COMANCHE PEAK STEAM ELECTRIC STATION

TRAIN C CONDUIT
TWO INCH AND UNDER

Criteria Document

REGULATORY GUIDE 1.29 ISSUE

(T.R.T. ISSUE 1.c)

Prepared for:

Texas Utilities Generating Company

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Impell Report No. 01-0210-1479
Revision 0

March, 1986

COMANCHE PEAK STEAM ELECTRIC STATION

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This report has been prepared by Impell Corporation, compiling results from work performed by Impell Corporation, Gibbs and Hill, and Ebasco Services. All work by each company has been performed in accordance with that company's quality assurance program.

The signatures below verify the accuracy of this report and its compliance with the applicable Impell quality assurance requirements, for work performed by Impell Corporation.

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FOREWARD

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1.0 INTRODUCTION

This report describes the plan to resolve the TUGCo Technical Review Team (TRT) Issue 1.c regarding Train C conduit two inch diameter and under. This plan considers the issue, from engineering, walkdown, and possible rework perspectives.

The report includes the justification of the proposed plan. Justification is based on the USNRC Regulatory Guides and Standard Review Plan. Other documents (attached as appendices to this report) give details for implementing this plan.

The Train C plan will provide the following: (a) a tracking system which documents conduit supports that are qualified, and the manner in which they are qualified; (b) a tracking system which documents the reworked conduit supports; and (c) the documentation of all work performed to support this activity.

Except as specifically noted in the text, this criteria document does not address Train C conduit greater or equal to two and one-half inch diameter.

2.0 BACKGROUND

2.1 Regulatory Requirements

TUGCo's commitments to resolve the TRT 1.c issue are given below.

2.1.1 Regulatory Guide 1.29

Regulatory Guide 1.29 [1] requires that portions of structures, systems, or components, whose failure could reduce the functioning of any plant feature needed for safety, be constructed so that the Safe Shutdown Earthquake does not cause such failure.

2.1.2 Standard Review Plan 3.7.2

Standard Review Plan (SRP) 3.7.2 [2] describes the methods used by the NRC to review how TUGCo evaluates the interaction of non-category 1 structures with category 1 structures. The Train C conduit and support system at Comanche Peak are non-category 1 structures.

To be acceptable, the interfaces between Category 1 and non-Category 1 structures and equipment must be designed for the dynamic loads and displacements produced by both the Category 1 and non-Category 1 structures and equipment. All non-Category 1 structures and equipment may meet any of the following requirements:

1. The collapse of any non-Category 1 structure will not cause the non-Category 1 structure to strike a seismic Category 1 structure or component.

2. The collapse of any non-Category 1 structure will not impair the integrity of seismic Category 1 structures or components.

3. The non-Category 1 structures will be analyzed and designed to prevent their failure under SSE conditions in a manner such that the margin of safety of these structures is equivalent to that of Category 1 structures.

TUGCo plans to use a combination of these three requirements to address the Train C conduit issue. This plan is described in detail in Section 3.0 of this report.

The Train C conduit must be considered for both seismic loads and seismic displacements. In this report, the "loads" check is referred to as a "strength" check; this "strength" check is met by showing that the conduit meets the acceptance criteria described in Section 3.2.2 of this report. The "displacement" check is met by showing that the conduit cannot impact a target or by showing that the target is not impaired by any impacts. The impacts considered arise from either a "falling" conduit (if a support is hypothesized to fail the strength check) or a "swaying" conduit (if a conduit displaces during the SSE enough so as to bump into adjacent safety related features).

The underlying philosophy incorporated in this plan is as follows:

1. Review one hundred percent of the Train C conduit supports and conduit runs for both "strength" and "sway/displacement" requirements.

2. Screen out the conduit supports and conduit runs using an eight level screening process (Section 3.3). This screening process will identify all conduit supports and runs which meet the SRP 3.7.2 requirements.

3. For supports and runs which do not meet the SRP 3.7.2 requirements, suitable rework options will be taken such that these supports or conduits meet the requirements.

2.1.3 CPSES FSAR Commitments

The CPSES FSAR [3] states that all non-Category I equipment and components will comply with the provisions of R.G. 1.29.

2.2 Technical Review Team Item 1.c

The TRT Item 1.c is described in the CPRT action plan [4]. The following sections describe these issues, and identify the actions already taken to address them.

2.2.1 Technical Review Team Concerns

The Train C conduit at Comanche Peak are non-Category I features and are not needed for plant safety. The original plant design called for dead-weight hangers for these conduits. Subsequent to construction of these supports, the question arose whether or not these dead-weight supports could withstand the SSE motions, and whether or not the requirements of Regulatory Guide 1.29 were met.

The Train C conduits are divided into two groups: those conduit with diameters two and one-half inches or greater, and those conduit with diameters two inches or less. Prior damage study evaluations have considered the larger size Train C conduit.

Some of the two inch diameter and under Train C conduit at Comanche Peak are "gang" hung; that is, many conduit are supported on a single support. The total mass of conduit on a single "gang" support could be substantial. Therefore, the CPRT instituted an action plan [4] to review the small diameter conduit.

In December 1984, a sampling program was initiated to evaluate the small diameter Train C conduit. The sampling study is described below.

2.2.2 Sampling Study

The sampling study was performed by Gibbs and Hill [5] to determine the capability of the small diameter Train C conduit to meet the requirements of Regulatory Guide 1.29 and SRP 3.7.2. The study addressed 257 conduit runs and 2413 supports. The study was subdivided into two subsamples:

Random Sample:	126 runs	1227 supports
Engineering Sample:	131 runs	1186 supports
Total	257 runs	2413 supports

The studies focused on conduit runs considered to be the most significant. The Random Sample included conduit of:

1.5 or 2 inch diameter

The Engineering Sample included conduit which met any five of nine criteria:

1. safety related equipment nearby
2. 2 or 1.5 inch diameter
3. over 20 foot span for total conduit run
4. over an 8 foot span (for a single span)
5. greater than 15 feet of conduit unrestrainted longitudinally
6. 3 conduits or greater in one support
7. a conduit run with more than 25 percent "special" supports
8. conduit in a congested area (i.e., being closer than 6 inches to unrelated hardware)
9. conduit located in the upper half of the building

Although the Random and Engineering samples have somewhat different selection criteria, for the purposes of this report and plan, all 257 runs and 2413 supports are considered to be from one single larger sample. This report does not draw distinctions in Train C conduit seismic capabilities based upon the above two subclassifications.

The criteria used in the Sampling Study are described in Section 3.2.1 of this report. The results of this work are as follows:

- 2413 supports were reviewed
  230 support failed the strength criteria

- 257 conduit runs were reviewed
  14 conduit runs failed the strength criteria

- 46 conduit runs pass the strength criteria, with one or more individual supports not meeting the strength criteria (no zippering effect)

Several runs have displacements over 0.5 inch, which required a field verification to identify if any safety related equipment could be impacted. Some of these runs could impact adjacent equipment.

The above results need to be considered in light of several very conservative assumptions that were incorporated into the sample studies. Considering the above results, the following actions were undertaken to quantify these conservatisms:

1. conduct tests of various support components to determine their actual strength to withstand seismic events.

2. conduct investigations of the conduit runs to eliminate conservative modeling assumptions made in the sampling analyses.

3. determine the actual conduit fill masses.

4. use acceptance criteria consistent with the functional requirements of Train C conduit. (This approach is consistent with CPRT action plan which states in 4.1.2.3 that "Later screening may be considered if it is required to verify acceptable performance of runs which do not pass the initial screening criteria. Analytical techniques may be refined and/or limited ductility considered, consistent with the intended performance requirements." [4])

The key results from this "refined" sampling study are that of the 2413 supports in the sample, only 43 failed the strength criteria.

The conclusions from these sampling studies are as follows:

1. The first-phase sampling study analysis was overconservative,

2. The second-phase sampling study analysis using the refined criteria is a reasonable prediction of what may occur in the plant. About 1.8 percent of all supports may possibly be overloaded during the SSE. These "overloads" are mainly due to loads on Hilti Kwik Bolts in excess of one-third their nominal design strengths, or slippage of conduit along their axes through clamps. In fact, it is likely that no support will actually break due to the SSE event.

3. The failure rate of 1.8 percent for strength requires TUGCo to perform a plant walkdown to identify and correct these possible problems.

4. The sample shows over 98 percent of supports as passing the criteria. The sample is representative of the plant as a whole, since it is large (2413 supports); and based on largest diameter and heaviest weight conduit (mostly 2-inch and 1.5-inch diameter). Therefore, TUGCO expects that over 98 percent of all Train C supports in Comanche Peak are adequately safe. The emphasis of the plant walkdown is to identify and correct as appropriate the remaining suspect Train C supports.

2.2.3 <u>Total Scope of Train C Conduit</u> (2 inch and Under)

In Unit 1 and Common areas of CPSES, there are estimated to be the following:

- 13,500 Conduit Runs
- 60,000 Supports

These are broken down further as follows:

	Diameter	% of Total	Number
•	2 inch	9	1,215
•	1.5 inch	19	2,523
•	1 inch	17	2.337
•	0.75 inch	55	7,425
			13,500

3.0 WALKDOWN PLAN

3.1 Scope of Plan

TUGCo will include all Train C conduit, two inches and under, in Unit 1 and Common areas into this plan. It is expected that this plan will also be implemented for Unit 2.

3.2 Overall Criteria of the Plan

As described in Section 2.1 of this report, TUGCo will conduct this plan to meet the requirements of Regulatory Guide 1.29. As will be described in the following sections, there are several methods which TUGCo will use to qualify the Train C conduit. Each method has detailed criteria. In Section 3.2.1 through 3.2.3, some of the key criteria are described. In Section 3.3 and 3.4, the justifications for the screening levels are described.

3.2.1 Sampling Study - "Old" Criteria

As described in Section 2.2.2, a sampling study was performed for 257 conduit runs, and 2413 conduit supports. This section describes the criteria used in this study. (This criteria is referenced to as the "old" criteria.) It should be noted that this study was an interim step in responding to the Train C issue.

The following are the "old" criteria used in the original sampling study:

- check Safe Shutdown Earthquake loads only
- stress allowable = $0.90F_y$
- F_y = 36 ksi for structural steel shapes
- F_y = 33 ksi for unistrut shapes
- SSE factor of safety of three for Hilti Kwik Bolt concrete expansion anchors
- 7 percent damping for conduit
- use Nastran code to perform response spectrum analyses of conduit
- use maximum fill weights for all conduit
- all gang type supports qualified using absolute summation of individual conduit reaction loads
- floor response spectra are the "refined" CPSES floor response spectra (These spectra are the same floor spectra being used for all CPESES design verification work, including piping and cable tray efforts.)

The definition of a failure depends on the item being reviewed. If a conduit run has no support failures, the run is considered as "passed". If a conduit run has support failures, but then is reanalyzed with the failed supports removed, and all remaining supports have no failures, then the run is considered as "passed", with no zipper effect. Other conduit runs are considered as "failed".

The analyses also calculate the deflections (sway) of the conduit. Deflections which are under 1/2 inch are considered to have no potential for impact to adjacent plant features. Deflections which are over 1/2 inch are considered as potential candidates for impact to adjacent plant features. The potential candidates are then further investigated to see if any plant features exist nearby. If so, these runs are listed as "potential sway failures".

The conclusions from the original sampling analysis are as follows:

- 230 supports fail strength
- 14 runs fail strength
- 46 runs pass, but include one or more failed supports (no zipper effect)
- Several runs have large displacements, some of which are identified as having potential sway failures. Evaluations of potential target plant features was not performed in this sampling study.

Further details of the assumptions and criteria used in the original sampling study are included in Gibbs and Hill calculation files [5].

The main conclusions from this original study are that:

1. a refined criteria is needed to accurately evaluate the acceptability of Train C conduit;
2. a reevaluation of the sampling study is needed to accurately establish the acceptability of Train C conduit;
3. a walkdown effort is needed to identify and possibly rework, if necessary, potentially "weak" conduit supports.

3.2.2 Sampling Study, Refined Criteria

After completion of the original sampling study, described in Section 3.2.1, several additional tasks were performed. A series of test programs was run in order to further examine the strength of unistrut supports [6, 7, 8]. Further site investigations were performed to obtain conduit layout information, to reduce the need for conservative modeling assumptions. Actual conduit mass weights were obtained. A database listing the margins of all supports in the Sampling study was prepared. A set of refined acceptance criteria was established.

This section describes the refined criteria. The details of these criteria are given in an Impell Project Instruction, [9] attached as Appendix A to this report. These refined criteria are based upon qualifying the non-safety related Train C conduit to withstand the SSE earthquake without any collapse that could impair plant safety features.

The refined criteria allow a limited amount of yielding in ductile unistrut supports. The criteria are consistent with CPRT action plan [4]. The criteria are also consistent witn the CPSES commitment in the FSAR (3.7B.2.1.5), which states:

> "For the SSE earthquake, primary stresses should remain below yield, based on elastic system analysis, or primary stresses may exceed yield, if validated by plastic analysis. Secondary (self-limiting) stresses need not be considered. Some permanent deformations are allowed after the SSE earthquake, provided functionality is maintained" [3].

For Train C conduit, no functionality requirement exists, either for performance during or after the SSE earthquake. Therefore, permanent deformations of either the conduit itself, of the conduit supports, are acceptable. However, the refined criteria should assure that collapse of the conduit supports cannot occur.

3.2.2.1 <u>Analysis Methods</u>

The refined criteria allow three alternative analysis methods:

(1) If stress is limited to yield, as calculated by elastic analyses, then no further checks are needed (essentially the same criteria rules as described in Section 3.2.1) (See Section 3.2.2.1.1 for details)

(2) If stresses exceed yield, then a simple plastic analysis is performed. This simple plastic analysis follows the requirements of ASME Appendix F [10] for level D conditions. An elastic system analysis (including the conduit run and its supports) is performed; then the resultant loads are applied to a nonlinear subsystem load-deflection curve (the subsystem being the yielding support). The nonlinear subsystem (support) load-deflection curve is taken from test data of the support in question. (See Section 3.2.2.1.2 for details)

(3) If the stresses exceed yield, then a detailed plastic analysis is performed. This detailed plastic analysis follows the requirements of ASME Appendix F for level D conditions. A complete nonlinear system and subsystem model of the conduit system is developed and run. (See Appendix B for details)

Although all three of the above qualification methods are adequate to qualify the conduit system, in practice only the first two methods are used. This is to maintain low analysis costs, and to maintain a margin of conservatism in the analysis. To demonstrate that any approximations in the "simple" plastic analysis are conservative, one detailed plastic analysis has been performed. Results of this detailed plastic analysis, and its comparison to the simple plastic analysis, are included in Appendix B to this report.

3.2.2.1.1 Elastic Analysis Method

The Elastic Analysis method used in the "refined" criteria is essentially the same as used using the "old" criteria. Only the SSE load condition is checked. This is justified, as the first method keeps all supports at or below yield, thus no nonlinear effects occur. The OBE stresses are not signficant, and therefore checking the SSE induced stresses is sufficient to demonstrate qualification of the support.

3.2.2.1.2 Simple Fatigue Analysis Method

Check OBE and SSE contributions to fatigue damage when using a simple fatigue analysis method. The fatigue damage is defined as follows:

$$\frac{10}{N_{SSE}} + \frac{50}{N_{OBE}} \leq 1.0$$

This fatigue damage formula is typical of the requirement of IEEE 344-1975 [11] for qualification by test, which assumes five OBE earthquakes and one SSE earthquake. This fatigue damage formula also is based upon as assumed 10 equivalent peak-to-peak load cycles per earthquake, as recommended as being a conservative assumption per the ASME code [12].

In reference [9], a number of fatigue curves have been included for use in the qualification of CPSES unistrut supports and unistrut connections. These fatigue curves have been taken from test data. These fatigue curves have built into them a factor of safety of 1.5 on cycles.

For cases where explicit fatigue data does not exist, then the following procedures are used: use static ultimate strength test data, and limit the maximum allowable load to the lesser of the following: 2/3 ultimate; a displacement ductility of 3.

This 2/3 limitation is comparable to the 0.70 ultimate limitation imposed by the ASME code Appendix F rules [10].

The ductility limitation of 3 is a well documented [13] allowable ductility for:

> "facilities, structures, equipment, instruments or components that can deform inelastically to a moderate extent without unacceptable loss of function."

Further, since Train C conduit supports are non-safety related, the use of even higher ductilities is suggested in [14], where it states that ductility factors of between 3 and 8 can be used for:

> "all other items which are usually governed by ordinary seismic design codes; structures requiring seismic resistance in order to be repairable after an earthquake. Ductility factor = 3 to 8, depending on material, type of construction, design of details and control of quality."

This ductility limit is further justified for non-safety related components, as ordinary design codes allow a ductility of 8 [15] for ductile steel components.

Further, in [14], it is noted that the definitions of ductilities used in [13] are for system ductility, and that consistent with these allowable system ductilities it is expected that member ductilities will be even larger. Since the refined criteria specifies that the member ductility limit is 3, even lower system ductilities are implied.

3.2.2.2 Stress Allowables

For hot rolled structural steel members, the stress allowable = 1.0 F_y. This limit of yield stress is consistent with industry practice for non-safety related features. It is actually conservative as compared to the ASME code Appendix XVII [16] code recommendation of 1.2 F_y for pipe supports loaded to Level D limits. It is 4 percent higher than the stress allowable used for safety-related AISC qualified steel structures for CPSES, as allowed in CPSES FSAR 3.8.4.

For cold rolled steel members, the stress allowable = 1.0 Fya. F_{ya} is the term used in the AISI code to consider the effects of cold working, as per AISI code section 3.1.1 [17].

3.2.2.3 Concrete Expansion Anchor Factor of Safety

This section describes the justifications for using a factor of safety (F.S.) of 3 for the Hilti Kwik Bolts used in the Train C conduit supports. This section augments a report which has previously been submitted to the NRC [18] which justifies the use of F.S.=3 for Hilti Kwik Bolts.

A comprehensive report [19] has been compiled concerning the Hilti Kwik Bolt Factor of Safety. This report is included as Appendix C to this report. The following paragraphs highlight the key conclusions of this report:

1. For non-nuclear conventional structures, there is documented instances where building code officials have allowed a F.S. = 3 for Hilti Kwik Bolts, when including seismic loads.

2. A review of engineering parameters affecting Hilti Kwik Bolt strength has been made. These parameters include:

   - embedment depth
   - concrete strength
   - air entrainment
   - edge distance
   - bolt spacing
   - aggregate size and hardness
   - tolerance on hole size
   - anchor pre-load
   - static or cyclic loading

   These parameters have no significant affects on the decreasing the Hilti Kwik Bolt strength for Train C conduit supports.

3. By considering actual minimum concrete strengths, the criteria F.S. is 3.45. By also considering average concrete strengths and bolt embedments, the criteria F.S. is between 3.94 and 4.11.

4. Not all Hilti Kwik Bolts are loaded up to their criteria allowable F.S. = 3. A statistical study (preliminary) of over 2200 Train C conduit supports with Hilti Kiwk Bolts showed an average F.S. = 10. By considering average concrete strengths and embedments, 99%$\pm$ of the supports in the sample have F.S. over 4.00.

5. Statistical evaluations of the required F.S. to maintain nuclear plant safety suggest that at a F.S. = 2, a 1% failure rate for individual Hilti Kwik Bolts can be expected, including workmanship defects. This 1% failure rate leads to a 0.01% failure rate for a collapse of a multiply supported Train C conduit run, and less than one in one million probability of such a failure during the plants lifetime. At a F.S. =3, the probability of failure goes up by another order of magnitude.

It is concluded form the above that the Criteria of F.S. = 3, as used in the Sampling studies, for both the "old" and "refined" criteria, is acceptable and safe for Train C applications.

3.2.2.4 Damping

This damping value is consistent with the SSE level damping values used for the unistrut-hung conduit at many other nuclear plants, including San Onofre 1, 2, 3; Diablo Canyon 1, 2; Palo Verde 1, 2, 3; Hope Creek, Braidwood 1, 2; Byron 1, 2; South Texas 1, 2; and many other plants of similar vintage to Comanche Peak [20]. This value of 7% damping is also suggested for bolted structures, as described in Regulatory Guide 1.61 [21]. Recent test programs [22] have also confirmed that 7% damping for rigid conduit is conservative for moderate levels of shaking.

