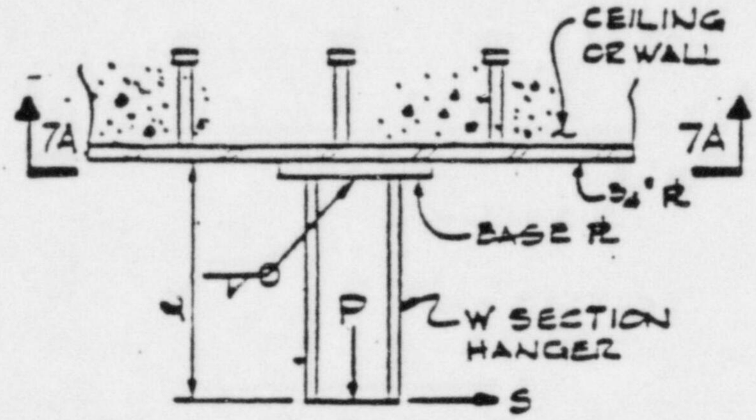
NOTE:

1. FOR ANY SUPPORT WHERE ONE ATTACHMENT IS IN TENSION AND ONE ATTACHMENT IS IN COMPRESSION, THE MIN. 12' SEPARATION CRITERIA (d) REFERS TO THE DISTANCE BETWEEN THE WELD OF THE TENSION ATTACHMENT TO THE CENTER LINE OF THE COMPRESSION ATTACHMENT.

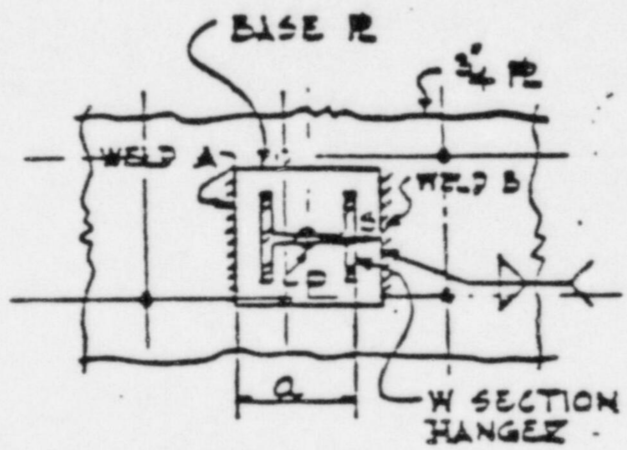


SECTION 5A-5A

HANGER AT INTERIOR REGION (AREA A) OF LARGE STEEL PLATE



PLAN OR ELEVATION



SECTION 7A-7A

FIGURE A5W-3

55-30

APPENDIX 6

ALLOWABLE LOAD CRITERIA FOR
1-1/2" DIAMETER - A193 GROUTED IN
ANCHOR BOLTS

ALLOWABLE LOAD CRITERIA FOR 1-1/2" DIAMETER - A193 GROUTED-IN ANCHOR BOLTS

For a single grout-in bolt installed in accordance with procedure set forth in CP-EI-13.0-3-Rev.1, allowable load criteria is as follows:

1. Allowable Tensile Capacity

   a. Ultimate load condition - 105 Kips

   b. Working load condition - 66 Kips

2. Allowable Shear Capacity

   a. Ultimate load condition - 69 Kips

   b. Working load condition - 34.5 Kips

3. Combined Tension and Shear

   a. Ultimate load condition

   $$\frac{T}{75\ \text{Ksi}} + \frac{V}{49\ \text{Ksi}} \leq 1.405\ \text{in.}^2$$ (Tensile stress area of 1-1/2" Diameter - A193 bolt)

   b. Working load condition

   $$\frac{T}{47\ \text{Ksi}} + \frac{V}{24.5\ \text{Ksi}} \leq 1.405\ \text{in.}^2$$

Use allowables given for ultimate load condition when designing for emergency/faulted (service level C&D) loads and when design is based on normal/upset (service level A&B) loads use allowables given for working load condition.

The above criteria can be used for a group of 4 bolts and 6 bolts in a 2'-9" min. concrete thickness, provided a minimum spacing of 14 in. for 4-bolt pattern and 18 in. for 6 bolt pattern is maintained.

In the event of, a) overlapping due to another anchor of a near-by support b) edge distance effect due to proximity of opening etc. above criteria cannot be applied directly. Such situations should then be independently examined on a case by case basis.