3.2.2.5 Quadratic Interaction for HKB.

A quadratic interaction equation shall be used for Hilti Kwik Bolts. A quadratic interaction equation has been used at other nuclear power plants and has been confirmed by correlation with experimental results [23, 24, 25].

3.2.2.6 Gang Supports.

These are qualified using an SRSS summation of individual conduit reaction loads, if conduit frequencies are widely spaced; otherwise, summation is by absolute summation. The SRSS summation is used only if conduit frequencies are widely spaced, using the R.G. 1.92 criteria [26]. The SRSS summation has also been checked for specific application to CPSES conduit [27].

3.2.2.7 Floor Response Spectra.

Floor response spectra are the "refined" CPSES floor response spectra. The same spectra that has been used in the original sampling analysis (Section 3.2.1).

3.2.3 Target Analysis - Acceptance Criteria

As described in Section 2.1.2, one of the three methods to show acceptance to the R.G. 1.29 issue is to demonstrate that the collapse of Train C conduit will not impair the integrity of seismic Category I structures or components. For purposes of this Train C conduit program, this check for integrity is called "target" analysis.

Target analysis is commonly used in the pipe break issue, as well as the R.G. 1.29 issue. For all target evaluations performed for the CPSES train C conduit, the same criteria normally used to address the pipe break issue are used.

3.2.3.1 Scope of Targets

The following types of features can be considered as targets:

1. Safety and non-safety class piping and conduit systems

2. HVAC ducts and supports

3. Structural members

4. Cable trays and cable tray supports

Category I safety features that are not allowed as targets:

1. Pipe fittings, including elbows, tees, reducers, stanchions, valves, valve extended operators, tubing, snubbers, springs, and other active or passive components.

2. Electrical cabinets, battery racks, or other electrical components required to be operable during or after the earthquake.

3. Cables exposed directly to possible impacts (ie., trays with no covers that a falling conduit or conduit support can directly hit).

4. Mechanical equipment, such as tanks, heat exchangers, etc.

5. Any structural connection.

6. Any impact to a cable tray, HVAC, structural members pipe, or conduit that occurs within one-sixth of the span length from the span's supports. This condition can be relaxed only if it is shown that the support does not rely upon concrete expansion anchors to resist the energy of the impact.

7. Flexible hose.

8. Tubing or electrical equipment attached to pipe (like heat tracers, pressure temperature gauges, pneumatic tubing, etc.).

9. Any structural member connected to concrete.

Further restrictions as to acceptable targets are described in the following sections.

3.2.3.2 Acceptance Criteria

This section describes the acceptance criteria used for targets. Details of the application of these criteria are described in the Ebasco report [28], attached as Appendix D of this report.

Targets will be shown acceptable if the potential energy of the postulated falling Train C conduit can be absorbed by plastic strain energy. The maximum allowable strain imposed on targets other than piping is limited to the lesser of:

1. 10 percent of the strain at ultimate tensile stress, for the material.

2. 10 times the yield strain. Yield strain is defined as the yield stress divided by Young's modulus.

For the evaluation of pipes, the plastic strain energy is limited to that of 70 percent of ultimate bending moment capability of the cross section, as per ASME code, Appendix F [10].

The usable strain energy absorbed by the target due to the impact cannot double count the energy being used to withstand the deformations imposed by other loading conditions. A conservative assumption is made that the target structure prior to impact is stressed all the way to its yield stress, or to the highest design stress (or strain) that the target's applicable code will allow, whichever is greater. The energy corresponding to this stress (or strain) is then deducted from the allowable strain energy of the target structure which will then be available for impact.

In addition to the above deformation-related criteria, safety related targets must also be able to maintain fuctionality during and after the impact. Accordingly, distortions in the target structures are limited to assure the target can properly perform its function.

For potential Train C conduit missiles caused by postulated support failures, it is assumed that there is only one impact during the earthquake. For Train C conduit that can sway during the earthquake, and thereby occasionally interact with other neighboring features, then the target analysis is adjusted to account for potential multiple impacts for the duration of the 10 second earthquake event.

Pipes and Conduits

The target piping may be class I, II, or III. It is conservatively assumed that all pipes are class I. Conduit may be classified as pipes.

1. No impacts are allowed if the impacted span is supported no closer than one-half that required by ANSI B31.1. (A table will be provided in the walk-down procedure.)

2. Impacts are allowed if the impacted pipe is a straight pipe, with or without insulation.

The above limitations (and those in Section 3.2.3.1) may only be relaxed through further detailed engineering evaluations of the particular case.

The distortion of the pipe cross sections shall not cause a reduction of the net flow area by more than five percent.

HVAC Ducts

The maximus allowable strain for ducts is 10 percent of ultimate strain, or 10 times yield, whichever is smaller.

The maximum allowable reduction in net flow area of the duct is less than 10 percent.

Structural Members

The maximum permissable strain is 10 times the yield strain for structural supports.

Spans must be longer than four times the structural member's depth.

Missile Evaluation

To perform the target evaluation, the weight of the potential Train C missile needs to be be established. In this section, the procedure to get the weight of a falling Train C conduit is presented.

First, the number of Train C supports, in a row, that are prone to fail are identified by the walkdown engineer. The engineer may consider if adjacent Train C supports are prone to also fail (zipper) due to the first failure, or may consider that the entire length of conduit fails (from end to end, including termination points).

Once the entire length of potentially non-hung conduit is identified, all targets in the possible zone of failure are identified. This step is performed with the aid of a catenary design chart, showing the maximum possible deflections (vertical and horizontal) of the non-hung conduit.

If any targets exist within this identified zone of failure, it is assumed that the target must resist the energy associated with a portion of the length of this span of non-hung conduit. If it has been conservatively assumed that the entire length of conduit falls, then the entire span of non-hung conduit is assumed to be in free fall. Not all the weight of a very long non-hung conduit will be effective in impacting a single target. Thus, based upon the results of detailed impact analyses, the following length reduction factors may be employed:

Length of non-hung conduit	Percent of non-hung span used to evaluate weight (%)
up to 16 feet	100
16 to 23 feet	80
over 23 feet	60

Each target is assumed to have to bear the entire impact of the falling conduit, without benefit of other target's capabilities. This assumption can be relieved with further engineering evaluation.

3.3 Eight Screening Levels

TUGCo plans to close out the Train C conduit issue by using a full plant walkdown approach. There are eight screening levels which the walkdown team may use before opting to use a rework option (rework option are described further in Section 3.4). All of these screening levels are in accordance with the requirements of R.G. 1.29 and S.R.P. 3.7.2.

In this section of the report, the eight screening levels are briefly described. The justifications for each of these screening levels are described. Figure 1 shows the flow of the screening levels.

3.3.1 Screen 1 - Weight

If a support has less than 6 pounds per foot of conduit, then it is satisfactory. This criteria is met by supports having only one-two inch diameter conduit, or combinations of other smaller conduit, with a total weight less than 6 pounds per foot.

Example combinations of acceptable supports meeting this screening level are:

1. 1-1.5" + 1-1"
2. 3-1"
3. 4-.75"
4. Any single conduit

This screening criteria is justified for the following reasons:

1. Supports are strong enough to prevent a conduit run to fail through zippering, due to the light loads imposed by the above configurations. This has been born out through a detailed review of 2413 supports from the sampling study, and results of this study are included in the reference [29].

2. This is a common assumption in nuclear plant design, and is in use at many other operating and NTOL nuclear plants.

3.3.2 Screen 2 - Good Supports

There are certain type of unistrut supports used for the Train C conduit which are very resistent to seismic loads. In the sampling study, there were a total of 2413 supports reviewed. Of these supports, only 43 supports were postulated to be overstressed.

Thus, there is likely only 1.8 percent of all Train C supports in CPSES are in fact potential candidates to be overstressed during the SSE. These supports have been reviewed [29] and are found to involve mainly the overstress of Hilti Kwik Bolt concrete expansion anchors or slippage of conduit through clamps. These 1.8% of supports can be classified as being in potential "bad" support groups.

Conversely, 98.2 percent of all supports are not overstressed during the SSE. These supports can be classified as being in "good" support groups.

In order to define a "good" support group, there must have been zero failures of this type of support out of the entire sample population of 2413 supports. The actual descriptions of the "good" support groups are described in [30], included as an Appendix E to this report.

3.3.3 Screen 3 - No Interaction Potential Check

By using this screen, the walkdown team ascertains whether there is any possible targets within the zone of possible conduit fails. The zone is described using the procedure described in Section 3.2.3.2 of this report.

If there are no possible targets in this zone, then the support is considered acceptable.

Note, that for conduits supported on rod hangers, or long lengths of "non-hung" conduit, an additional check must be made to assure that no "sway" displacements from the vibrating conduit can result in interactions with other plant features. A chart is provided to the walkdown team which defines the maximum possible sway for rod hung conduit. This chart varies according to the elevation of building, and the length of the rod support. More refined checks may be made by the walkdown team to minimize the sway deflections, including pendulum effects, and stiffening effects of adjacent conduit branches and conduit supports.

If there are no sway potentials for interaction, then the support and attached conduit is considered qualified.

Note, that any conduit with seismic deflections less than 0.5 inches do not need to be checked for sway interactions. This is in keeping with the common CPSES criteria of a minimum of one inch separation between all features.

3.3.4 Seismic Capacity Check (In the Field)

By using this screen, the walkdown engineer calculates a conservative seismic load for a particular support, and compares this load to a pre-calculated seismic capacity. The pre-calculated seismic capacities are available to the walkdown engineer in the field in tabulated forms.

There are two basic activities the walkdown engineer performs: (1) determines the tributary spans and associated deadweight of all the conduit attached to the support in question. (2) compares this deadweight to the allowable deadweight from the load capacity tables.

The criteria involved are as follows:

1. Tributary spans: the walkdown engineer determines the spans for all conduit on the support being design verified. This calculation depends on each conduit's schematic (layout), and the types (one-way, two-way or three-way) of adjacent supports. The tributary span is calculated for each of the three orthogonal directions. These orthogonal spans may differ due to the differences in load resistance direction capability for the adjacent supports. Once the span to the adjacent support is known, then one-half of this span is assumed to cause reactions to the support being design verified. For spans which terminate with flex hose or into junction boxes, the entire span is assumed to cause reactions to the support being design verified. Finally, this span is multiplied by the maximum weight per unit length for the conduit diameter, for each conduit. The final weight to be applied to the support being design verified is the sum of the span weights.

2. Load Capacity Tables. For each type of generic support often found in the plant, a set of load capacity tables are developed. The criteria used in developing these tables are the same as those described in Section 3.2.3 of this report (refined criteria), with one exception: all capacities assume that the tributary conduit weight is vibrating at the peak of the response spectra, including a 1.5 factor to account for maximum possible higher mode effects, independent of conduit frequency. The load capacity tables are given for loads applied in one, two or three directions, and for intermediate combinations of the capacity tables are further refined to be consistent with the varying levels of floor response spectra, by subdividing the buildings into five seismic zones. Zone 1 is at the top of CPSES buildings, and Zone 5 is at the bottom of CPSES buildings. Thus, a support located in Zone 5 can withstand more conduit weight than the same support located in Zone 1.

   This screening level is justified for the following reasons:

   1. It uses the same acceptance criteria as the supports in the sampling study.

   2. It determines the applied load to a given support always assuming that the conduit is vibrating at the peak of the floor spectra, including a 1.5 higher mode factor.

   3. It assumes maximum conduit fills for all conduits.

3.3.5 Screen 5 - Seismic Dynamic Analysis Check (In Office)

This screen is intended to be used only if screen 4 shows that the support is overloaded but by a margin within the bounds possible to be demonstrated by using more detailed analysis techniques. Other factors include the complexity of potential support modifications or seismic restraints.

Should the walkdown team decide that this screen be employed, then the procedures to qualify the support are based upon the same criteria and procedures given in Section 3.2.2. Walkdown information as to the extent of conduit runs and adjacent support details will be collected in order for a detailed evaluation and analysis to be performed in the office.

This screen is justified, as it used the same procedures described in Section 3.2.2 of this report.

3.3.6 Screen 6 - Target Check

By using this screen, the walkdown engineer will evaluate the acceptability of impacts to adjacent plant features.

This screen may be used to evaluate the acceptability of either falling conduit, or swinging conduit. The details of the falling and swinging target evaluations are similar, as described in detail in Section 3.2.3.2 of this report.

This screening procedure is justified, as it is explicitly allowed by SRP 3.7.2. The criteria for acceptability of targets have been described in detail in Section 3.2.3.2. These criteria are adopted from SRP 3.6.2 [31], ANSI/ANS 58.2 [32] and ASME code Appendix F [10].

3.3.7 Screen 7 - Safe Shutdown System Check

As described.n R.G. 1.29 [1], it is necessary to demonstrate that only certain safety functions of the CPSES are needed during and after the SSE. If the walkdown team identifies a support which is not qualified through the use of the other Screen levels, then a safe shutdown system check can be performed.

3.3.8 Screen 8 - Seismic Restraints

In some areas of CPSES, there have already been installed seismic restraints (also known as aircraft cable restraints). The purpose of these supports is to restrain any possible falling non-safety related features, as determined from prior damage studies of the plant. Some of these restraints may be able to also withstand the loading of potential Train C conduit.

The criteria to judge whether the existing seismic restraints are adequate are based upon the criteria used in Section 3.4.1 of this report. A calculation will be made by the walkdown engineer for the existing restraints that are required by this screen.

3.4 Rework Options

Should a support not be qualified by any of the 8 screen levels, then any of three rework options may be pursued. First, a new seismic restraint (aircraft cable) may be installed. Second, the existing support may be modified. Third, the conduit may be rerouted.

3.4.1 Rework Option 1 - New Seismic Restraint

The criteria for design and installation of new seismic restraints are describe in [33]. These criteria require that all stresses remain at or below yield.

In designing these seismic restraints, the maximum possible conduit weight is considered, considering all possible supports that can fail. Weight from other plant features that could also fail due to the Train C failure are also considered.

3.4.2 Rework Option 2 - Modify Existing Support

The criteria for design and installation for support modifications are the same as described in Section 3.2.1 of this report, with the exception that the minimum factor of safety for Hilti Kwik Bolt expansion anchors is 4.

3.4.3 Rework Option 3 - Reroute Conduit

The walkdown team may determine that rerouting the conduit to be an effective solution for certain instances. In Unit 1 and Common areas, this option will not be considered.

Any rerouted conduit will be supported such that these supports can meet any of the screen levels 1 through 8.

In addition, any rerouted conduit will have all functional tests performed as is required for Train C conduit.

4.0 CONCLUSIONS

This report summarizes the criteria to be used to close out the Train C T.R.T. Issue 1.c. This report gives justifications for each of the criteria. This report also refers to backup documentation which provides the details of the criteria.

All criteria are meet the commitments of the FSAR, and meet the requirements of R.G. 1.29. There are no FSAR changes required by this plan to close out the Train C issue.

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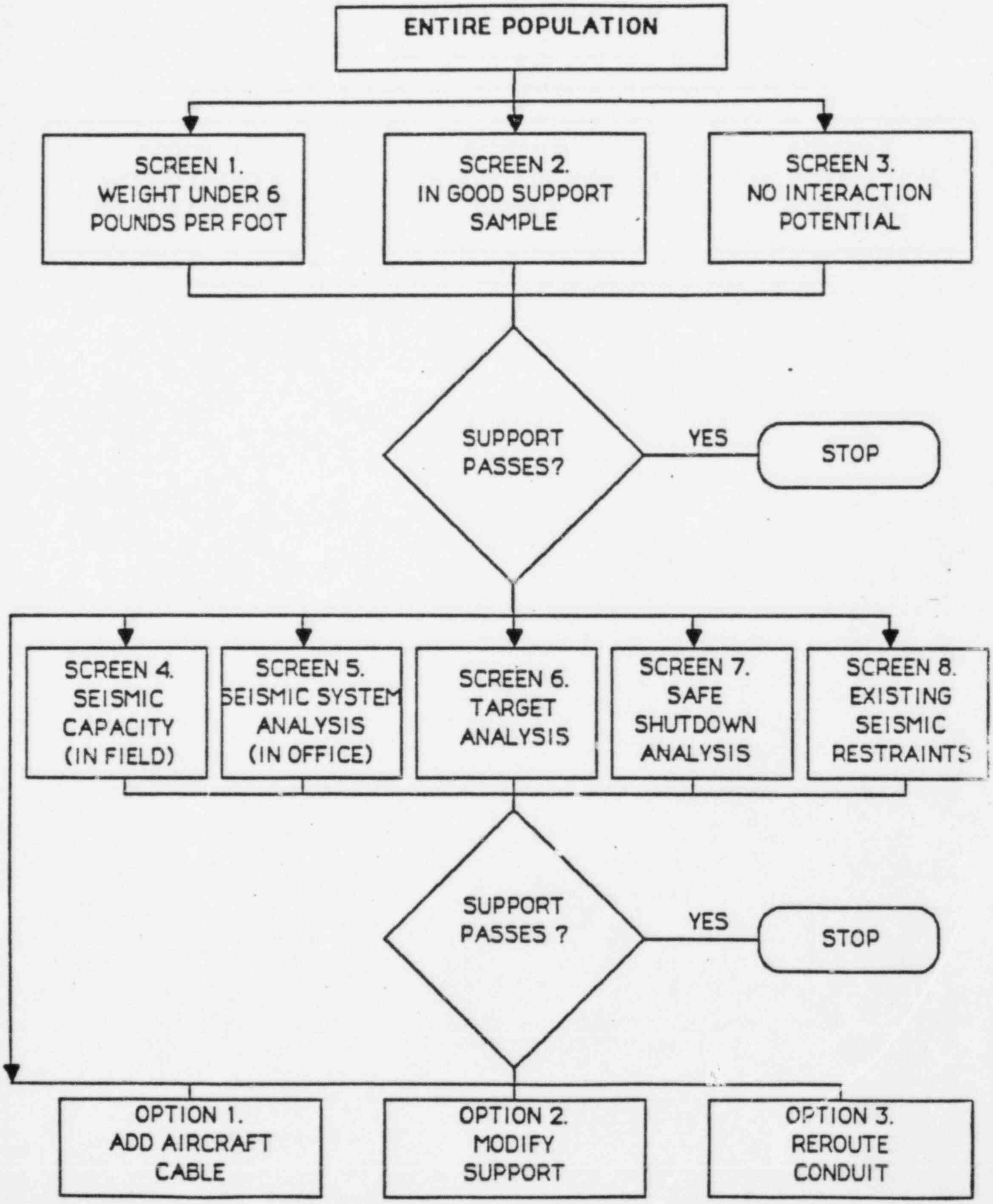
FIGURE 1 - WORKFLOW



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1.0 INTRODUCTION AND BACKGROUND

The Train C Conduit System at Comanche Peak Steam Electric Station is not safety-related and therefore does not have to remain functional (or operable) during an earthquake. However, the Train C Conduit System must not impede the operability of certain components that are safety-related and may be required to remain operable during a Safe Shutdown Earthquake. The NRC Standard Review Plan (Ref. 1) recommends three acceptable methods to address this issue. They are:

A. Ensure that safety-related items necessary for a safe shutdown condition of the plant are not impacted by falling non-safety-related items (e.g. Train C Conduit).

B. Ensure that safety-related items necessary for safe shutdown condition if impacted do not lose their operability.

C. Ensure that non-safety-related items maintain their structural integrity (e.g. Train C Conduit is evaluated for structural integrity in a manner similar to safety-related items).

Texas Utilities Generating Company (TUGCO) has satisfactorily addressed this issue for Train C Conduit which has conduit diameters larger than or equal to 2-1/2 inches. Conduit less than or equal to 2 inches in diameter was assumed to be adequate based on industry practice and engineering judgement. No engineering evaluations were performed for conduit less than or equal to 2 inches in diameter. The Nuclear Regulatory Commission's Technical Review Team found this unacceptable since there are some instances where many Train C Conduits are supported by one support which may have marginal capacity relative to the seismic loads of many conduits.

Gibbs & Hill initiated a program to address this issue. They have focused on Method C (described above). For conduit of 2 inch diameter and under, dynamic analyses were performed by Gibbs & Hill on an Engineering Sample (131 conduit runs with 1186 supports) and a Random Sample (126 conduit runs with 1227 supports). (The entire population consists of about 60,000 supports.) Of the 2413 supports reviewed, approximately 233 (9.7%) failed the Comanche Peak Review Team acceptance criteria, and of the 257 conduit runs, 14 (5.5%) failed the acceptance criteria. Additional runs have "failed supports", but no "zipper" effect occurs. This failure rate does not justify that Train C conduit is not a safety issue. This Project Instruction defines refined criteria to evaluate the Train C supports and to determine the extent of any possible safety concerns for the above Random and Engineering samples. 2

Impell has provided assistance to TUGCO to develop an overall program with Gibbs & Hill and Ebasco to address this issue. The program consists of a multilevel screening process which uses a combination of Methods A, B, and C described above. This Project Instruction addresses Method C. Under this scope of work, the Comanche Peak Review Team (CPRT) Item 1(c) acceptance criteria will be refined to remove unnecessary conservatisms and thereby reduce the 9.7 percent failure rate. Additionally, Impell and Gibbs & Hill will quantify the margin of safety (ratio of support capacity to seismic load) for individual support categories. This will justify other screening levels to be used by a walkdown team at a later date whereby supports with (known) large capacity/load ratios can be qualified using simple screening evaluations. This is an important aspect for the success of the overall program. 2

2.0 OBJECTIVES

About 9.7 percent of the sample (233 supports) which failed the CPRT Item 1(c) acceptance criteria will be reduced. It is expected that the failure rate can be reduced to about 2 percent (or better) using a refined criteria. The refined criteria is described in Section 4.0 of this Project Instruction. The criteria provides methods for reducing seismic loads (by eliminating conservatisms in analysis) and provides methods for increasing capacities. 2

Additionally, all 2413 supports evaluated by Gibbs & Hill will be reviewed to quantify their margin of safety (capacity/load). Support types which have significant capacities relative to the expected seismic loads (i.e. large margin of safety) will be readily identified. Factors of safety for Hilti-Kwik Bolts (capacity to actual load) will be calculated to form a basis to understand the true margin for these bolts.

3.0 SCOPE OF WORK

3.1 Review "Failed Supports"

The 233 failed supports will be reviewed using the refined criteria. If necessary, hand calculations will be performed to reduce support loads. The refined criteria and detailed procedures for implementing the criteria are described in Section 4.0. 2

3.2 Walkdown Effort

A walkdown team will support this effort by gathering information, such as span lengths, support configurations, and conduit fill rates. TUGCO personnel will perform the walkdowns. Impell will treat this data as design/analysis input.

3.3 Computer Analyses

If hand calculations are not successful in reducing the seismic loads, Gibbs & Hill's existing computer models will be modified to more accurately reflect the as-built conduit runs.

3.4 Review "Passed Supports"

Supports which were acceptable using the CPRT Item 1(c) criteria will be reviewed to quantify the margin of safety. Support types with suitable margins will be candidates for support groups which can be later evaluated by simple screening evaluations. Margins for Hilti-Kwik Bolts will be specifically addressed. (At a later date, a walkdown team will "walkdown" the remaining supports of the 60,000 population and qualify supports by inspection without performing an engineering evaluation.) Appendix D gives procedures for this review.

4.0 REFINED CRITERIA

The following refined set of criteria are to be used to perform the seismic evaluation of the 233 failed supports.

4.1 Load Determination

Samples of Train C conduit systems were analyzed by Gibbs & Hill using the NASTRAN computer program (Ref. 7). These computer analyses will be reviewed to determine whether or not support loads were overly estimated as a result of conservatisms in calculating the conduits' masses (number of conduits, conduit sizes, branch conduits, conduit spans). If support loads can be reduced, the conduit support will be reevaluated using the equivalent static analysis method or the elastic response spectrum analysis method.

4.1.1 Equivalent Static Analysis Method

This method calculates the lower bound frequency of the conduit systems to establish the seismic coefficient in computing support loads. The procedure is as follows:

- Frequency Calculation
  - Evaluate a lower bound frequency of the system to justify spectral acceleration, if appropriate. Appendix E contains formulas for calculating the frequency for single or multi-span beams with various boundary conditions.

- If the calculated frequency is less than or equal to the frequency corresponding to the peak acceleration in the response spectrum, the peak acceleration times 1.5 (or 1.1 where appropriate) will be used to account for higher mode participation. A higher mode participation factor of 1.1 as justified in Reference 2 should be used only for straight runs of conduit where supports are evenly spaced and the span is not adjacent to an elbow.

  For situations where a support is near an elbow, the effects of the system response and not just the local response adjacent to the support shall be considered. When both conduits and supports are flexible, Dunkerley's formula can be used to estimate a lower bound frequency of the system:

$$\frac{1}{f^2} = \frac{1}{fc^2} + \frac{1}{fs^2} \qquad [1]$$

  f_c = frequency of conduit
  f_s = frequency of supports
  f = frequency of system

- If the support is a trapeze rod hanger, two effects contribute to a hanger's stiffness: rod bending and the pendulum restoring force. The pendulum-restoring-force term represents the geometric stiffness due to the dead-load tensile force in the rods. The lower bound frequency of a trapeze rod hanger can be estimated as follows:

$$f_s = \frac{1}{2\pi}\sqrt{\frac{k_T}{M}} \qquad [2]$$

  where m is the mass of the support and conduit, and

$$K_T = K_R + K_p = \frac{24EI_R}{L_1{}^3} + \frac{WL_H}{L_1} \qquad [3]$$

  where

  K_R = stiffness due to rod bending
  K_P = stiffness due to pendulum effect
  E = Young's modulus
  I_R = Rod's moment of inertia (root area)
  L_1 = Top tier height, as shown in Figure 1
  W = weight per unit length of conduits
  L_H = hanger spacing

- Support Load Calculation

  Response spectra with 7 percent damping generated by Gibbs & Hill are used in estimating conduit support loads. Only the SSE case is evaluated for stress, but both the OBE and SSE must be considered for fatigue. Support loads are calculated based on conduit tributary weight. The seismic load is obtained by multiplying the tributary weight and the seismic coefficient. For vertical supports, the total load is the algebraic combination of gravity and seismic loads.

  For multiple conduit supports, conduit phasing may be considered, to justify the support loads. If the frequencies (f) of two parallel conduits are within ten percent, the conduits are moving in phase, and the absolute-summation (ASUM) method is used to combine conduit loads. If the frequencies of conduits are different by more than ten percent, the conduits can be assumed to vibrate out of phase, and therefore, the square-root-sum-of-the-square (SRSS) method is to be used to combine conduit loads. This procedure can be used as long as the system frequency is below the floor cut-off frequency. For system frequencies above the cut-off frequency, ASUM will be used.

4.1.2 Elastic Response Spectrum Analysis Method

When the conduit system is too complicated to analyze using the equivalent static analysis method, it will be reanalyzed using the NASTRAN computer program. For this reevaluation, the previous computer model will be revised to more realistically model the as-built Train C conduit system.

4.2 Support Evaluation

Train C Conduit supports are evaluated using elastic criteria or using a simple fatigue analysis. Detailed fatigue analyses which would require a nonlinear system analysis are not covered by this Project Instruction.

4.2.1 Elastic Criteria

This method calculates the stresses for each component of the support and compares the actual stress to an allowable stress as given in Section 4.3. The seismic loads from three orthogonal directions are considered to occur simultaneously and are combined.

4.2.2 Simple Fatigue Analysis

Support components (such as Unistrut members, P1000 or P1001, or anchor bolts which fail using the elastic method) and angle fittings (e.g. P1331, P1026) shall be evaluated using a simple fatigue analysis method. Here "simple" implies that a rotation will be obtained from a nonlinear moment rotation curve (based on test results). The moment (obtained from a linear elastic analysis) will be used to define the rotation on the nonlinear moment rotation curve. This is a conservative (i.e. upper bound) estimate of rotation. This rotation will then be used to obtain the fatigue life from a fatigue curve.

The moment versus rotation curves, fatigue curves, and ductility curves were obtained from test data (Refs. 3, 4, and 5) and are provided in Appendix F.

Because the fatigue test data were obtained from various test programs, some have a preload; whereas, others do not. Where preload was included in the test, it shall be ensured that the preload stress from the test envelops the stress due to gravity. Where preload was not included in the test, the seismic demand stress shall include both seismic and dead load contributions.

The fatigue capacity of a component can be evaluated using a cumulative usage factor:

$$\frac{5N_{EQ}}{N_{OBE}} + \frac{N_{EQ}}{N_{SSE}} \leq 1.0 \qquad [4]$$

where

N_{EQ} = Total number of maximum peak to peak load/stress cycles per earthquake, N_{EQ} = 10 (Ref 6).

N_{OBE} = Allowable number of load/stress cycles per Operating Basis Earthquake (OBE) which provides a factor of safety of 1.5 on cycles.

N_{SSE} = Allowable number of load/stress cycles per Safe Shutdown Earthquake (SSE) which provides a factor of safety of 1.5 on cycles.

In this evaluation, OBE load is 80% of SSE load. 2

From Table 3.2 of Reference 3, the moment associated with an apparent elastic stress value of 50 ksi (from elastic system analysis) on Unistrut P1000 and P1001 is justified as the section moment capacity for linear elastic analysis. Furthermore, since the strut survived more than 200 cycles, the 50 ksi capacity can be adequately used without explicit equation (Ref. 3) fatigue checks.

Examples of fatigue analyses are given in Appendix B for unistrut members and connections. 2

Support components which have static test data (i.e., an incrementally increasing load-deflection curve) can be evaluated using a ductility requirement. Ductility evaluations shall be performed when fatigue data is unavailable. A calculated ductility factor μ less than three is acceptable for steel members. μ is defined as

$$\Delta / \Delta_y$$

where Δ is the maximum displacement based on the applied load and is obtained from the static load-deflection curve. Δ_y is the displacement at incipient yielding of the support component.

4.2.3 Detailed Fatigue Evaluation

A detailed fatigue evaluation is similar to the 'simplified' fatigue evaluation except now a nonlinear system analysis is performed to obtain the bending moment and force on the connection and members (instead of a linear system analysis). For production work, detailed fatigue evaluations shall not be used although they are an option which could be used.

4.3 Capacity Determination

4.3.1 Member stress shall not exceed the following refined criteria allowables.

(1) Hot-Rolled Steel Members

(a) Bending or Axial (Tension)

F_b and $F_a = F_y$

where

F_y = yield stress

(b) Shear

$F_v = 0.57\ F_y$ (Ref. 2)

(2) Cold-Formed Steel Members

(a) Axial Tension

$F_a = F_{ya}$

where F_{ya} is the average yield point of the full section for cold-formed steel members. See Table 1 for values of F_{ya} of selected Unistrut members.

(b) Axial Compression

$F_a = 1.6\ F'_a$

where F'_a is given in the table entitled, "Design Loads for Axially Loaded Unistrut Columns or Compression Members" of Reference 16.

(c) Bending about the Major Axis for Single Web Sections

The allowable bending stress is dependent upon whether or not the beam is prone to lateral-torsional buckling.

- For $L^2 S_{xc}/(dI_{yc}) < 0.36\ \pi^2 EC_b/F_y$,

  $F_b = F_{ya}$

- For $0.36\pi^2 EC_b/F_y < L^2 S_{xc}/(dI_{yc}) < 1.8\pi^2 EC_b/F_y$,

  inelastic lateral-torsional buckling governs and

$$F_b = F_{ya} - \frac{F^2_{ya}}{4.05\pi^2\ EC_b} \cdot \frac{L^2}{d} \cdot \frac{S_{xc}}{I_{yc}}$$

where: F_{ya} = average yield point, ksi
E = elastic modulus, ksi
C_b = bending coefficient; conservatively $C_b = 1.0$ (see Ref. 12, Sec. 3.3)
d = section depth, inches
L = unbraced length, inches
S_{xc} = compression-side section modulus of entire section about centroidal axis perpendicular to loading plane, inches^3
I_{yc} = moment of inertia of compression portion of section about its centroidal axis parallel to the loading plane, inches^4

For particular Unistrut (or equivalent) profiles, the above formula reduces to:

P1000, $F_b = 45.9 - .0028\ L^2$

P1001, $F_b = 45.9 - .0010\ L^2$

P3300, $F_b = 48.9 - .0096\ L^2$

P5000, $F_b = 38.2 - .0011\ L^2$ 2

- for $L^2 S_{xc}/(dI_{yc}) > 1.8\pi^2\ EC_b/F_y$, elastic lateral-torsional buckling governs and

$$F_b = 0.6\pi^2\ EC_b \frac{dI_{yc}}{L^2 S_{xc}}$$

(d) Bending about the Minor Axis of Multiweb Sections

Two web sections are more stable laterally than single-web sections. Only beams bent about their strong axis show a tendency for lateral buckling. Therefore,

$F_b = F_{ya}$

(e) Shear Stress in Webs

The maximum average shear stress F_v, in kips per square inch, on the gross area of a flat web is governed by shear buckling or yielding in shear.

- For h/t less than or equal to $547/\sqrt{F_y}$, inelastic shear buckling or yielding governs and

$$F_v = 151\sqrt{F_y} / (h/t) \leq 0.4F_y$$

- for h/t greater than $547\sqrt{F_y} / (h/t)$, elastic shear buckling governs and

$$F_v = 83{,}300 / (h/t)^2$$

where: t = web thickness, inch
h = clear distance between flanges measured along the plane of the web, inch
F_y = yield stress, ksi

Where the web consists of two or more sheets, each shall be considered as a separate member carrying its share of shear.

4.3.2 Pullout and Slip of Unistrut Bolts

Allowable bolt loads for pullout F_P and slip F_S are given in Unistrut General Engineering Catalog No. 9 (Ref. 16). Those values have a factor of safety of 3. For this job, allowable pullout loads F_{PA} and slip loads F_{SA} are

$$F_{PA} = 1.6\ F_P$$

$$F_{SA} = 1.6\ F_S$$

To account for interaction of slip and pullout, a quadratic interaction equation shall be used.

$$\left(\frac{f_{PE}}{F_{PA}}\right)^2 + \left(\frac{f_{SE}}{F_{SA}}\right)^2 \leq 1.0$$

where f_{PE} and f_{SE} are the calculated pullout and slip loads and F_{PA} and F_{SA} are the allowable pullout and slip loads. A quadratic interaction equation is recommended for bolted connections by the ASME Code NF-3324.6(a)(3).

4.3.3 Conduit Clamps

$F_a = F$

where F is the allowable design load. Table 2 gives allowable design loads for two-hole clamps (which provide a safety factor of 1.5). For one-hole clamps, Reference 14 gives allowable design loads based on tests. For interaction of longitudinal, transverse, and pullout loads, the following interaction equation shall be used.

$$\left(\frac{f_L}{F_L}\right)^2 + \left(\frac{f_T}{F_T} + \frac{f_P}{F_P}\right)^2 \leqslant 1.0$$

where f_L, f_T, and f_P are the calculated longitudinal, transverse, and pullout loads and where F_L, F_T, and F_P are the allowable loads. This interaction equation was developed in a test program conducted by Impell for Southern California Edison for conduit clamps (Ref. 19).

4.3.4 Conduit Strap

$F_a = F$

where F is the allowable design load as specified in Table 3, which provides a factor of safety of 1.5. Interaction equation in Section 4.3.3 is applied here.

4.3.5 Concrete Expansion Anchors

The allowable tensile load F_{Ta} and shear load F_{Sa} for concrete expansion anchors are defined by

$$F_{Ta} = \frac{F_T}{3}$$

$$F_{Sa} = \frac{F_S}{3}$$

where F_T is the average ultimate tensile load and F_S is the average ultimate shear load as specified in Table 4 of this Project Instruction.

For interaction of tensile and shear loads, the following interaction equation shall be used.

$$\left(\frac{f_T}{F_{Ta}}\right)^2 + \left(\frac{F_S}{F_{Sa}}\right)^2 \leq 1.0$$

where f_T and f_S are the calculated tension and shear loads, respectively.

A quadratic interaction equation has been used at other nuclear power plants (Ref. 1) and has been confirmed by correlation with experimental results (Refs. 17 and 20).

4.3.6 Welds

The allowable stress F_w on welds is

$F_W = 1.6 \times F_S$

where F_S = 13.6 ksi (E60 Electrodes)

F_S = 15.8 ksi (E70 Electrodes)

4.3.7 Fatigue

Fatigue evaluations will be performed as specified in Section 4.2.2. Appendix F contains fatigue curves which give capacities of selected components. These curves include a factor of safety of 1.5 on cycles.

4.3.8 Ductility

Ductility evaluations shall be performed as described in Section 4.2. The capacity will be based on a ductility ratio of 3 where ductility is defined by displacement.

5.0 RESULTS AND DELIVERABLES

5.1 Review of Failed Supports

Calculation packages will be prepared to document the review of and implementation of the refined criteria for the 233 "failed" supports. These results will be summarized in tabular form so that trends can be seen and conclusions drawn. The format should be such that TUGCO's management can understand the objective and results of our work.

5.2 Review of Passed Supports

Summary sheets will be prepared to document the margins of the supports Gibbs & Hill qualified using their criteria. The format of this summary should clearly indicate which supports have a high margin and therefore can be qualified by inspection rather than by engineering evaluation.

6.0 QUALITY ASSURANCE AND CHECKING

Impell personnel shall perform all work in accordance with Impell's QA program. Gibbs & Hill's work and TUGCO's walkdown information shall be treated as design input. At the close of the job, this information shall be verified as final and approved design input before Impell's work is approved.

REFERENCES

1. USNRC Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants, NUREG-0800, US Nuclear Regulatory Commission, Washington, DC, July, 1981 (Section 3.7.2).

2. Impell Calculation No. I-EORS-GENERAL, SCE/SONGS 1, Evaluation of Raceway Supports, Rev. 0, Job No. 0310-077-1355, Impell Corp., Walnut Creek, California, October 8, 1985.

3. URS/BLUME Report No. 8050, "Analytical Techniques, Models and Seismic Evaluation of Electrical Raceway System", Revision 0, prepared for SEP Owners Group, URS/BLUME, San Francisco, California, August 26, 1983.

4. PG&E Design Criteria for Seismic Review of Class 1E Electrical Raceway Supports at Diablo Canyon Power Plant DCM No. C-15, Pacific Gas and Electric Co., San Francisco, California, Rev. 4, June 15, 1984.

5. Test Report for Static and Cyclic Testing of Train C Conduit Support Components for Comanche Peak Steam Electric Station, Test Report No. A-707-86, dated February 14, 1986. 2

6. IEEE Standard 344-1975, "IEEE Recommended Practices for Seismic Qualification of Class 1E Equipment for Nuclear Generating Stations," Institute of Electrical and Electronics Engineers, New York, NY, 1975.

7. Gibbs & Hill Calculation Books No. DMI-17C - Set 3, 4, 7, and 8; DMI-24P-31 to 52; and SCS-238 - Set 1 to Set 8, Gibbs & Hill, Inc., New York, NY (Preliminary).

8. ASME Boiler and Pressure Vessel Code, Section III, Appendix F, American Society of Mechanical Engineers, New York, NY, 1983 Edition.

9. Impell Calculation No. I-EOTC-01, "Calculation of Allowables of Various Components," Rev. 0, Job No. 0310-077-1355, Impell Corp., Walnut Creek, Calif., Oct. 1985.

10. Project No. 85-1903.03/.11, "Preliminary Summary of Static Testing Results - Train C Conduit Supports," Corporate Consulting Co, Ltd., Research Triangle Park, North Carolina, Dec. 23, 1985.

11. Yu, Wei-Wen, Cold-formed Steel Structures, McGraw-Hill, New York, 1973, p. 126.

12. Specification for the Design of Cold-formed Steel Structural Members, American Iron and Steel Institute, Washington, DC, Sept. 3, 1980.

13. Detroit Testing Lab Inc. Report No. 103341-C-3, Prepared for Unistruct Division, Michigan (Jul. 22, 1983), and Unistrut Calculation, "Determinant Allowable Design Stresses Utilizing Cold Work Pre Galvanized Finish", Unistrut Division, GTE Products Corp., Wayne, Michigan, Oct. 1983.