SPEC TUGCO(2)

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GIBBS & HILL, INC.

304

COMANCHE PEAK STEAM ELECTRIC STATION
DESIGN CHANGE AUTHORIZATION

(WILL) (~~WILL NOT~~) BE INCORPORATED IN DESIGN DOCUMENT   DCA NO. 15,338 R-1

1. SAFETY RELATED DOCUMENT: XX YES ____ NO

FOR OFFICE AND ENGINEERING USE ONLY

2. ORIGINATOR: CPPE XX ORIGINAL DESIGNER ____

3. DESCRIPTION:

A. APPLICABLE SPEC/DWG/DOCUMENT 2323-SS-30 REV. 0

B. DETAILS THIS REVISION VOIDS AND SUPERSEDES DCA-15,338 R-0

Add Appendix 6, "Allowable Load Criteria for 1-1/2"Ø - A193 Grouted in Anchor Bolts", to the referenced specification.

35-1195
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JAN 12 1984
DOCUMENT CONTROL

4. SUPPORTING DOCUMENTATION:

GTN-57677, ~~GTN-61623~~, GTN-62137 (PP 1/16/84)

Deleted Page 2 of Revision "0" of this DCA per telecon between E. L. Bezkor and P. Patel on 1-12-84.

5. APPROVAL SIGNATURES: PP/ccp

A. ORIGINATOR: Pravin Patel DATE 1-12-84

B. DESIGN REPRESENTATIVE: [signature] DATE 1-12-84

C. DESIGN REVIEW PRIOR TO ISSUE: [signature] DATE 1/12/84

6. VENDOR RELATED CHANGE X NO ____ YES: P.O. NUMBER ____

7. STANDARD DISTRIBUTION:

ARMS (ORIGINAL)	(1)	B. F. JONES-PROCUREMENT	(2)
QUALITY ENGINEERING	(1)		
DCTG FOR ORIG. DESIGN	(1)		
TS FOR ORIG. DESIGN	(1)		
PSE	(1)		
CIVIL ENGINEERING	(1)		

DCA FORM 9-83

SPECIFICATION 2323-SS-30

APPENDIX 6

ALLOWABLE LOAD CRITERIA FOR 1½"φ - A 193 GROUTED-IN ANCHOR BOLTS

For a single grout-in bolt installed in accordance with procedure set forth in CP-EI-13.0-3-Rev.1, allowable load criteria is as follows:

1. Allowable Tensile Capacity -
   a) Ultimate load condition - 105 Kips
   b) Working load condition - 66 Kips

2. Allowable Shear Capacity
   a) Ultimate load condition - 69 Kips
   b) Working load condition - 34.5 Kips

3. Combined Tension & Shear
   a) Ultimate load condition

$$\frac{T}{75Ksi} + \frac{V}{49Ksi} \leq 1.405 \text{ in.}^2$$ (Tensile stress area of 1½φ-A193 bolt)

   b) Working load condition

$$\frac{T}{47Ksi} + \frac{V}{24.5Ksi} \leq 1.405 \text{ in.}^2$$

Use allowables given for ultimate load condition when designing for emergency/faulted (service level C&D) loads and when design is based on normal/upset (service level A&B) loads use allowables given for working load condition.

The above criteria can be used for a group of 4 bolts and 6 bolts in a 2'-9" min. concrete thickness, provided a minimum spacing of 14 in. for 4-bolt pattern and 18 in. for 6 bolt pattern is maintained.

In the event of, a) overlapping due to another anchor of a near-by support b) edge distance effect due to proximity of opening etc. above criteria cannot be applied directly. Such situations should then be independently examined on a case by case basis.

CHANGE INDEX: OEI
II
III XX

COMANCHE PEAK STEAM ELECTRIC STATION
DESIGN CHANGE AUTHORIZATION

(WILL) (XXXX/XXX) BE INCORPORATED IN DESIGN DOCUMENT   DCA NO. 15,338

1. SAFETY RELATED DOCUMENT: XX YES · NO

2. ORIGINATOR: CPPE XX ORIGINAL DESIGNER

3. DESCRIPTION:

A. APPLICABLE SPEC/DWG/DOCUMENT 2323-SS-30 REV. 0

B. DETAILS 1. Add sheet 2 of 3 hereof to Appendix 3 of the referenced specification.

2. Add Appendix 6, "Allowable Load Criteria for 1 1/2"Ø - A193 Grouted in Anchor Bolts", to the referenced specification.

JOB NO. 35-1195
RECEIVED
DEC 10 1982
RECEIVED

4. SUPPORTING DOCUMENTATION:

GTN-57677, GTN-61623, GTN-62137

5. APPROVAL SIGNATURES: JCG/sgf 12-8-82

A. ORIGINATOR: J. C. Gilbreth DATE 12-8-82

B. DESIGN REPRESENTATIVE: C R Hooton DATE 12-8-82

6. VENDOR TRANSMITTAL REQUIRED: YES XX NO

7. STANDARD DISTRIBUTION:

ARMS (Original) (1) B.F. Jones-Procurement (2)
Quality Engineering (1)
TS for Orig. Design (1)
Westinghouse-Site (1)
Civil Engineering (1)

DCA FORM 11-80
Admin. Rev 7-82

SPECIFICATION 2323-SS-30

APPENDIX 6

ALLOWABLE LOAD CRITERIA FOR $1\frac{1}{2}$"φ - A 193 GROUTED-IN ANCHOR BOLTS

$2\frac{1}{2}$" φ & UNDE
GRADE B7: F_y 105 ksi | F_u 125.

For a single grout-in bolt installed in accordance with procedure set forth in CP-EI-13.0-3-Rev.1, allowable load criteria is as follows:

1. Allowabl Tensile Capacity -
   a) Ultimate load condition - 105 Kips
   b) Working load condition - 66 Kips
2. Allowable Shear Capacity
   a) Ultimate load condition - 69 Kips
   b) Working load condition - 34.5 Kips
3. Combined Tension & Shear
   a) Ultimate load condition

$$\frac{T}{{}^{*}75\text{Ksi}} + \frac{V}{49\text{Ksi}} \leq 1.405 \text{ in.}^2$$ (Tensile stress area of 1½Ø-A193 b

   b) Working load condition

$$\frac{T}{{}^{*}47\text{Ksi}} + \frac{V}{24.5\text{Ksi}} \leq 1.405 \text{ in.}^2$$

*SAME AS FOR A-320

Use allowables given for ultimate load condition when designing for emergency/faulted (service level C&D) loads and when design is based on normal/upset (service level A&B) loads use allowables given for working load condition.

The above criteria can be used for a group of 4 bolts and 6 bolts in a 2'-9" min. concrete thickness, provided a minimum spacing of 14 in. for 4-bolt pattern and 18 in. for 6 bolt pattern is maintained.

In the event of, a) overlapping due to another anchor of a near-by support b) edge distance effect due to proximity of opening etc. above criteria cannot be applied directly. Such situations should then be independently examined on a case by case basis.

APPENDIX 3

(DELETED)