14. Gibbs & Hill Calculation No. SCS-223C, Set 1, Test Results for Lighting Conduit Supports--One-hole Clamp," Rev. 0, Gibbs & Hill, Inc., New York, NY, Oct. 20, 1982.

15. Blevins, RD, Formulas for Natural Frequency and Mode Shape, Van Nostrand Reinhold, New York, 1979.

16. General Engineering Catalog No. 9, Unistrut Division, GTE Products Corp., Wayne, Michigan, 1980.

17. Czarnecki, R.M., "Development of Anchorage Guidelines for Nuclear Plant Equipment," Preliminary Document, Electric Power Research Institute, Palo Alto, Calif.

18. "Electrical Raceway Supports Seismic Reevaluation Criteria," San Onofre Nuclear Generating Station Unit 1, Southern California Edison Co., Rosemead, Calif., Aug. 1982.

19. "Static and Dynamic Tests of One-Bolt Conduit Clamps," Impell Report No. 01-0310-1469, prepared for Southern California Edison Co., Impell Corporation, Walnut Creek, Calif., Nov. 1985.

20. Clatto, R.D., and Boentgen, R.R., "Strength of Concrete Expansion Anchor for Pipe Supports," Effect of Piping Restraints on Piping Integrity, ASME Century II Engineering Technology Conference, San Francisco, Calif., Aug. 1980.

Table 1

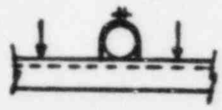
AVERAGE YIELD STRENGTH OF UNISTRUT MEMBERS

UNISTRUT	F_{YA} (psi)*
P1000	45,977
P1001	45,977
P3000	46,825
P3300	48,977
P5000	38,276

* F_{YA} is based on Equation 3.1.1-1 of Reference 12 and on Reference 13.

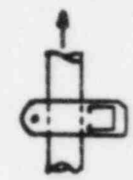
Table 2

ALLOWABLE DESIGN LOAD FOR CONDUIT CLAMPS



Pull-out

Transverse



Longitudinal

CONDUIT DIAMETER	PIPE CLAMP NO.	PULL OUT (lbs.)	TRANSVERSE SLIP (lbs.)	LONGITUDINAL SLIP (lbs.)
3/8"	P-1109B	1357	167	133
1/2"	P-1111B	1200	233	133
3/4"	P-1112B	1733	250	233
1"	P-1113B	1917	450	415
1-1/4"	P-1114B	1600	300	233
1-1/2"	P-1115B	2333	281	200
2"	P-1117B	2800	348	200

References: Unistrut Corp., Test reports C-13-H and C-36-A dated 10/6/77 and 5/13/77.

Table 2 (Continued)

CONDUIT DIAMETER	PIPE CLAMP NO.	PULL OUT (lbs.)	TRANSVERSE SLIP (lbs.)	LONGITUDINAL SLIP (lbs.)
2-1/2"	P-1118B	2600	633	367
3"	P-1119B	2800	700	648
3-1/2"	P-1120	3500	733	681
4"	P-1121	3967	1233	1100
5"	P-1123	3300	633	467
6"	P-1124	3200	881	615

Notes: (1) This Project Instruction addresses supports for conduit 2" or under; however, supports for these conduits may also support larger conduit.

(2) Above values incorporate a factor of safety applied to the 1.5 ultimate load.

Table 3

ALLOWABLE DESIGN LOADS FOR 2-BOLT CONDUIT CLAMPS

CONDUIT DIAMETER	PIPE CLAMP NO.	PULL OUT (lbs.)	TRANSVERSE SLIP (lbs.)	LONGITUDINAL SLIP (lbs.)
1/2"	P-2558-05	1853	960	353
3/4"	P-2558-07	2010	853	507
1"	P-1558-10	1733	840	250
1-1/4"	P-2558-12	1687	573	300
1/-1/2"	P-2558-15	1933	760	473
2"	P-2558-20	3333	3667	693
2-1/2"	P-2558-25	5400	4933	693
3"	P-2558-30	5567	4733	1527
4"	P-2558-40	5567	4667	847

Note: See Table 2 for orientation of clamps and load directions.

References: Unistrut Corp., Test reports C-13-H and C-36-A dated 5/13/77 and 10/6/77.

Table 4

AVERAGE ULTIMATE TENSILE AND SHEAR LOADS FOR KWIK-BOLTS

CONCRETE STRENGTH		2000 psi		4000 psi		6000 psi	
Diameter	Embedment	Tension (lbs.)	Shear (lbs.)	Tension (lbs.)	Shear (lbs.)	Tension (lbs.)	Shear (lbs.)
1/4"	1-1/8"	975	1653	1455	2612	1755	2389
	1-1/2"	1875	1653	2225	2612	2935	2389
	1-3/4"	2275	1653	2700	2612	3300	2389
	2"	2525	1653	3125	2612	3350	2389
	2-1/4"	2680	1653	3310	2612	3350	2389
	2-1/2"	2800	1653	3350	2612	3350	2389
3/8"	1-5/8"	2245	3748	2355	5107	2810	6266
	2"	2725	3748	3025	5107	3650	6266
	2-1/2"	3075	3748	3900	5107	4450	6266
	3"	3300	3792	4300	5419	5000	6266
	3-1/2"	3425	3792	4600	5419	5275	6266
	4"	3520	3792	4750	5419	5375	6266
	4-1/2"	3580	3792	4800	5419	5400	6266
1/2"	2-1/4"	4545	7444	5510	8316	6845	9341
	2-3/4"	5800	7444	7200	8316	9800	9341
	3-1/2"	7000	7444	9450	8316	13200	9341
	4-1/2"	7275	8897	11225	10232	14550	11522
	5-1/2"	8250	8897	12050	10232	15150	11522
	6"	9000	8897	12300	10232	15300	11522

*

Reference: Hilti Architects and Engineers Anchor and Fastener Design Manual dated 1/84.

Table 4 (Continued)

AVERAGE ULTIMATE TENSILE AND SHEAR LOADS FOR KWIK-BOLTS

CONCRETE STRENGTH		2000 psi		4000 psi		6000 psi	
Diameter	Embedment	Tension (lbs.)	Shear (lbs.)	Tension (lbs.)	Shear (lbs.)	Tension (lbs.)	Shear (lbs.)
5/8"	2-3/4"	5410	11198	6600	11562	7700	13500
	3-1/2"	6250	11198	9100	11562	9560	13500
	4-1/2"	7000	11198	12000	11562	14500	13500
	5-1/2"	7550	13378	14300	15437	20300	15437
	6-1/2"	8025	13378	16000	15437	21000	15437
	7-1/2"	9000	13378	17000	15437	21000	15437
3/4"	3-1/4"	8155	13257	10150	17133	10860	18102
	4"	9700	13257	13400	17133	13700	18102
	5"	11700	13257	16500	17133	17600	18102
	6"	13800	15195	18000	18466	22500	21009
	7"	15800	15195	21000	18466	23600	21009
	8"	16000	15195	23000	18466	23600	21009
	9"	16000	15195	23500	18466	23600	21009

Table 4 (Continued)

AVERAGE ULTIMATE TENSILE AND SHEAR LOADS FOR KWIK-BOLTS

CONCRETE STRENGTH		2000 psi		4000 psi		6000 psi	
Diameter	Embedment	Tension (lbs.)	Shear (lbs.)	Tension (lbs.)	Shear (lbs.)	Tension (lbs.)	Shear (lbs.)
1"	4-1/2"	14000	27355	16000	26879	20500	32112
	5"	15500	27355	18900	26879	23441	32112
	6"	17600	27355	23441	26879	23441	32112
	7"	18200	27355	23441	26879	23441	32112
	8"	18200	27355	23441	34491	23441	36394
	9"	18200	27355	23441	34491	23441	36394
	10"	18200	27355	23441	34491	23441	36394
1-1/4"	5-1/2"	19000	36750	23000	35680	31200	45195
	6-1/2"	21600	36750	27100	35680	36500	45195
	7-1/2"	23600	36750	31100	35680	42000	45195
	8-1/2"	25100	39843	34600	35680	44400	47098
	9-1/2"	26200	39843	37800	35680	44400	47098
	10-1/2"	26800	39843	40900	35680	44400	49596

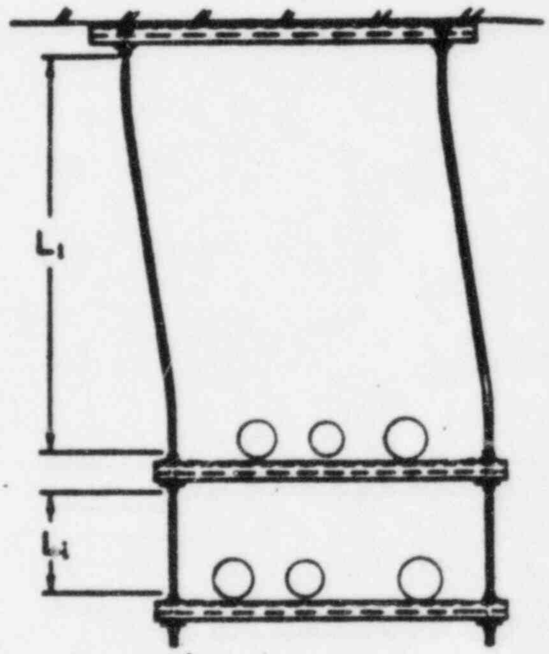
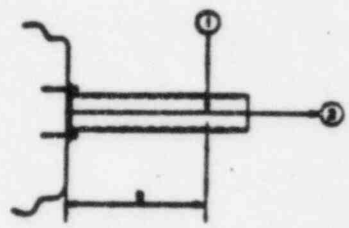
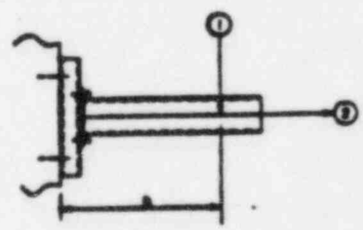


Figure 1 TRANSVERSE BEHAVIOR OF A TRAPEZE ROD HANGER

Type (1a): Cantilever Welded Foot Hanger



Type (1b): Cantilever Welded Foot Member with Unistrut Header



Type (1c): Cantilever Member with Unistrut Header and P1026 or Similar Fitting

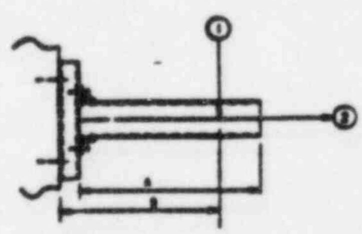
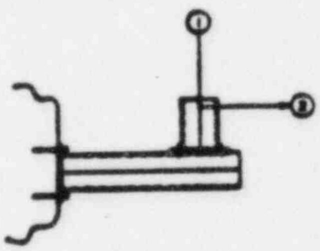


Figure 2 GENERIC SUPPORT CATEGORIES

Type (2a): Double Cantilever Supports Using Welded Foot Members



Type (2b): Double Cantilever Supports with Unistrut Header Using Welded Foot Members

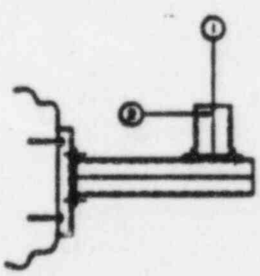
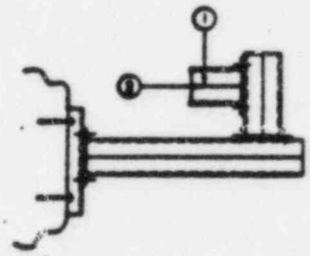


Figure 2 GENERIC SUPPORT CATEGORIES (Continued)

Type (3a): Triple Cantilever Supports with Unistrut Header Using Welded Foot Members



Type (3b): Triple Cantilever Supports with Unistrut Header Using Unistrut Fittings

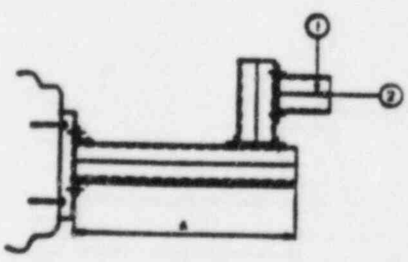
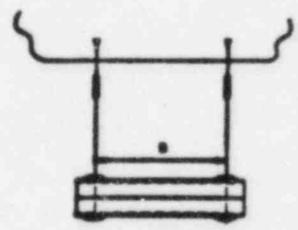


Figure 2 GENERIC SUPPORT CATEGORIES (Continued)

Type (4a): Trapeze Supports Attached to Ceiling or Underside of Beam



Type (4b): Trapeze Supports Attached to Side of Beam Using P1026 Connection

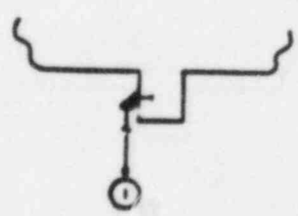
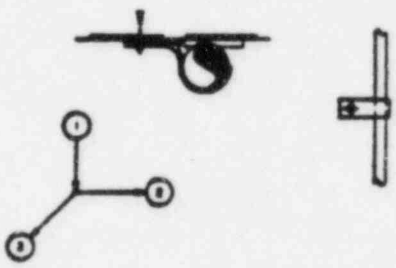


Figure 2 GENERIC SUPPORT CATEGORIES (Continued)

Type (5): One-Hole Pipe Strap



Type (6): Two-Hole Clamp

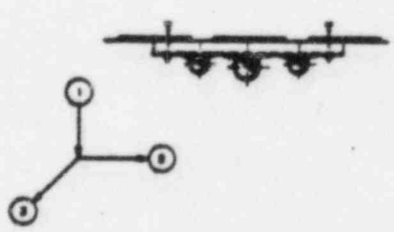
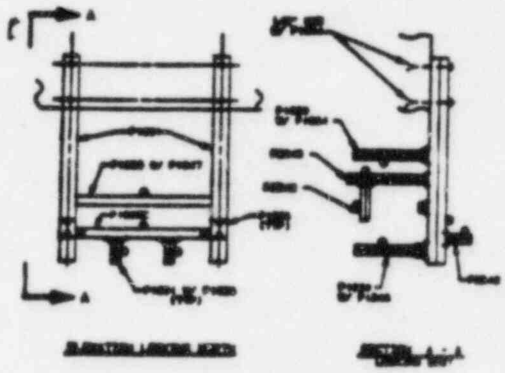


Figure 2 GENERIC SUPPORT CATEGORIES (Continued)

Type 7 Special Supports



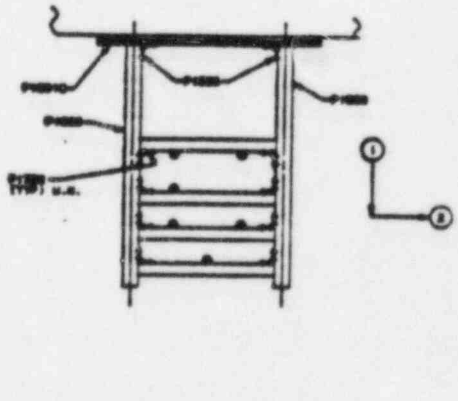
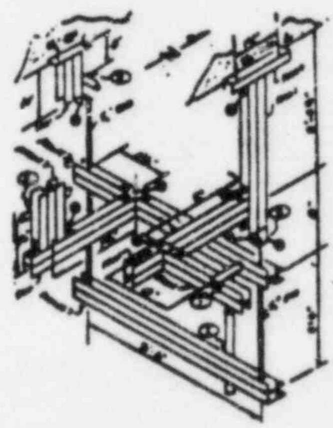


Figure 2 GENERIC SUPPORT CATEGORIES (Continued)

Type (8): Supports with Large Seismic Displacements



2

Figure 2 GENERIC SUPPORT CATEGORIES (Continued)

APPENDIX A

CHECKING CRITERIA

OBJECTIVE

This procedure defines the checking process used in safety-related analyses performed by the Advanced Engineering Division. This procedure contains general guidelines for checking, documentation of checking, and resolution of checker's comments, and is consistent with QAP 3.6. This procedure may be supplemented or amended as required for specific project use as long as the resulting checking procedure remains consistent with QAP 3.6. The specific analyses that warrant additional checking criteria must be clearly addressed in the Project Instructions for the subject task. The instructions must be specific about each calculation (or type of calculation) to be produced. A detailed checklist addressing the contents of the calculation and incorporating all significant assumptions should be included (see Attachment B). The specific checking criteria should clearly define milestones for design review and checking. Note that the Project Instructions, including the checking criteria, must be verified by the design reviewer.

RESPONSIBILITIES

A. SECTION MANAGER

The Section Manager is responsible for ensuring that the checking methods used in his section are in accordance with sound engineering practice and the requirements of this procedure.

B. PROJECT ENGINEER

The Project Engineer is responsible for directing the checking process, providing guidelines for checking, designating independent checkers, verifying the adequacy of the individual checks performed, and assuring that the design review required by this procedure is complete.

On a project basis, the Project Engineer shall identify any project-specific guidelines to be used and shall include or reference them, along with this procedure, as part of the Project Instructions. The Project Engineer shall have all checking procedures included in the technical design reviews.

Additionally, the Project Engineer shall review the checking process and discuss checking criteria as part of the project training.

C. DESIGN REVIEWER

The design reviewer shall ensure that the established checking procedures are adequate to assure that the work is properly performed and verified.

D. ENGINEER (ORIGINATOR)

The originating Engineer is responsible for producing accurate and fully verifiable results for the task assigned. The originator is additionally responsible for resolving the checker's comments, as appropriate.

E. CHECKER

The designated checker is responsible for performing and documenting his check in accordance with the requirements of this procedure, project-specific guidelines and criteria, and QAP 3.6.

F. JOINT RESPONSIBILITIES

It should be noted that the responsibility for the correctness of any item is shared by the originator and the checker. Their signatures on an item indicate that they both agree that the item is correct. Once any item has been completely checked and signed off, it should not be altered by anyone without issuing a revision of the item.

GENERAL CHECKING CRITERIA

The purpose of an engineering check is to provide assurance that a task is performed and documented thoroughly and that the results are correct and reasonable. The check requires more than simply verifying numbers on a page (e.g., the correctness of a detailed calculation). It includes review of the purpose, theoretical methods, and assumptions used in the item being checked. The check also covers the use of correct design input information and references along with complete, clear, and traceable documentation.

Engineers who are assigned technical tasks are directed to use approved technical procedures or Project Instructions, which shall include or reference these as well as any project-specific checking criteria. These criteria are sufficiently detailed to direct the checker to review those elements of the work that will lead to a safe design.

A guideline for checking criteria is presented in Attachment A. This list serves as a guideline for the thought process of the checker while the analysis item is reviewed. The originator should also review this criteria list to ensure that an analysis item is complete and ready to be checked. The originator's review will streamline the checking process and minimize the overall effort required to produce a quality analysis.

Note that there are no short cuts in checking. The checking process is an integral part of producing a high-quality analysis. Impell is contractually committed to its clients to provide this quality.

GUIDELINES AND INSTRUCTIONS FOR CHECKING

A. Independent engineering checking is performed at the task level by technically competent checkers who are familiar with the AED technical analysis procedures and with any project-specific procedures that apply to the task being checked. Also, checkers should not have participated in the specific work effort or distinct operation to be checked.

B. More specific criteria are provided by checking sheets for the following items:

Attachment A: Checking Criteria Guidelines

Attachment B: Calculation File Organization and Contents

C. Project Instructions on checking should be prepared and issued by the Project Engineer for AED project-specific use. Parts or all of this AED checking procedure may be referenced in the Project Instruction.

D. The originator and checker of a task, such as a calculation or a drawing, should be designated by the Lead Engineer. The originator is responsible for producing accurate and fully verifiable results for the task assigned and for resolving the checker's comments, as appropriate. The checker is responsible for performing and documenting his check in accordance with the established Project Instruction on checking.

E. The checker shall trace the impact of any identified errors or discrepancies throughout the calculation, drawing, or other items checked. Once resolved with the originator, it is not necessary to record the identified error or discrepancies on a separate sheet. The original calculation, drawing, or other items checked should be corrected <u>prior</u> to signing "checked."

F. When checklists and prepared data forms are used, no item should be left blank. If a particular item is not applicable, an N/A should be entered in that space.

G. In cases where the originator and the checker cannot resolve comments, the Proejct Engineer shall resolve any outstanding items.

H. Alternate calculations may be used to verify the item or items to be checked (e.g., to verify the calculation written on an HP computer/calculator). Alternate calculations used for checking shall be included as part of the calculation package. The checker shall sign and date the alternate calculation in the "by" space provided, and shall sign and date the checked calculation sheets in the "checked" space. The check space on alternate calculations shoud be marked N/A.

I. If any identified errors or discrepancies have a potentially generic impact on the project, it is the responsibility of the checker to inform the Project Engineer. It is the responsibility of the Project Engineer to review and resolve these findings.

ATTACHMENT A

CHECKING CRITERIA GUIDELINES

1. Are the title, purpose, and function of the item checked adequately described?

2. Is the method clearly stated?

3. Are assumptions identified? Are open items flagged for subsequent verification where necessary?

4. Are design bases and references correctly selected and incorporated?

5. Are applicable codes, standards, and regulatory requirements identified?

6. Can the analytical steps involved be verified without recourse to the originators?

7. Is each sheet identifiable to its place in the calculation and to the calculation/problem number?

8. Are all markings legible, and identifiable as to purpose or function?

9. Is each sheet traceable to originator, date, and job or equivalent control number?

10. Does the calculation clearly reference any final computer runs used?

11. Do final computer runs include an input listing and output?

12. Are computer results reasonable based on inputs and methodology?

13. Are final computer runs traceable back to the calculation?

14. Are final computer runs identifiable by a unique number or code?

15. Are calculation results consistent with inputs, technical procedures, and design criteria?

16. Are revisions clearly documented?

ATTACHMENT B

CALCULATION FILE ORGANIZATON AND CONTENTS

Typical Checklist Guidelines for a Finite Element Analysis:

1. Introduction

    Statement of Problem
    Proposed Method of Analysis

2. Description of Component System

    Geometry
    Materials of Construction
    Function

3. Design Criteria

    KNRC
    ASME
    Other

4. Method of Analysis

    Analytical Method
        Computer Programs
    Major Assumption & Justification

5. Modeling

    Refinement
        Nodes Element Mass Points
    Boundary Conditions
        Supports
    Enveloping Techniques

6. Loading Input

    Thermal-Hydraulic
    Seismic
    Pressure
    LOCA
    Other

7. Structural Analysis

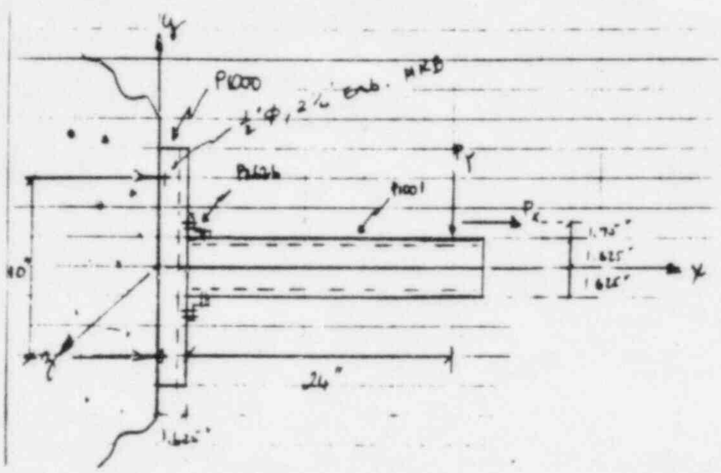
    Tables & Plots Summarization
    Critical Responses
    Discussion

8. Code Compliance

9. Conclusions & Recommendations

APPENDIX B

SAMPLE CALCULATIONS



Gravity P_Y = 100 #

Seismic P_Y = 450 #
(SSE)

P_X = 100 #

conduit size = 3" ϕ

Check P1001 Member

$M_Z = 100 \times 24 + 450 \times 24 + 100 \times (1.75 + 1.625)$

$= 2400 + 11{,}138 = 13{,}538$ #-in
grav. seismic
(SSE)

$$\sigma_b = \frac{M}{S} = \frac{13{,}538}{.572} = 23{,}668 \text{ psi}$$

$$\text{Axial load allowable} = 22{,}500 \times \frac{2.4}{3.0} \times 1.6 = 28{,}800 \text{ psi}$$

Unistrut
Cat. #10R

$$\text{Interaction} = \frac{100}{28{,}800} + \frac{23{,}668}{45{,}977} = .52 < 1.0$$

Table 1 of
Proj. Instr.

P1001 is qualified.

Check Clip P2626

Tension load on bolts due to gravity

$$T_{Gravity} = \frac{100 \times 24}{1.625 + 1.625 + 1 + 1} = 457 \text{ \#}$$

The P2626 moment versus rotation curve has a preload of 1000 pounds. Since the effect of preload on the connection is greater than that due to gravity, only seismic load will be considered in the fatigue check.

M_{SSE} = 11,138 lb-in $\rightarrow$ θ = .05 rad $\rightarrow$ 70 cycles

For OBE event, M_{OBE} = .8 M_{SSE} = 11,138 x .8 = 8,910 lb-in

From P2626 fatigue data,

M_{OBE} = 8,910 lb-in $\rightarrow$ θ = .035 rad $\rightarrow$ 200 cycles

$$\text{Usage factor} = \frac{5 \times \text{OBE}}{400} + \frac{1 \text{ SSE}}{70} = \frac{5 \times 10}{200} + \frac{10}{70} = .39 < 1.0$$

P2626 is qualified.

Check P1000 member

$$\text{Bending } \sigma_b = \frac{M}{S} = \frac{100 \times (24 + 1.625) + 450\ (24 + 1.625) + 100\ (1.75 + 1.625)}{S}$$

$$= \frac{2,563 + 11,531 + 337}{S} = \frac{14,431}{S} = \frac{14,431}{.203}$$

$$= 71,089 \text{ psi} > 45,977 \text{ psi}$$

Check fatigue.

P1000 was tested without preload; thus, gravity load will be included in fatigue check. 2

N_{SSE} = 200 cycles

σ_bOBE = Gravity + .8 SSE

= 2,563 + .8 (11,531 + 337) = 12,057 psi

From P1000 fatigue curve, N_{OBE} = >1,000 cycles

$$\text{Usage factor} = \frac{5 \text{ OBE}}{1,000} + \frac{1 \text{ SSE}}{200} = \frac{50}{1,000} + \frac{10}{200} = .10 < 1.0$$

P1000 is qualified.

Check 1/2" Hilti Bolt, 2-1/4" Embedment

$$\text{Pull out} = \frac{(100 \times (24 + 1.625) + 450\ (24 + 1.625) + 100\ (1.75 + 1.625))}{10} + \frac{100}{2}$$

= 1,493 #

$$\text{Shear} = \frac{100 + 450}{2} = 275 \text{ \#}$$

Allowable

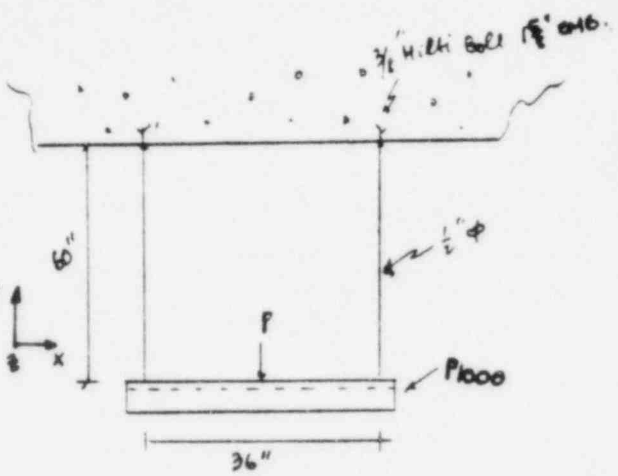
$$\text{Tension} = \frac{5,510}{3} = 1,837 \text{ \#}$$

$$\text{Shear} = \frac{8,316}{3} = 2,772 \text{ \#}$$

$$\text{Interaction} = \left(\frac{1,493}{1,837}\right)^2 + \left(\frac{275}{2,772}\right)^2 = .67 < 1.0$$

Bolts are qualified.

6



Gravity = 450 #

Seismic = 1000 #

Total = 450 # + 1000 #

= 1450 #

Displacements of Conduit:

$\Delta x = 4.5"$

$\Delta z = 1.5"$

Check P1000 member

Moment due to bending:

$$M = \frac{PL}{4} = \frac{1450 \times 36}{4} = 13{,}050 \text{ lb-in}$$

$$\sigma_b = \frac{M}{S} = \frac{13{,}050}{.203} = 64{,}286 \text{ psi} > 45{,}977 \text{ psi}$$

Then check fatigue.

From P1000 weak axis fatigue curve in the Project Instruction.

No. of cycles to failure = >200 cycles

OBE Case, $M_{OBE} = M_{Grav} + .8\ M_{SSE} = \frac{450 \times 36}{4} + .8 \times \frac{1000 \times 36}{4} = 4{,}050 + 7{,}200 = 11{,}250$ lb-i

No. of cycles to failure = $>1{,}000$ cycles

$$\text{Usage factor} = \frac{5 \times OBE}{1{,}000} + \frac{1\ SSE}{200} = \frac{50}{1{,}000} + \frac{10}{200} = .10 < 1.0$$

Therefore, P1000 is qualified.

Check tension on rod

$$T = \frac{P}{2} = \frac{1450}{2} = 725\ \#$$

$$\sigma_a = \frac{725}{A} = \frac{725}{.142} = 5{,}106 \text{ psi} < F_y \text{ (36,000 ksi)}$$

Check bending on rod

Total displacement of rod

$$\Delta = \left(\Delta_x^2 + \Delta_z^2\right)^{1/2} = \left((4.5)^2 + (1.5)^2\right)^{1/2} = 4.74\ "$$

From Reference 9 of Project Instruction, the allowable displacement on 1/2" rod with tension less than 745 # is 9" which is larger than 4.74"

Rod is qualified.

Check Hilti bolts

$$\text{Pull out} = \frac{P}{2} = \frac{1450\#}{2} = 725\ \#$$

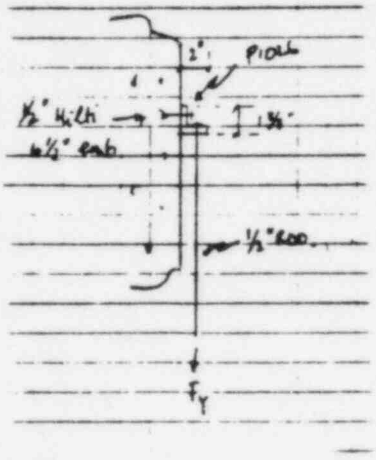
From Project Instruction, Table 4 for 3/8" HKB with 1-5/8" minimum embedment,

$$\text{Pull out}_{\text{allow.}} = \frac{2,355}{3} = 785\ \#$$

$$\text{Ratio to allowable} = \left(\frac{725}{785}\right)^2 = .85 < 1.0$$

Bolts are qualified.

SAMPLE CALCULATION 3



F_Y = 600 + 1,900 = 2,500 #
Grav. + Seismic (SSE)

Ref: (1) CCL test report, "Preliminary Summary of Static Testing Results-Train C Conduit Supports" CCL Project No. 85-1903. 03/.11 dated 12/23/85.

Check P1026 connection

From Ref. 1, test A-01, the ultimate load on P1026 is 3,900 #.

- compare to $\frac{2}{3} F_u$

$\frac{2}{3}$ x 3,900 # = 2,600 # $>$ 2,500 #, then

- calculate ductility

$\mu = \frac{\Delta \text{calculated}}{\Delta \text{yield}}$

From test A-01, Δ_{yield} = .07 in.

$\Delta_{\text{calculated}}$ = .15 in.

$= \frac{.15}{.07} = 2.14 < 3$. Therefore, P1026 is qualified.

Check 1/2" ϕ rod

Tensile stress = $\frac{2,500}{A} = \frac{2,500}{.142}$ = 17,606 psi $< F_Y$ (36,000 psi)

Rod is qualified.

Check 1/2" anchor bolt

Shear = 2,500 #

Pullout = $\frac{2,500 \times 1}{.9375}$ = 2,667 #

Allowables:

$$\text{Shear} = \frac{10{,}232}{3} = 3{,}411\ \#$$

$$\text{Pull out} = \frac{11{,}225}{3} = 3{,}742\ \#$$

$$\text{Interaction} = \left(\frac{2{,}500}{3{,}411}\right)^2 + \left(\frac{2{,}667}{3{,}742}\right)^2 = 1.05$$

Bolts failed by 5 percent.

APPENDIX C

CALCULATIONS, FILE ORGANIZATION, AND FORM

CALCULATION/PROBLEM COVER SHEET

Calculation/Problem No:

Title: Evaluation of Train C Conduit Using Refined Criteria

Client: TUGCO

Project: CPSES-Unit 1

Job No: 0210-051-1355

Design Input/References:

- Contained Within

Assumptions:

- Contained Within

Method:

- Contained Within

Remarks:

REV. NO.	REVISION	APPROVED	DATE
0	Original Issue		

QAM-3.4-E 11/81

Sheet ____ of ____

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					IMPELL CORPORATION	JOB NO	PAGE
REV	BY	DATE	CHECKED	DATE		CALC NO	OF

COMANCHE PEAK STEAM ELECTRIC STATION

TRAIN C CONDUIT
TWO INCH AND UNDER

Criteria Document--Appendices

REGULATORY GUIDE 1.29 ISSUE

(T.R.T. ISSUE 1.c)

Prepared for:

Texas Utilities Generating Company

Prepared by:

Impell Corporation
Gibbs and Hill, Inc.
Ebasco Services

Appendices to
Impell Report No. 01-0210-1479
Revision 0

March, 1986

LIST OF APPENDICES

APPENDIX A

SEISMIC EVALUATION OF TRAIN C CONDUIT USING REFINED CRITERIA

IMPELL CORPORATION

PROJECT INSTRUCTION

TITLE: SEISMIC EVALUATION OF TRAIN C CONDUIT USING REFINED CRITERIA

INSTRUCTION NUMBER: 0210-051-01	PAGE 1 OF 34

CLIENT: Texas Utilities Generating Company

PROJECT: Comanche Peak

JOB NUMBER(S): 0210-051-1355

DIVISION(S): Advanced Engineering Division

REV.	ISSUE DATE	PREPARED	APPROVED
0	1/22/86	Kim Hoang	[illegible]
1	1/31/86	Kim Hoang	[illegible]
2	3/6/86	Kim Hoang	[illegible]

REFERENCES:

1. Impell Project Instruction No. 001, "Evaluation of Train C Conduit Using Refined Criteria," Revision 0, Job Number 0210-051-1355, Impell Corporation, Walnut Creek, CA, January, 1986.

2. Manual of Steel Construction, 7th Edition (Specification for the Design, Fabrication and Erection of Structural Steel for Buildings, Feb. 12, 1969 with Supplements 1, 2, and 3 through 1973), American Institute of Steel Construction, New York, NY, 1970.

3. Gibbs and Hill Calculation No. __________, Rev. ___, dated ________, Gibbs and Hill, Inc., New York, NY.

					TUGCO	
					Eval. of Train C Conduit Using Refined Criteria	
					IMPELL CORPORATION	JOB NO 0210-051-1355 / CALC NO
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I. PURPOSE

The purpose of this calculation is to reevaluate the failed Train C conduit supports (approximately 230 supports) using Refined Criteria as stated in [1].

Additionally, all 2,413 supports evaluated by Gibbs and Hill will be reviewed to quantify their margin of safety (capacity/load) and to justify screening levels to be used by walkdown teams at a later date.

					TUGCO
					Eval. of Train C Conduit Using Refined Criteria
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II. ANALYSIS PROCEDURE

All supports are reviewed. Supports which were qualified by Gibbs and Hill are reviewed for their margin of safety. Supports which failed by using Gibbs and Hill criteria will be reviewed using refined criteria for qualification. If supports are qualified (using existing support loads and refined criteria), no further evaluation is required, or method 1 or 2 is proceeded.

Method 1 - Equivalent Static Analysis

Step 1: Review Gibbs and Hill Analysis to determine whether or not support loads can be reduced.

Step 2: If support loads cannot be reduced, go to Step 3, or,

- Calculate conduit system lower bound frequency, if necessary to determine the actual load.
- Calculate support loads. For vertical support, the total load is the algebraic combination of gravity and seismic loads.

Step 3: Evaluate support using refined criteria stated in [1] for qualification.

A table of support loads, support capacities, anc support status will be provided

Method 2 - Elastic Response Spectra Analysis

When the conduit system is too complex to analyze using the equivalent static analysis method, it will be reanalyzed using the NASTRAN (or equivalent) computer program. In this reevaluation, modifications to the previous computer model will be made to ensure that the revised model realistically represents the as-built Train C conduit system.

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1.0 Scope of Work

This calculation contains Train C conduit sample No. ______ with a total of ______ supports. ______ supports were overstressed using Gibbs and Hill criteria. These supports will be reevaluated using refined criteria in Section 2.0 of this calculation. List of supports being reevaluated are given in Section 2.0.

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1.1 GIBBS AND HILL SUPPORT STATUS (Before Refined Criteria Applied)

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2.0 <u>Support Evaluation</u>

2.0 <u>List of Supports Evaluated Using Refined Criteria</u>

Support No. _______

Support No. _______

Support No. _______

Support No. _______

Support No. _______

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2.2 <u>Evaluation of Supports</u>

2.2. Support No. _______

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III. **SUMMARY OF THE RESULTS**

The results of Train C conduit supports using refined criteria are summarized here. In addition, the margin of safety of supports which were qualified previously by Gibbs & Hill are also provided.

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SUMMARY OF SUPPORTS USING GIBBS AND HILL CRITERIA

BUILDING:________ ROOM NO.:________ ELEVATION NO.:________ CONDUIT RUN NO.:________

SAMPLE NO.:________

SUPPORT NO.	SUPPORT TYPE	MAX. CONDUIT SIZE (IN)	NO. OF CONDUITS	MAX. SPAN LENGTH (ft.)	DIRECTION	EVALUATED SUPPORT LOAD (1) (lbs.)	SUPPORT CAPACITY (2) (lbs.)	RATIO (2)/(1)	CRITICAL MEMBER	BOLT SIZE RATIO TO EXISTING LOAD

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SUMMARY OF SUPPORTS USING REFINED CRITERIA

BUILDING:________ ROOM NO.:________ ELEVATION NO.:________ CONDUIT RUN NO.:________

SAMPLE NO.:________

SUPPORT NO.	SUPPORT TYPE	MAX. CONDUIT SIZE (IN)	NO. OF CONDUITS	MAX. SPAN LENGTH (ft.)	DIRECTION	EVALUATED SUPPORT LOAD (1) (lbs.)	SUPPORT CAPACITY (2) (lbs.)	RATIO (2)/(1)	CRITICAL MEMBER	BOLT SIZE RATIO TO EXISTING LOAD

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APPENDIX D

PROCEDURES FOR REVIEWING PASSED SUPPORTS

Summary sheets shall be prepared for all "passed" supports in Gibbs & Hill's random and engineering samples. Gibbs Hill's calculation packages shall be used to complete the summary sheets. The purpose of the summary sheets is to quantify the seismic design margin for all passed supports. The procedure is as follows:

1. Review Gibbs & Hill calculation package and identify: (a) sample number, (b) identifying information (bldg., room, elevation, etc.), (c) phase of analysis (1, 2A, 2B), (d) support number and description, (e) conduit sizes and numbers, (f) span lengths (if available), (g) applied loads (including direction), and (h) support capacity if included. If the support is qualified by calculating member stresses and comparing these to allowables, enter these rather than piping loads. Calculate ratio of design capacity (i.e., allowable) to seismic demand (i.e., applied load or stress) and enter on sheet.

   Critical members shall be entered if Gibbs & Hill calculation indicates which component is critical.