DATA TRANSFERRED TO APPENDIX 2

APPENDIX 4

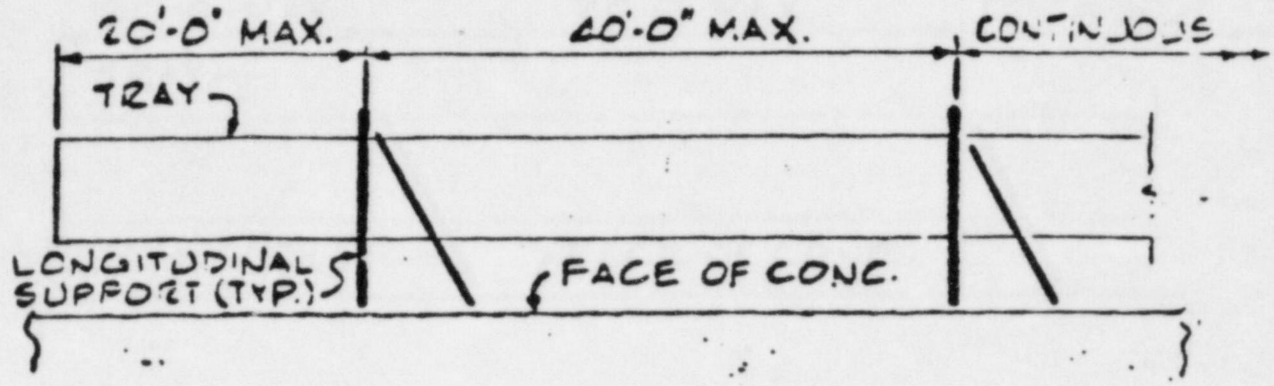
Maximum Longitudinal

Cable Tray Support Span

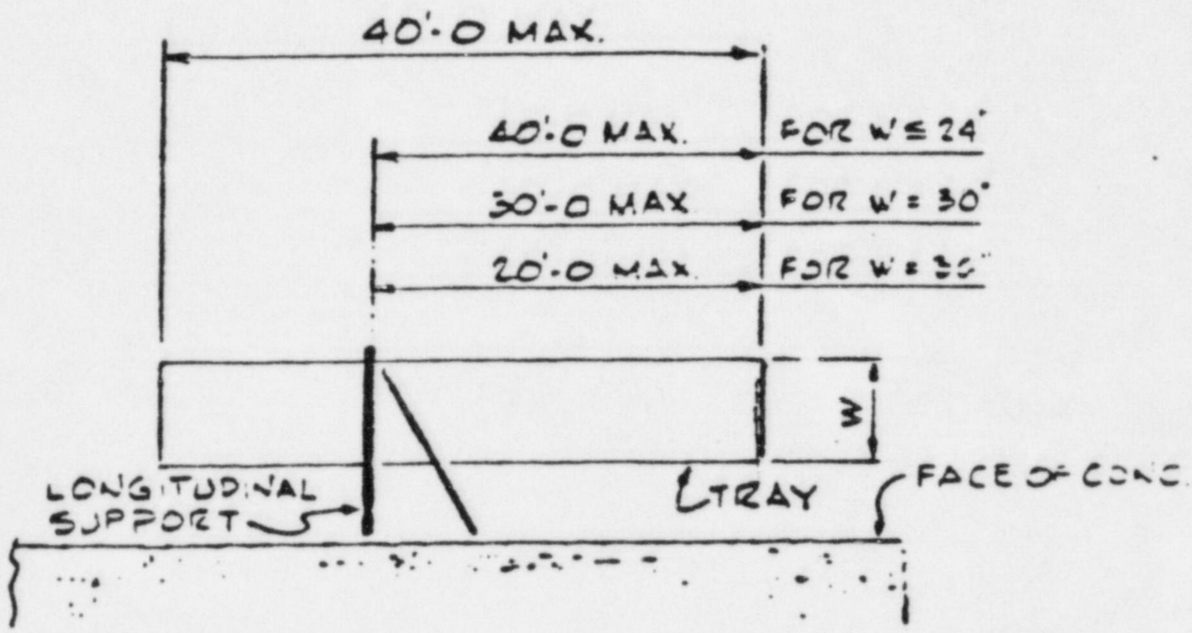
Note: This is a Gibbs & Hill document incorporated in the design criteria without any changes