2. For supports qualified by reference to generic tables of support allowables or other Gibbs & Hill calculations, list table number or reference in "ratio" column or as a footnote. To determine ratio of allowable to applied load, find allowable load from the table considering the support geometry, dimensions, member size, and anchor bolt dimensions as contained in the support walkdown or sketch in Gibbs & Hill support calculation. Select entry which bounds the loading conditions and results in approximately even margins for each applied load. Calculate minimum ratio of applied load to allowable load. The ratio (which will be greater than 1) will be entered as greater than the minimum ratio. The result is a lower bound of the design (allowable) capacity-to-demand ratio. Since the tables do not identify the critical member, this may be left blank unless the analyst is familiar with the support type and can provide the critical member by engineering judgement. The anchor bolt ratio (which is less than 1), is the inverse of the design allowable to seismic load (or demand) ratio.

3. Supports qualified in Gibbs & Hill calculations by reference to generic tables vary from exact conformance to the support geometry indicated in the table, and thus require some interpretation. As an option, the geometry may be briefly noted on the summary sheet. The support type should include a brief description of the support and a generic support type number from Figure 2 of this Project Instruction. Supports that are similar to, but not exactly like, generic types should be entered as a variation of the appropriate type. Combinations of support types may also be entered (i.e., comb. type 4a/4b for trapeze with one Hilti Kwik Bolt and one P1026 connection). For variations and combinations, the support geometry should be conservatively modeled when determining allowable load from Gibbs & Hill generic tables.

4. Seismic calculation S-0910 supports shall be identified, and the margin need not be entered in the summary sheet. Special cases (such as non-Unistrut frames) shall be noted.

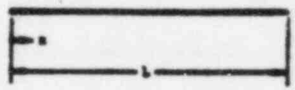
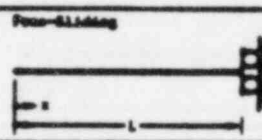
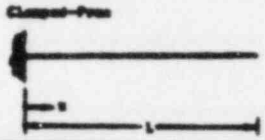
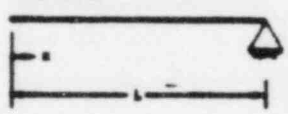
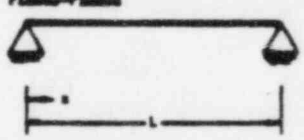
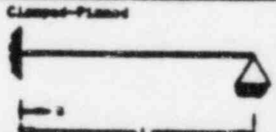
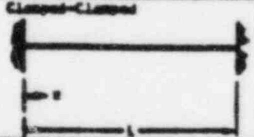
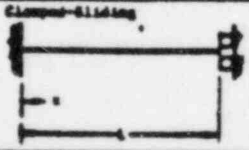
APPENDIX E

VIBRATION FREQUENCIES FOR SIMPLE STRUCTURAL ELEMENTS

Table 8-1. Single-Span Beams.

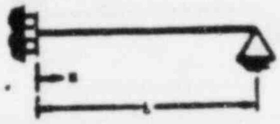
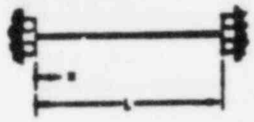
Notation: x = distance along span of beam; m = mass per unit length of beam;
E = modulus of elasticity;
I = area moment of inertia of beam about neutral axis (Table 6-1); L = span of beam;
see Table 3-1 for consistent sets of units

General Frequency (hertz): $f_i = \frac{\lambda_i^2}{2\pi L^2}\left(\frac{EI}{m}\right)^{1/2}$; i=1,2,3...

Description[a]	λ_i; i=1,2,3...	Mode Shape, $\tilde{y}_i\left(\frac{x}{L}\right)$	σ_i; i=1,2,3...
1. Free-Free 	4.73004074 7.85320462 10.9956078 14.1371655 17.2787597 $(2i+1)\frac{\pi}{2}$; i>5	$\cosh\frac{\lambda_i x}{L} + \cos\frac{\lambda_i x}{L} - \sigma_i\left(\sinh\frac{\lambda_i x}{L} + \sin\frac{\lambda_i x}{L}\right)$	0.982502215 1.000777312 0.999966450 1.000001450 0.999999937 ≈1.0 for i>5 See Ref. 8-2
2. Free-Sliding 	2.36502037 5.49780392 8.63937983 11.78097245 14.92256510 $(4i-1)\frac{\pi}{4}$; i>5	$\cosh\frac{\lambda_i x}{L} + \cos\frac{\lambda_i x}{L} - \sigma_i\left(\sinh\frac{\lambda_i x}{L} + \sin\frac{\lambda_i x}{L}\right)$	0.982502207 0.999966450 0.999999933 0.999999993 0.999999999 1.0; i>5
3. Clamped-Free 	1.87510407 4.69409113 7.85475744 10.99554073 14.13716839 $(2i-1)\frac{\pi}{2}$; i>5	$\cosh\frac{\lambda_i x}{L} - \cos\frac{\lambda_i x}{L} - \sigma_i\left(\sinh\frac{\lambda_i x}{L} - \sin\frac{\lambda_i x}{L}\right)$	0.734095514 1.018467319 0.999224497 1.000033553 0.999998550 ≈1.0; i>5 See Ref. 8-2
4. Free-Pinned 	3.92660231 7.06858275 10.21017612 13.35176878 16.49336143 $(4i+1)\frac{\pi}{4}$; i>5	$\cosh\frac{\lambda_i x}{L} + \cos\frac{\lambda_i x}{L} - \sigma_i\left(\sinh\frac{\lambda_i x}{L} + \sin\frac{\lambda_i x}{L}\right)$	1.000777304 1.000001445 1.000000000 1.000000000 1.000000000 1.0; i>5
5. Pinned-Pinned 	$i\pi$	$\sin\frac{i\pi x}{L}$	—
6. Clamped-Pinned 	3.92660231 7.06858275 10.21017612 13.35176878 16.49336143 $(4i+1)\frac{\pi}{4}$; i>5	$\cosh\frac{\lambda_i x}{L} - \cos\frac{\lambda_i x}{L} - \sigma_i\left(\sinh\frac{\lambda_i x}{L} - \sin\frac{\lambda_i x}{L}\right)$	1.000777304 1.000001445 1.000000000 1.000000000 1.000000000 1.0; i>5
7. Clamped-Clamped 	4.73004074 7.85320462 10.9956079 14.1371655 17.2787597 $(2i+1)\frac{\pi}{2}$; i>5	$\cosh\frac{\lambda_i x}{L} - \cos\frac{\lambda_i x}{L} - \sigma_i\left(\sinh\frac{\lambda_i x}{L} - \sin\frac{\lambda_i x}{L}\right)$	0.982502215 1.000777312 0.999966450 1.000001450 0.999999937 1.0; i>5 See Ref. 8-2
8. Clamped-Sliding 	2.36502037 5.49780392 8.63937983 11.78097245 14.92256510 $(4i-1)\frac{\pi}{4}$; i>5	$\cosh\frac{\lambda_i x}{L} - \cos\frac{\lambda_i x}{L} - \sigma_i\left(\sinh\frac{\lambda_i x}{L} - \sin\frac{\lambda_i x}{L}\right)$	0.982502207 0.999966450 0.999999933 0.999999993 0.999999999 1.0; i>5

Reference: Blevins, R. D., Formulas for Natural Frequency and Mode Shape, Van Nostrand Reinhold; New York, 1979.

Table 8-1. Single-Span Beams. (Continued)

Natural Frequency (hertz): $f_i = \frac{\lambda_i^2}{2\pi L^2}\left(\frac{EI}{m}\right)^{1/2}$; i=1,2,3...			
Description[a]	λ_i; i=1,2,3...	Mode Shape, $\tilde{y}_i\left(\frac{x}{L}\right)$	σ_i; i=1,2,3...
9. Sliding-Pinned 	$(2i - 1)\frac{\pi}{2}$	$\cos\frac{(2i - 1)\pi x}{2L}$	—
10. Sliding-Sliding 	$i\pi$	$\cos\frac{i\pi x}{L}$	—

[a] The boundary conditions are defined mathematically in Eq. 8-6.

The dimensionless natural frequency parameters λ_i and σ_i can be numerically computed from the following formulas:

Boundary Conditions	Transcendental Equation for λ	Formula for σ_i
1. Free-free	$\cos\lambda \cosh\lambda = 1$	$\frac{\cosh\lambda_i - \cos\lambda_i}{\sinh\lambda_i - \sin\lambda_i}$
2. Free-sliding	$\tan\lambda + \tanh\lambda = 0$	$\frac{\sinh\lambda_i - \sin\lambda_i}{\cosh\lambda_i + \cos\lambda_i}$
3. Clamped-free	$\cos\lambda \cosh\lambda + 1 = 0$	$\frac{\sinh\lambda_i - \sin\lambda_i}{\cosh\lambda_i + \cos\lambda_i}$
4. Free-pinned	$\tan\lambda = \tanh\lambda$	$\frac{\cosh\lambda_i - \cos\lambda_i}{\sinh\lambda_i - \sin\lambda_i}$
5. Clamped-pinned	Same as free-pinned	
6. Clamped-clamped	Same as free-free	
7. Clamped-sliding	Same as free-sliding	

The boundary conditions are defined mathematically by Eq. 8-6. Since the longitudinal displacement of straight beams during transverse vibration is of second order ($\sim Y^2$), the presence or absence of longitudinal constraints does not affect the natural frequencies in Table 8-1.

The rigid body modes corresponding to the $\lambda = 0$ solution for the free-free, free-sliding, free-pinned, sliding-pinned, and sliding-sliding beams have been omitted from Table 8-1.

Reference: Blevins, R. D., Formulas for Natural Frequency and Mode Shape, Van Nostrand Reinhold; New York, 1979.

where

$$\eta_1 = \frac{-\sin \lambda_i \mu}{\sinh \lambda_i \mu},$$

$$\eta_2 = \frac{(\sin \lambda_i \mu \cosh \lambda_i \mu - \cos \lambda_i \mu \sinh \lambda_i \mu)(\cos \lambda_i \eta + \cosh \lambda_i \eta)}{2 \sinh \lambda_i \mu (1 + \cos \lambda_i \eta \cosh \lambda_i \eta)},$$

$$\eta_3 = \frac{-\sin \lambda_i \eta - \sinh \lambda_i \eta}{\cos \lambda_i \eta + \cosh \lambda_i \eta}.$$

Pinned-pinned-pinned [Fig. 8-5(c)]

$$\tilde{y}_\alpha(\xi) = \sin \lambda_i \xi + \eta_1 \sinh \lambda_i \xi,$$

$$\tilde{y}_\beta(\xi) = \eta_2 (\sin \lambda_i \xi + \eta_3 \sinh \lambda_i \xi), \qquad (8\text{-}16)$$

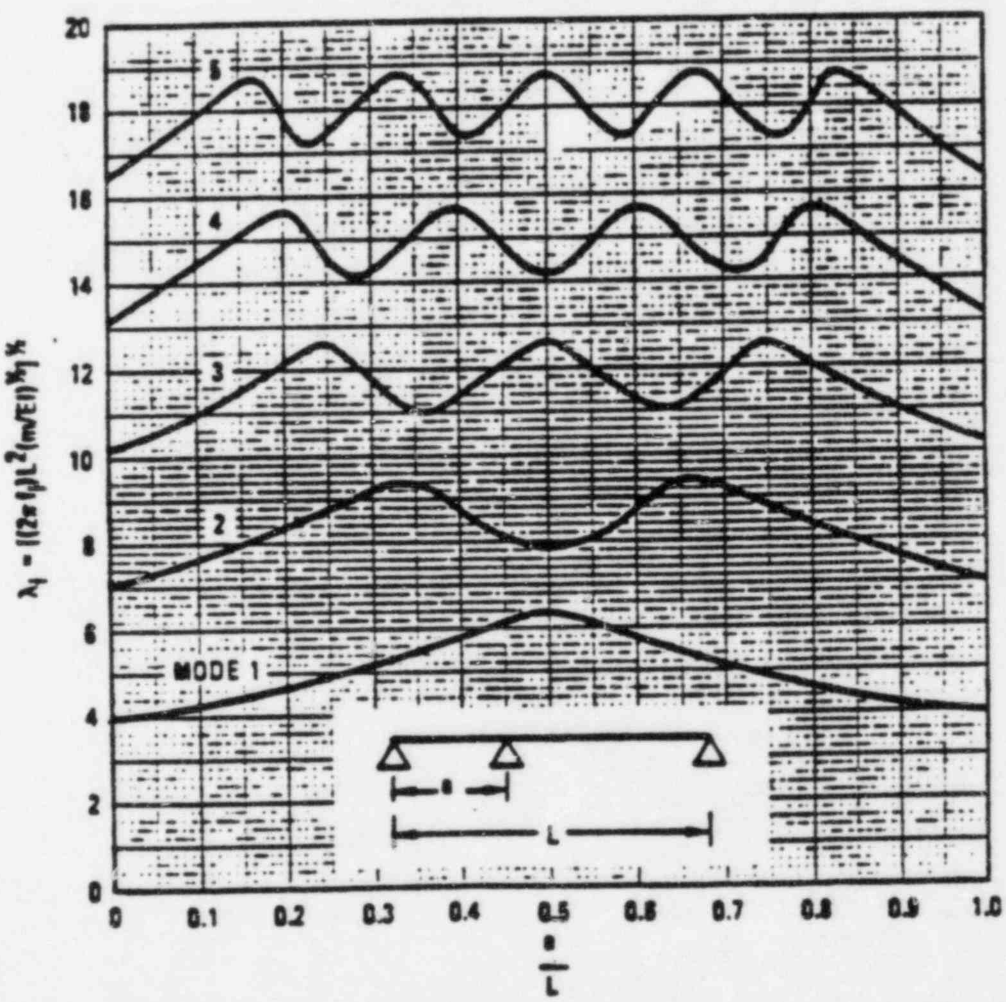


Fig. 8-6(c). Natural frequency parameters of a pinned-pinned-pinned two-span beam (Eqs. 8-12, 8-16). (Ref. 8-3)

Reference: Blevins, R. D., Formulas for Natural Frequency and Mode Shape, Van Nostrand Reinhold; New York, 1979.

where

$$\eta_1 = \frac{\sinh \lambda_i\mu - \sin \lambda_i\mu}{\cos \lambda_i\mu - \cosh \lambda_i\mu},$$

$$\eta_2 = \frac{(1 - \cos \lambda_i\mu \cosh \lambda_i\mu)(\cos \lambda_i\eta + \cosh \lambda_i\eta)}{(\cosh \lambda_i\mu - \cos \lambda_i\mu)(1 + \cos \lambda_i\eta \cosh \lambda_i\eta)},$$

$$\eta_3 = \frac{-\sin \lambda_i\eta - \sinh \lambda_i\eta}{\cos \lambda_i\eta + \cosh \lambda_i\eta}.$$

Clamped-pinned-pinned [Fig. 8-5(e)]

$$\tilde{y}_a(\xi) = \sin \lambda_i\xi - \sinh \lambda_i\xi + \eta_1(\cos \lambda_i\xi - \cosh \lambda_i\xi),$$

$$\tilde{y}_\beta(\xi) = \eta_2(\sin \lambda_i\xi + \eta_3 \sinh \lambda_i\xi), \tag{8-18}$$

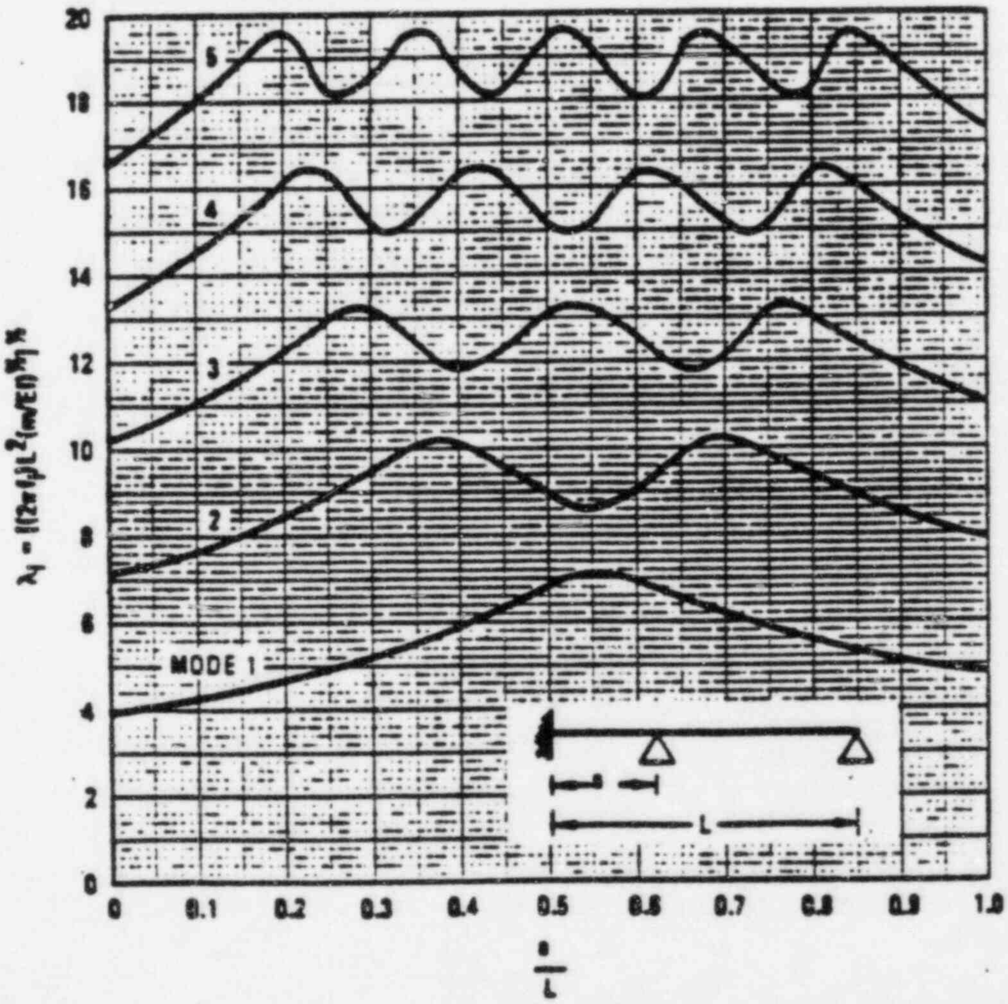


Fig. 8-6(e). Natural frequency parameters of a clamped-pinned-pinned two-span beam (Eqs. 8-13, 8-18). (Ref. 8-6)

Reference: Blevins, R. D., Formulas for Natural Frequency and Mode Shape, Van Nostrand Reinhold; New York, 1979.

where

$$\eta_1 = \frac{\sinh \lambda_i \mu - \sin \lambda_i \mu}{\cos \lambda_i \mu - \cosh \lambda_i \mu},$$

$$\eta_2 = \frac{-2(1 - \cos \lambda_i \mu \cosh \lambda_i \mu)}{(\cos \lambda_i \mu - \cosh \lambda_i \mu)(\cos \lambda_i \eta \sinh \lambda_i \eta - \sin \lambda_i \eta \cosh \lambda_i \eta)},$$

$$\eta_3 = \frac{-\sin \lambda_i \eta}{\sinh \lambda_i \eta}.$$

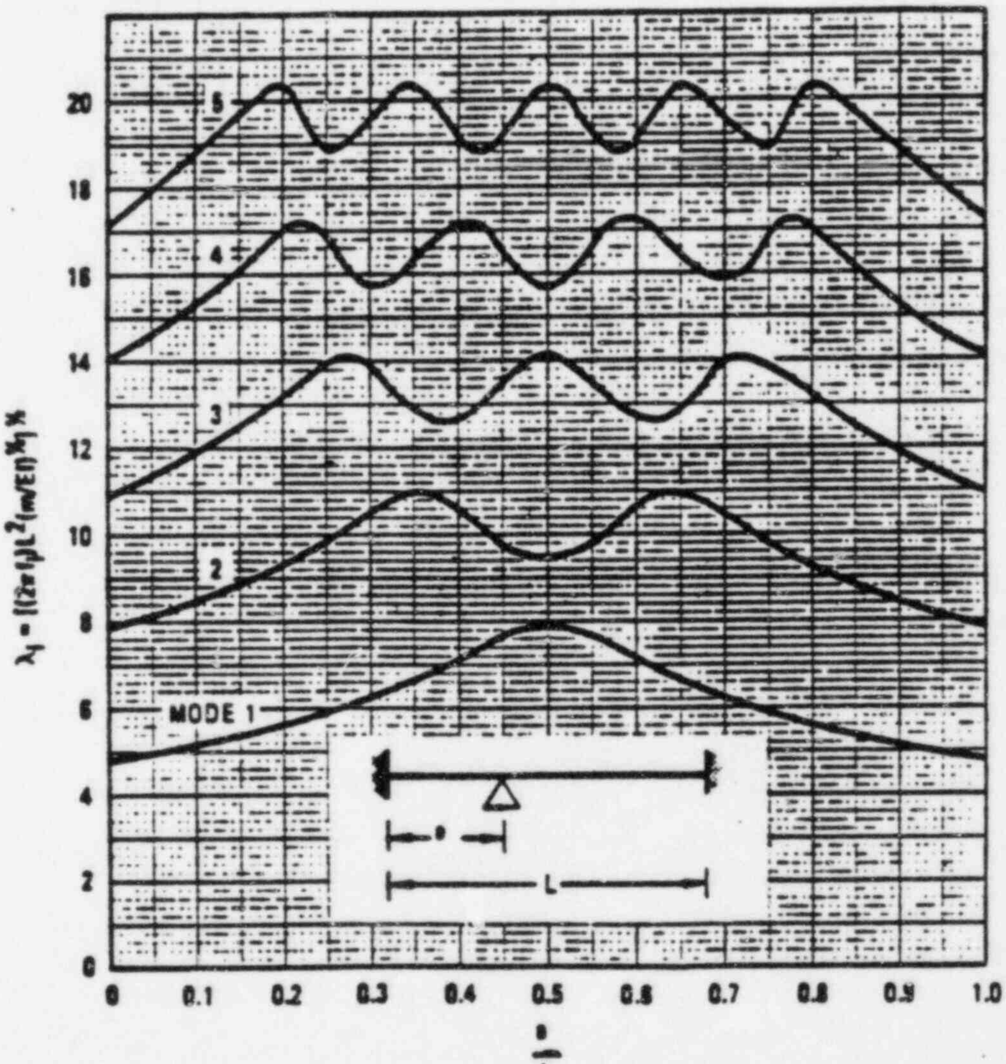
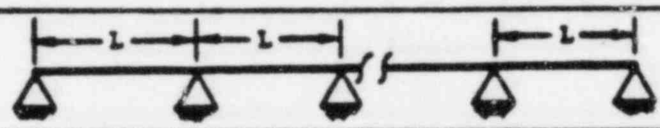


Fig. 8-8(f). Natural frequency parameters of a clamped-pinned-clamped two-span beam (Eqs. 8-13, 8-16). (Ref. 8-8)

Reference: Blevins, R. D., Formulas for Natural Frequency and Mode Shape, Van Nostrand Reinhold; New York, 1979.

Table 8-8d. Pinned-Pinned Multispan Beam with Pinned Intermediate Supports.



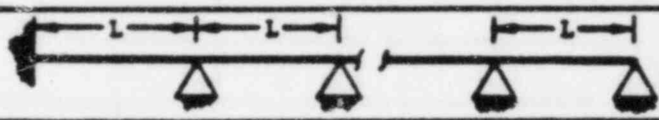
$$f_i = \frac{\lambda_i^2}{2\pi L^2}\left(\frac{EI}{m}\right)^{1/2}$$

Number of Spans (a)	$\lambda_i = \lambda_i$ (Number of Spans)					
	Mode Number (i)					
	1	2	3	4	5	6
1	3.142	6.283	9.425	12.57	15.71	18.85
2	3.142	3.927	6.283	7.068	9.424	10.21
3	3.142	3.557	4.297	4.713	6.707	7.430
4	3.142	3.393	3.928	4.463	6.283	6.545
5	3.142	3.310	3.700	4.152	4.550	6.284
6	3.142	3.260	3.557	3.927	4.293	4.602
7	3.142	3.230	3.460	3.764	4.089	4.394
8	3.142	3.210	3.394	3.645	3.926	4.208
9	3.142	3.196	3.344	3.557	3.800	4.053
10	3.142	3.186	3.309	3.488	3.700	3.927
11	3.142	3.178	3.282	3.436	3.621	3.823
12	3.142	3.173	3.261	3.393	3.557	3.738
13	3.142	3.168	3.244	3.359	3.504	3.666
14	3.141	3.164	3.230	3.332	3.460	3.607
15	3.141	3.161	3.219	3.309	3.424	3.557

(a) Span = a beam segment of axial length L.

Reference: Blevins, R. D., Formulas for Natural Frequency and Mode Shape, Van Nostrand Reinhold; New York, 1979.

Table 8-2a. Clamped-Pinned Multispan Beam with Pinned Intermediate Supports.



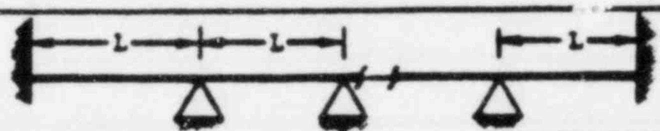
$$f_i = \frac{\lambda_i^2}{2\pi L^2}\left(\frac{EI}{m}\right)^{1/2}$$

Number of Spans[(a)]	$\lambda_i = \lambda_i$ (Number of Spans)					
	Mode Number (i)					
	1	2	3	4	5	6
1	3.927	7.069	10.21	13.35	16.49	19.63
2	3.393	4.463	6.545	7.591	9.687	10.73
3	3.261	3.927	4.600	6.410	7.070	7.727
4	3.210	3.645	4.207	4.655	6.357	6.795
5	3.186	3.488	3.926	4.366	4.682	6.332
6	3.173	3.393	3.738	4.115	4.463	4.697
7	3.164	3.331	3.607	3.927	4.247	4.527
8	3.159	3.290	3.514	3.784	4.069	4.341
9	3.156	3.260	3.444	3.675	3.927	4.178
10	3.153	3.239	3.393	3.592	3.813	4.041
11	3.151	3.222	3.354	3.525	3.721	3.927
12	3.149	3.210	3.322	3.472	3.645	3.832
13	3.148	3.200	3.297	3.428	3.583	3.751
14	3.147	3.192	3.277	3.393	3.531	3.684
15	3.147	3.186	3.261	3.364	3.489	3.627

[(a)] Span = a beam segment of axial length L.

Reference: Blevins, R. D., Formulas for Natural Frequency and Mode Shape, Van Nostrand Reinhold; New York, 1979.

Table 8-3[illegible]. Clamped-Clamped Multispan Beam with Pinned Intermediate Supports.



$$f_i = \frac{\lambda_i^2}{2\pi L^2}\left(\frac{EI}{m}\right)^{1/2}$$

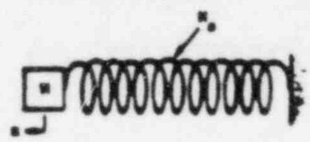
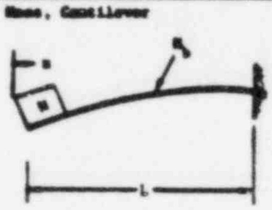
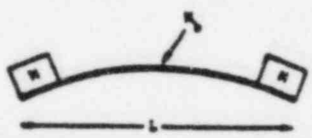
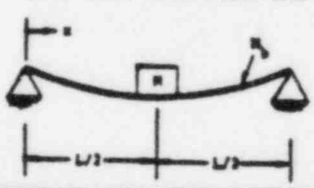
Number of Spans[a]	$\lambda_i = \lambda_i$ (Number of Spans)					
	Mode Number (i)					
	1	2	3	4	5	6
1	4.730	7.853	11.00	14.14	17.28	20.42
2	3.927	4.730	7.068	7.853	10.21	11.00
3	3.557	4.297	4.730	6.707	7.430	7.853
4	3.393	3.928	4.463	4.730	6.545	7.068
5	3.310	3.700	4.152	4.550	4.730	6.460
6	3.260	3.557	3.927	4.298	4.602	4.730
7	3.230	3.460	3.764	4.089	4.394	4.634
8	3.210	3.394	3.645	3.926	4.208	4.464
9	3.196	3.344	3.557	3.800	4.053	4.298
10	3.186	3.309	3.488	3.700	3.927	4.153
11	3.178	3.282	3.435	3.621	3.823	4.030
12	3.173	3.261	3.393	3.557	3.738	3.927
13	3.168	3.244	3.359	3.504	3.666	3.839
14	3.164	3.230	3.332	3.460	3.607	3.764
15	3.161	3.219	3.309	3.424	3.557	3.701

[a] Span = a beam segment of axial length L.

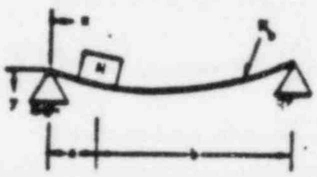
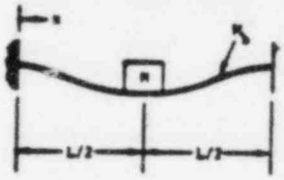
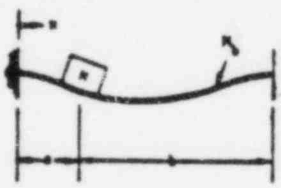
Reference: Blevins, R. D., Formulas for Natural Frequency and Mode Shape, Van Nostrand Reinhold; New York, 1979.

Table 8-8. Slender Beams with Concentrated Masses.

Notation: x = distance along beam; y = distance perpendicular to beam axis; $\tilde{y}$ = mode shape associated with transverse deformation; M = mass; M_b = mass of beam; E = modulus of elasticity; I = area moment of inertia of beam about neutral axis (Table 8-1); L = span of beam; see Table 3-1 for consistent sets of units.

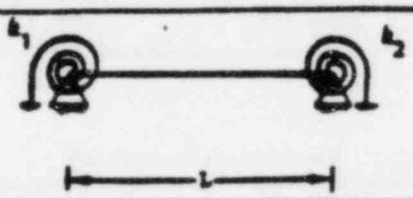
Description	Fundamental Natural Frequency, f_1 (hertz)	Mode Shape, $\tilde{y}(x)$
1. Mass, Helical Spring 	$\frac{1}{2\pi}\left(\frac{k}{M + 0.33\,M_s}\right)^{1/2}$	k = spring constant (Table 4-1) M_s = mass of spring See Frame 4 of Table 8-17 for exact solution.
2. Mass, Cantilever 	$\frac{1}{2\pi}\left[\frac{3EI}{L^3(M + 0.24\,M_b)}\right]^{1/2}$	$\left(\frac{x}{L}\right)^3 - 3\left(\frac{x}{L}\right) + 2$ See Refs. 8-13, 8-14, 8-15 for higher modes.
3. End Masses, Free-Free Beam 	$\frac{2}{\pi}\left\{\frac{EI}{L^3 M_b}\left[1 + \frac{5.45}{1 - 77.4\,(M/M_b)^2}\right]\right\}^{1/2}$	— See Ref. 8-14 for higher modes.
4. Center Mass, Pinned-Pinned Beam 	$\frac{2}{\pi}\left[\frac{3EI}{L^3\,(M + 0.49\,M_b)}\right]^{1/2}$	$3\frac{x}{L} - 4\left(\frac{x}{L}\right)^3$ See Ref. 8-15 for second mode.

Reference: Blevins, R. D., Formulas for Natural Frequency and Mode Shape, Van Nostrand Reinhold; New York, 1979.

5. Off-Center Mass, Pinned-Pinned Beam 	$\frac{1}{2\pi}\left\{\frac{3EI\,(a+b)}{a^2 b^2\,(M+(\alpha+\beta)\,M_b)}\right\}^{1/2}$ $\alpha = \frac{a}{a+b}\left[\frac{(2b+a)^2}{12b^2} + \frac{a^2}{28b^2} - \frac{a\,(2b+a)}{10b^2}\right]$ $\beta = \frac{b}{a+b}\left[\frac{(2a+b)^2}{12a^2} + \frac{b^2}{28a^2} - \frac{b\,(2a+b)}{10a^2}\right]$	$\left[3\left(1-\frac{x}{L}\right) - \frac{b^2}{L^2} - \left(1-\frac{x}{L}\right)^2\right]\left(\frac{x}{L}\right)$; $0 \le x \le a$ $\left[\frac{3x}{L} - \frac{a^2}{L^2} - \left(1-\frac{x}{L}\right)^2\right]\left(1-\frac{x}{L}\right)$; $a < x \le L$ $L = a + b$
6. Center Mass, Clamped-Clamped Beam 	$\frac{4}{\pi}\left[\frac{3EI}{L^3\,(M+0.37\,M_b)}\right]^{1/2}$	$3\left(\frac{x}{L}\right)^2 - 4\left(\frac{x}{L}\right)^3$; $0 \le \frac{x}{L} \le \frac{1}{2}$ See Ref. 8-16 for second mode.
7. Off-Center Mass, Clamped-Clamped Beam 	$\frac{4}{\pi}\left(\frac{3EI}{L^3\,(M+(\alpha+\beta)\,M_b)}\right)^{1/2}$ $\alpha = \frac{a}{a+b}\left[\frac{(3a+b)^2}{28a^2} + \frac{9\,(a+b)^2}{28a^2} - \frac{(a+b)(3a+b)}{4a^2}\right]$ $\beta = \frac{b}{a+b}\left[\frac{(3b+a)^2}{28b^2} + \frac{9\,(a+b)^2}{28b^2} - \frac{(a+b)(3b+a)}{4b^2}\right]$	$\left(\frac{x}{L}\right)^2\left(\frac{3a}{L}\frac{b}{L} + \frac{b}{L}\frac{x}{L} - \frac{3x}{L}\right)$; $0 \le x \le a$ $\left(1-\frac{x}{L}\right)^2\left[\frac{3b+a}{L}\left(1-\frac{x}{L}\right) - \frac{2b}{L}\right]$; $a < x \le b$

Reference: Blevins, R. D., Formulas for Natural Frequency and Mode Shape, Van Nostrand Reinhold; New York, 1979.

Table 8-8. Natural Frequencies of a Pinned-Pinned Beam with Unequal Torsion Springs at the Pinned Joints.[a]



Natural Frequency (hertz),

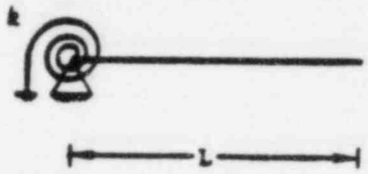
$$f_i = \frac{\lambda_i^2}{2\pi L^2}\left(\frac{EI}{m}\right)^{1/2}$$

		$\lambda_i = \lambda_i\left(\frac{k_1 L}{EI}, \frac{k_2 L}{EI}\right)$				
$\frac{k_1 L}{EI}$	$\frac{k_2 L}{EI}$	i = 1	i = 2	i = 3	i = 4	i = 5
0.0	0.0	3.142	6.283	9.425	12.566	15.708
0.01	0	3.143	6.284	9.425	12.567	15.708
0.01	0.01	3.142	6.283	9.425	12.566	15.701
0.01	0.1	3.127	6.276	9.420	12.563	15.705
0.01	1.0	2.941	6.197	9.369	12.525	15.675
0.01	10	4.642	8.460	11.943	15.255	18.495
0.01	100	3.969	7.146	10.325	13.507	16.691
0.01	∞	3.928	7.069	10.211	13.352	16.494
0.1	0	3.157	6.291	9.430	12.570	15.711
0.1	0.01	3.156	6.290	9.430	12.570	15.711
0.1	0.1	3.141	6.283	9.425	12.566	15.708
0.1	1.0	2.957	6.204	9.374	12.529	15.678
0.1	10	4.654	8.466	11.947	15.258	18.498
0.1	100	3.981	7.152	10.330	13.511	16.694
0.1	∞	3.940	7.076	10.215	13.356	16.496
1.0	0	3.273	6.356	9.475	12.605	15.739
1.0	0.01	3.272	6.355	9.474	12.604	15.739
1.0	0.1	3.258	6.348	9.470	12.601	15.736
1.0	1.0	3.084	6.271	9.419	12.563	15.706
1.0	10	4.763	8.523	11.985	15.287	18.522
1.0	100	4.083	7.211	10.371	13.543	16.721
1.0	∞	4.042	7.194	10.257	13.388	16.523
10	0	3.665	6.688	9.752	12.840	15.942
10	0.01	3.663	6.687	9.751	12.839	15.942
10	0.1	3.651	6.680	9.747	12.836	15.939
10	1.0	3.497	6.608	9.698	12.800	15.910
10	10	5.221	8.857	12.245	15.503	18.708
10	100	4.475	7.529	10.638	13.771	16.919
10	∞	4.430	7.450	10.522	13.614	16.720
100	0	3.889	7.003	10.119	13.236	16.354
100	0.01	3.888	7.003	10.118	13.235	16.354
100	0.1	3.876	6.996	10.114	13.232	16.351
100	1.0	3.727	6.927	10.067	13.196	16.322
100	10	5.569	9.260	12.662	15.928	19.136
100	100	4.735	7.866	11.020	14.177	17.339
100	∞	4.685	7.781	10.896	14.015	17.134
∞	∞	4.730	7.853	10.996	14.137	17.279

[a] Ref. 8-25.

Reference: Blevins, R. D., Formulas for Natural Frequency and Mode Shape, Van Nostrand Reinhold; New York, 1979.

Table 8-10. Natural Frequencies of a Pinned-Free Beam with a Torsion Spring at the Pinned Joint.[a]



Natural Frequency (hertz),

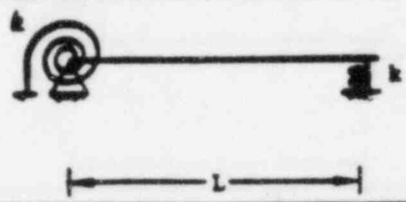
$$f_i = \frac{\lambda_i^2}{2\pi L^2}\left(\frac{EI}{m}\right)^{1/2}$$

$\frac{kL}{EI}$	$\lambda_i = \lambda_i\,(kL/EI)$					
	$i = 1$	$i = 2$	$i = 3$	$i = 4$	$i = 5$	$i = 6$
0	0	3.927	7.069	10.21	13.35	16.49
0.01	0.4159	3.928	7.069	10.21	13.35	16.49
0.1	0.7357	3.938	7.076	10.22	13.36	16.50
1	1.248	4.031	7.134	10.26	13.39	16.52
10	1.723	4.400	7.451	10.52	13.61	16.72
100	1.857	4.650	7.783	10.90	14.01	17.13
∞	1.875	4.694	7.855	11.00	14.14	17.28

[a] Ref. 8-26.

Reference: Blevins, R. D., Formulas for Natural Frequency and Mode Shape, Van Nostrand Reinhold; New York, 1979.

Table 8-11. Fundamental Natural Frequency of a Beam with Torsion and Translational Spring Boundaries.[a]



Fundamental Natural Frequency (hertz),

$$f_1 = \frac{\lambda_1^2}{2\pi L^2}\left(\frac{EI}{m}\right)^{1/2}$$

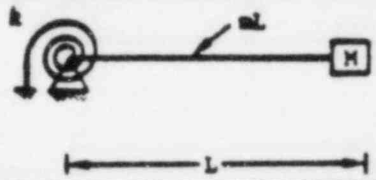
$$\lambda_1 = \lambda_1\left(\frac{kL^3}{EI}, \frac{kL}{EI}\right)$$

kL/EI	kL³/EI = 0	0.01	0.1	1	10	100	1000	∞
0	0	0.4162	0.7397	1.3098	2.2313	2.9886	3.1261	3.1416
0.01	0.4159	0.4948	0.7577	1.3134	2.2326	2.9901	3.1277	3.1432
0.1	0.7358	0.7541	0.8782	1.3437	2.2434	3.0030	3.1415	3.1572
1	1.2479	1.2520	1.2870	1.5358	2.3265	3.1084	3.2566	3.2733
10	1.7227	1.7245	1.7406	1.8793	2.5388	3.4412	3.6423	3.6646
100	1.8568	1.8583	1.8720	1.9939	2.6262	3.6133	3.8614	3.8892
1000	1.8732	1.8748	1.8882	2.0084	2.6376	3.6377	3.8940	3.9227
∞	1.8751	1.8766	1.8900	2.0100	2.6389	3.6405	3.8978	3.9266

[a] Ref. 8-27, copyright Academic Press (London) Ltd., reproduced with permission.