STRAIGHT RUN

SUPPORTS FOR LONGITUDINAL LOADING



PLAN



PLAN

TUSI

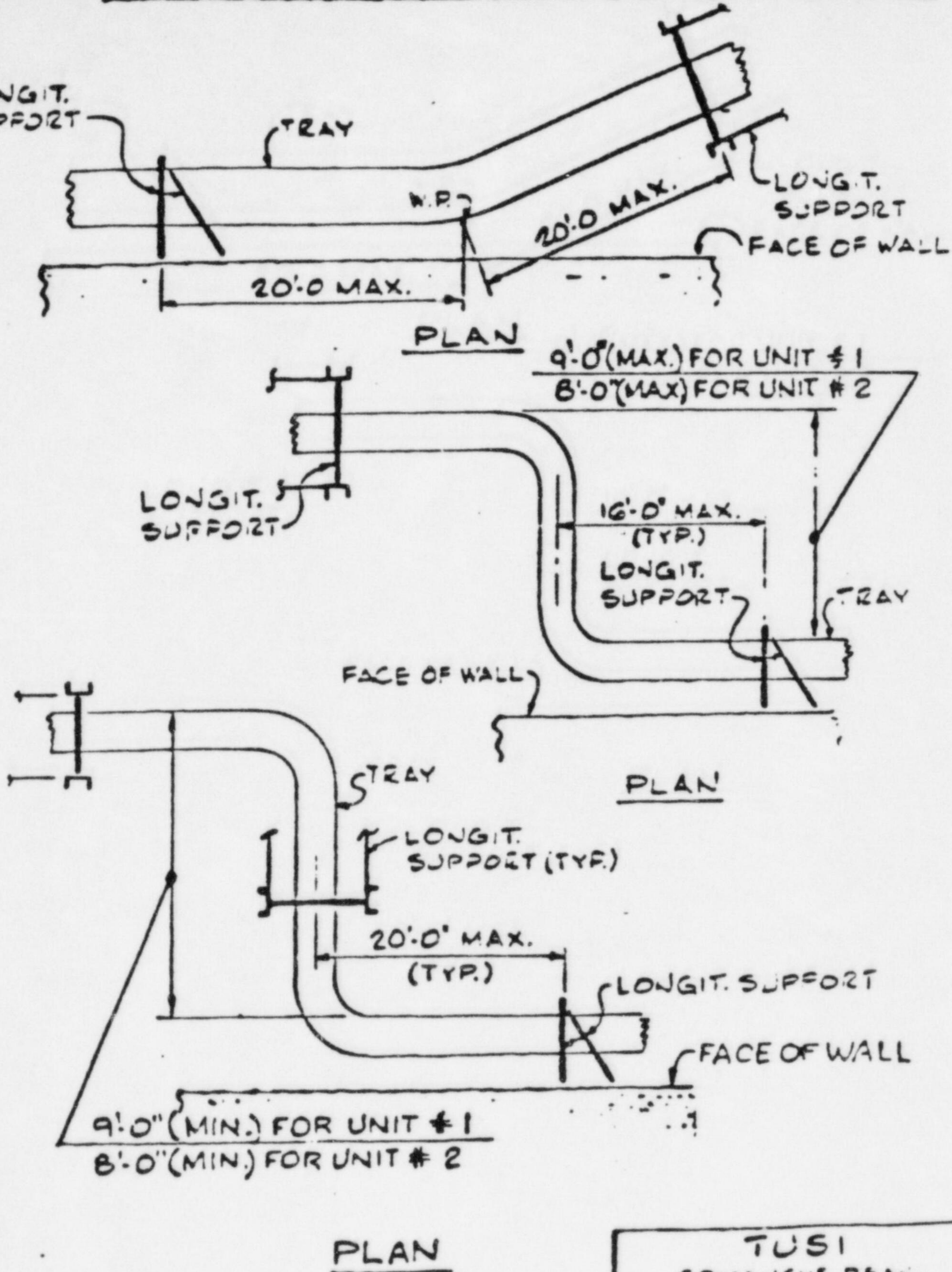
COMANCHE PEAK

ARRANGEMENT OF LONGITUDINAL CABLE TRAY SUPPORTS STRAIGHT RUN

NONE

SH 30

SUPPORTS FOR LONGITUDINAL LOADING



TUSI
COMANCHE PEAK

ARRANGEMENT OF LONGITUDINAL
CABLE TRAY SUPPORTS
HORIZONTAL ELBOW

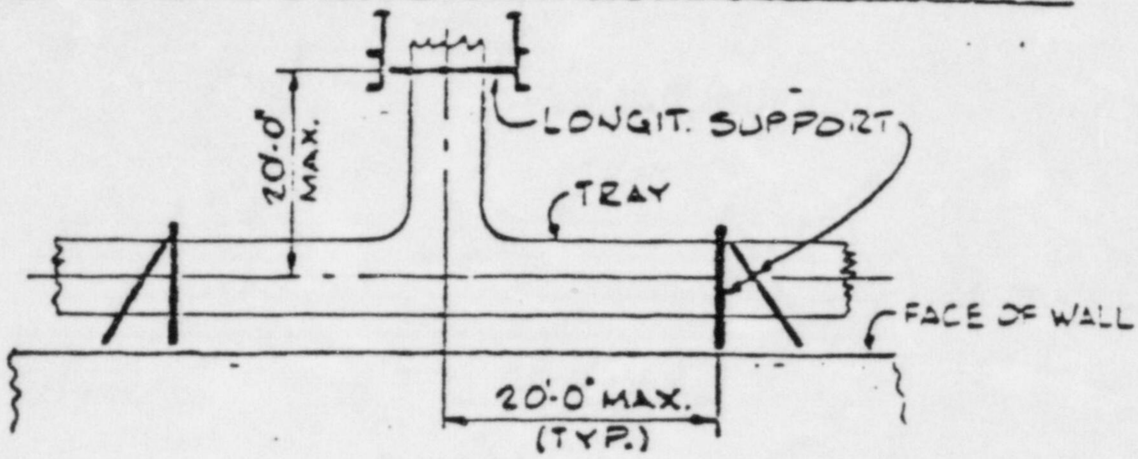
NONE

2323

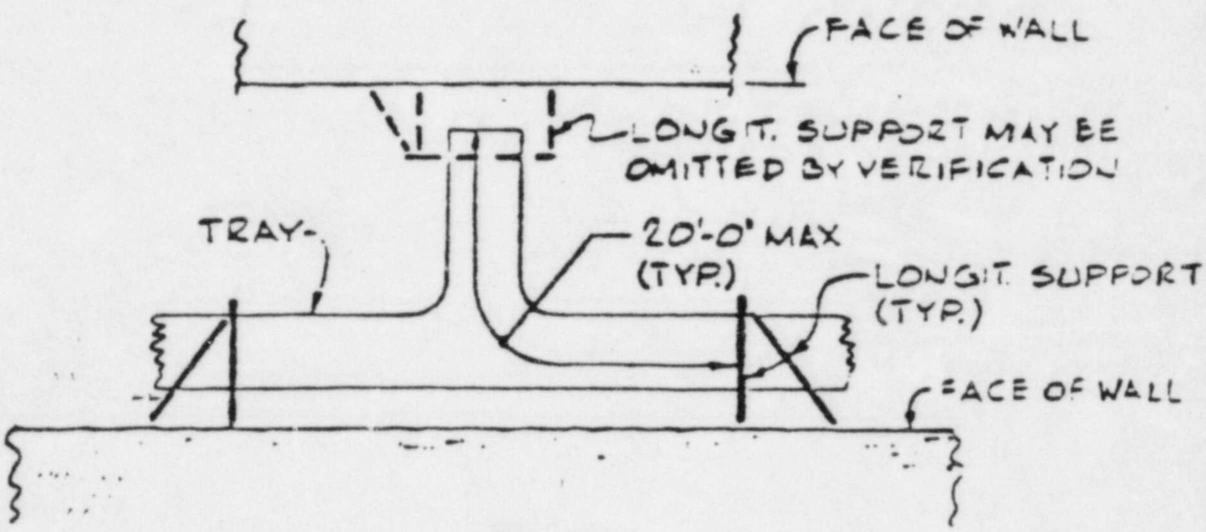
SH 3.1

HORIZONTAL TEE

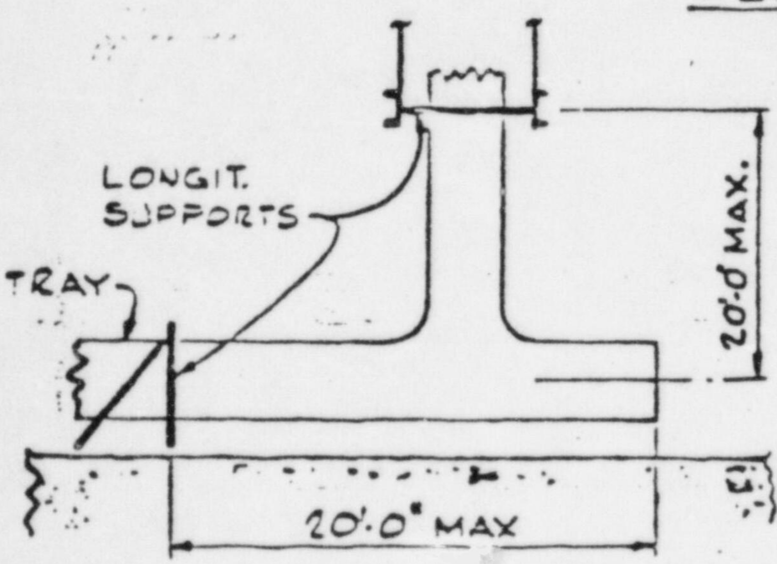
SUPPORTS FOR LONGITUDINAL LOADING



PLAN



PLAN

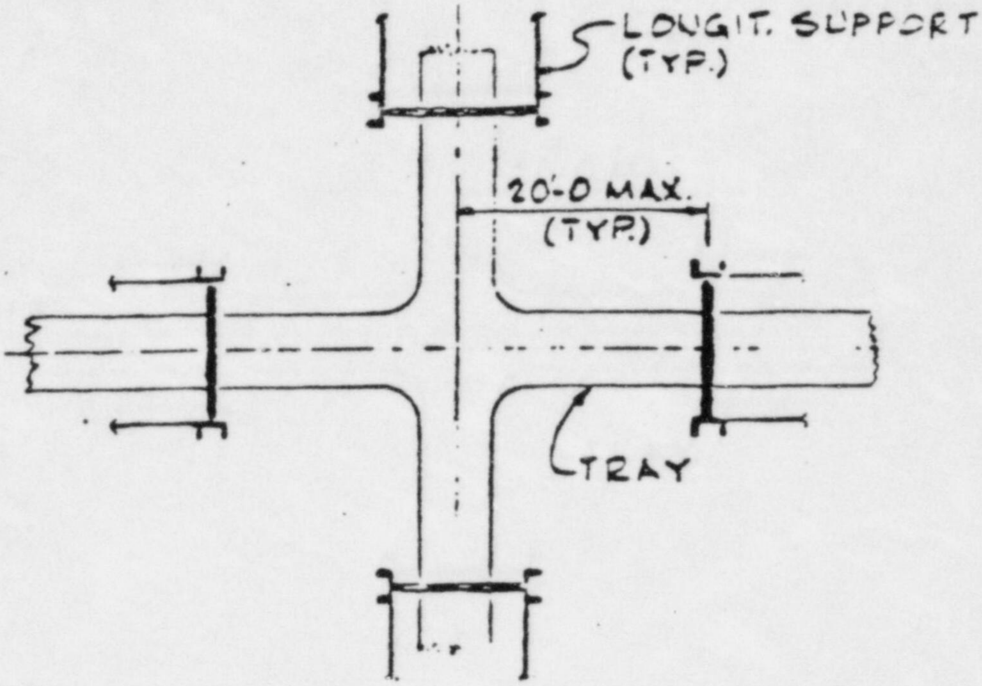


PLAN

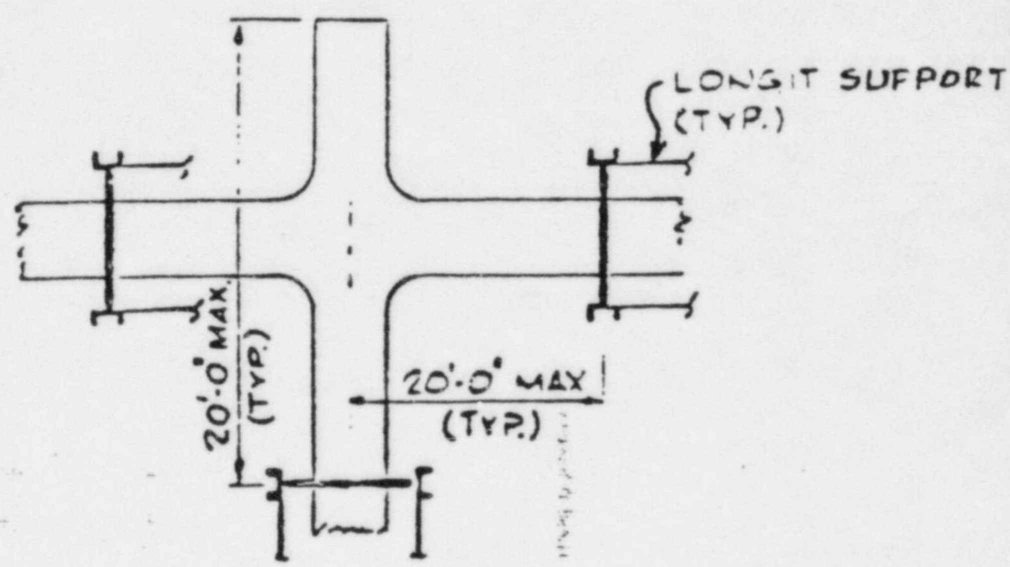
TUSI COMANCHE PEAK
ARRANGEMENT OF LONGITUDINAL CABLE TRAY SUPPORTS - HORIZONTAL TEE
Gibbs & Hill Inc — NONE
2323 — SH 3.2

HORIZONTAL CROSS

SUPPORTS FOR LONGITUDINAL LOADING



PLAN



PLAN

TUSI
COMANCHE PEAK

ARRANGEMENT OF LONGITUDIN
CABLE TRAY SUPPORTS
HORIZONTAL CROSS

NONE

2323

SH 3.3