Reference: Blevins, R. D., Formulas for Natural Frequency and Mode Shape, Van Nostrand Reinhold; New York, 1979.

Table 8-12. Fundamental Natural Frequency of a Beam with Torsion Spring and Point Mass.[a]



Fundamental Natural Frequency (hertz),

$$f_1 = \frac{\lambda_1^2}{2\pi L^2}\left(\frac{EI}{m}\right)^{1/2}$$

$$\lambda_1 = \lambda_1\left(\frac{M}{mL}, \frac{kL}{EI}\right)$$

$\frac{kL}{EI}$	$\frac{M}{mL} = 0$	0.01	0.1	1.0	10	100
0	0	0	0	0	0	0
0.01	0.4159	0.4129	0.3895	0.2941	0.1762	0.09983
0.1	0.7378	0.7303	0.6887	0.5194	0.3111	0.1762
1.0	1.2479	1.2381	1.1642	0.8705	0.5194	0.2941
10.0	1.7227	1.7071	1.5912	1.1642	0.6887	0.3895
100.0	1.8568	1.8388	1.7071	1.2381	0.7303	0.4129
∞	1.8751	1.861	1.723	1.247	0.735	0.416

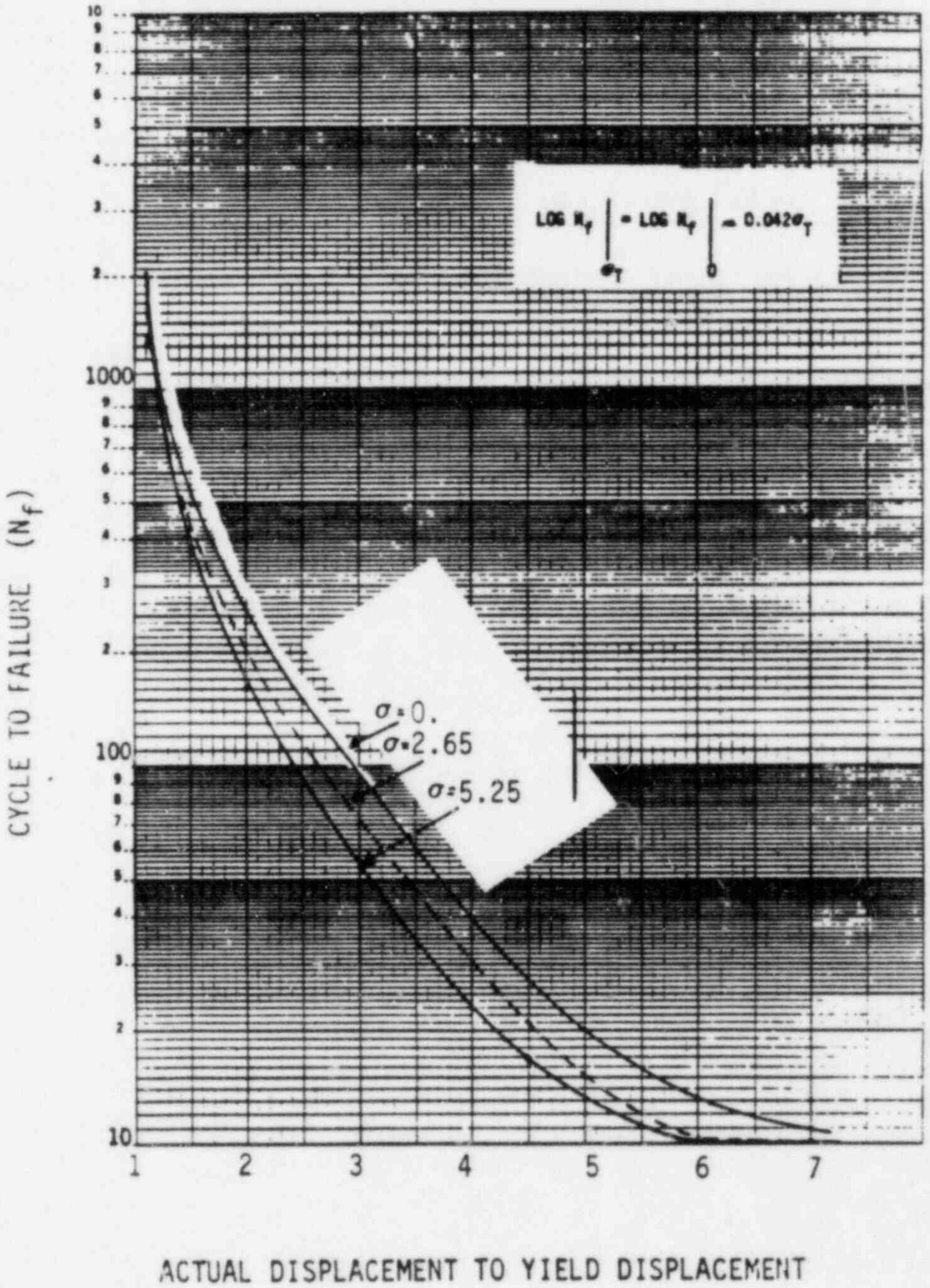
(a) Ref. 8-28.

Reference: Blevins, R. D., Formulas for Natural Frequency and Mode Shape, Van Nostrand Reinhold; New York, 1979.

APPENDIX F

MOMENT ROTATION AND FATIGUE CURVES FOR STRUCTURAL COMPONENTS

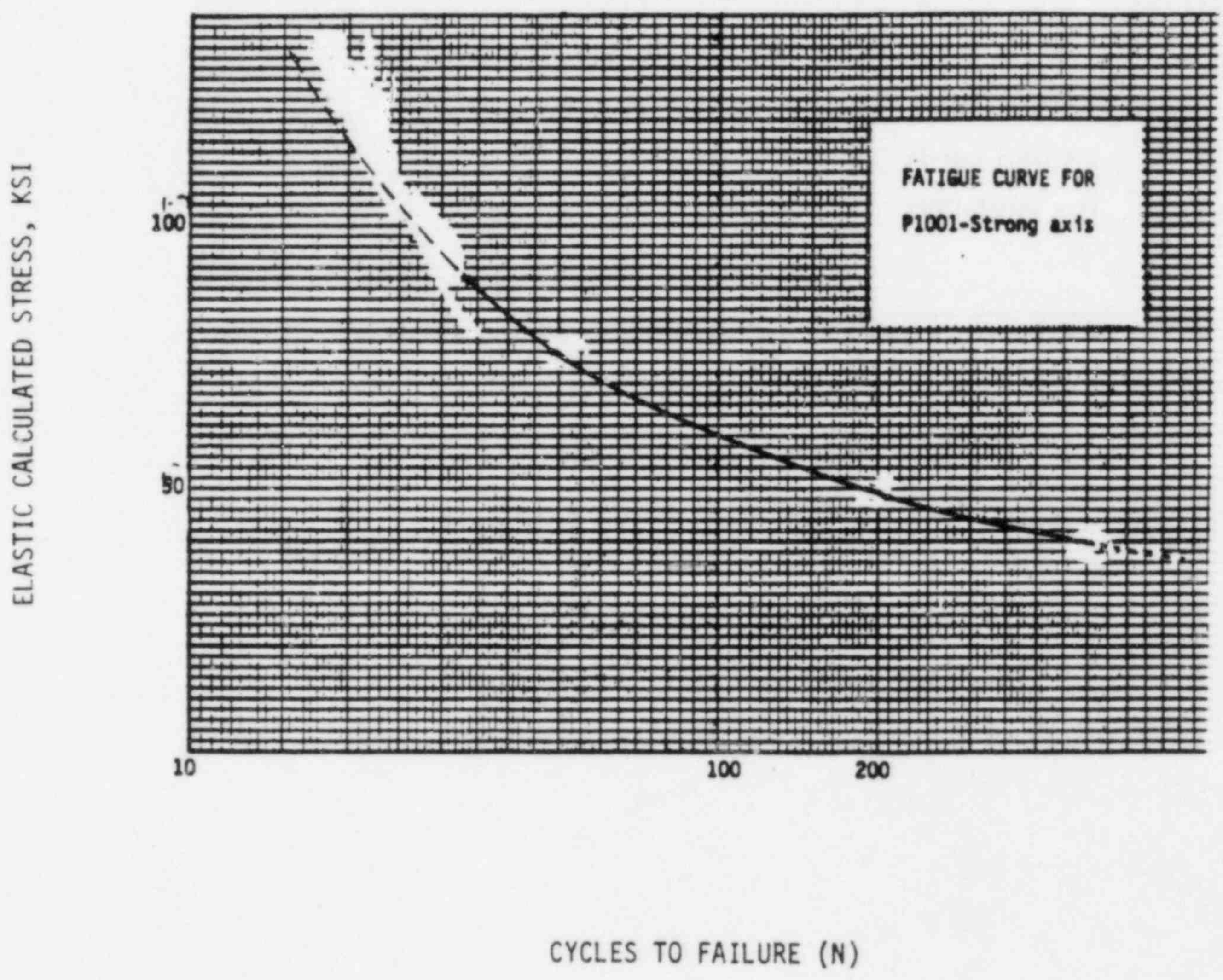
- Threaded Rods
- Unistrut P1001 -- Strong Axis
- Unistrut P1001 -- Weak Axis
- Unistrut P1000 -- Weak Axis
- Unistrut P1331 -- one side only
  - moment-rotation curve
  - fatigue curve
- Unistrut P1331 -- both sides
  - moment-rotation curve
  - fatigue curve
- Unistrut P1331, P1325
  - moment-rotation curve
  - fatigue curve
- Unistrut P1331, P2626
  - moment-rotation curve
  - fatigue curve
- Unistrut P1325 -- both sides
  - moment-rotation curve
  - fatigue curve
- Unistrut P2626 -- both sides, and P1026 -- both sides
  - moment-rotation curve
  - fatigue curve
- Unistrut P1458 -- both sides
  - moment-rotation curve
  - fatigue curve
- Unistrut P1505 -- both sides
  - moment-rotation curve
  - fatigue curve
- Unistrut P2543 -- both sides
  - moment-rotation curve
  - fatigue curve
- Unistrut P2543
  - moment-rotation curve
  - fatigue curve



2

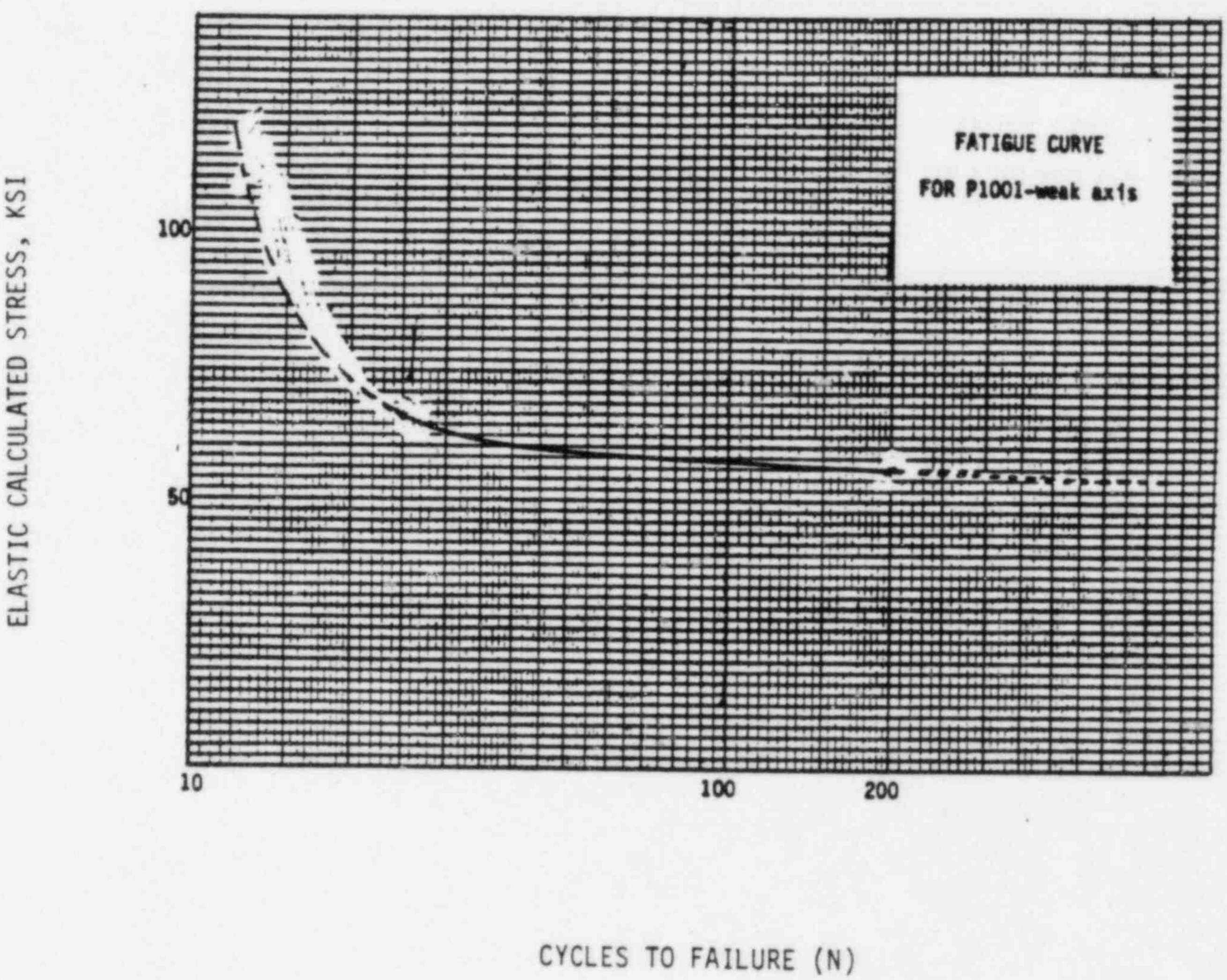
NOTE: Preload = 0, 2.65, and 5.25 ksi

FATIGUE CURVE FOR THREADED ROD



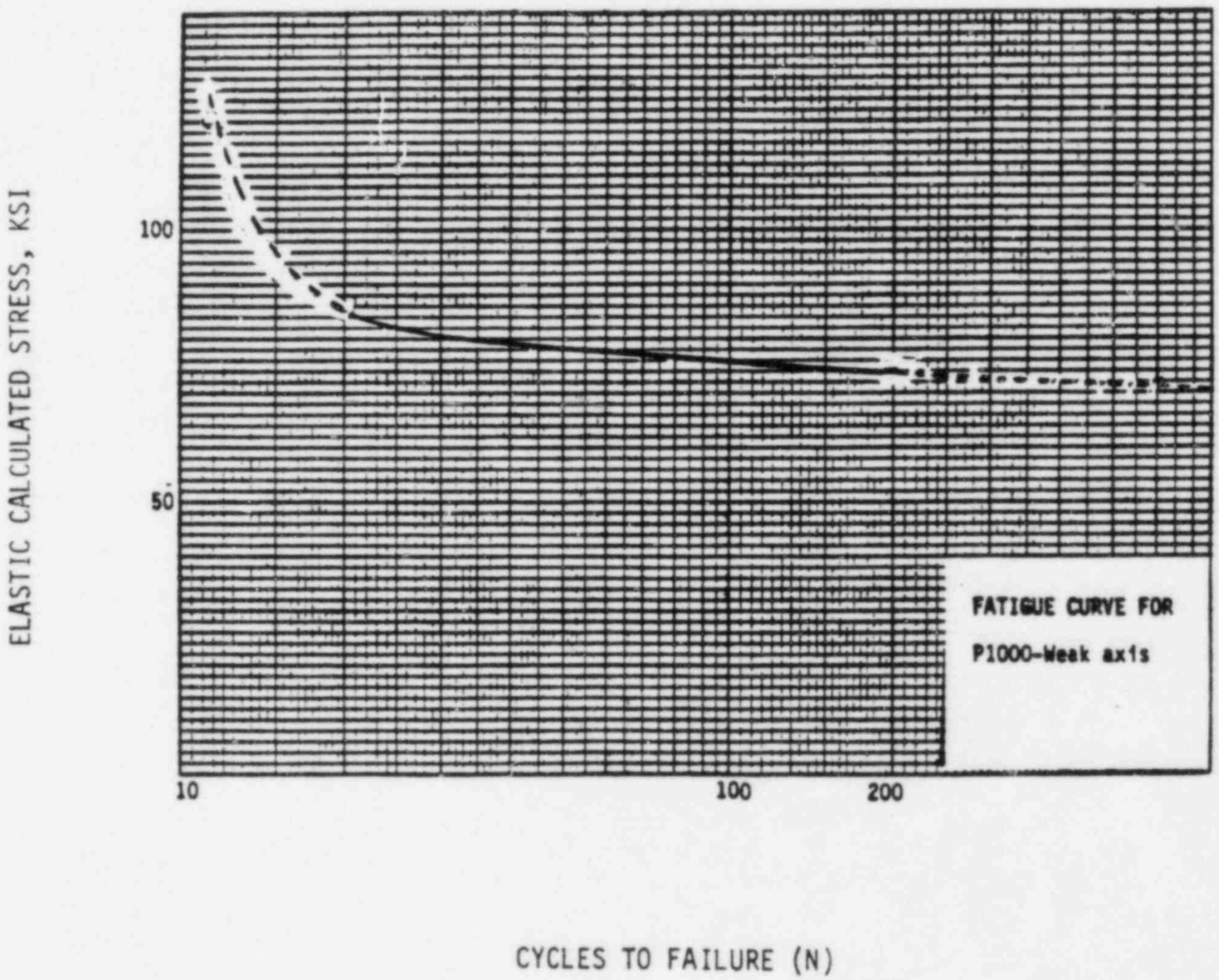
NOTE: No Preload.

MOMENT-ROTATION CURVE FOR UNISTRUT P1001--Strong Axis



NOTE: No Preload.

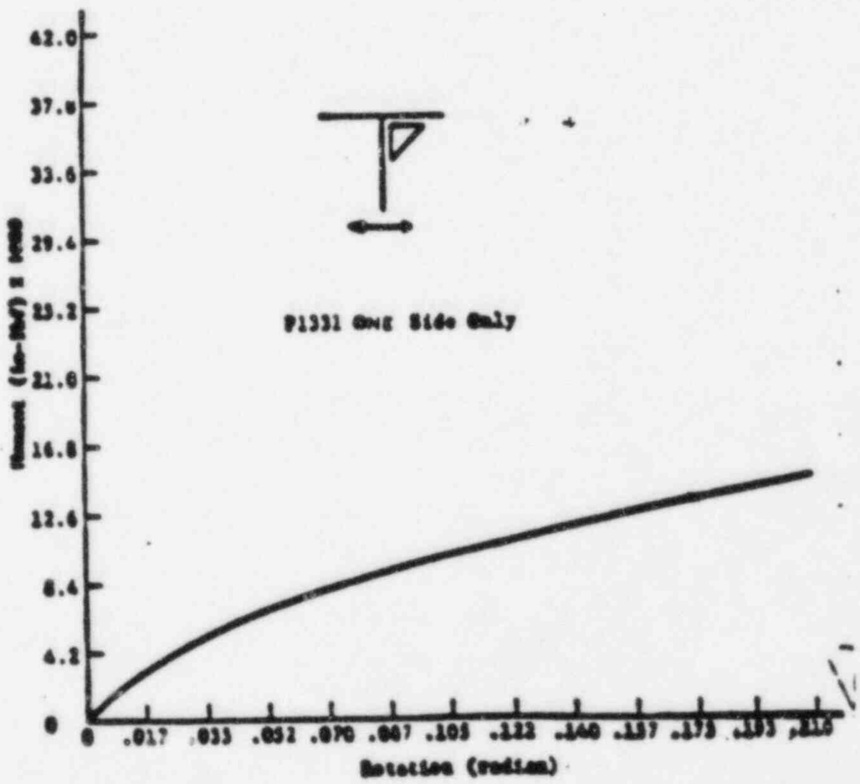
MOMENT-ROTATION CURVE FOR UNISTRUT P1001--Weak Axis



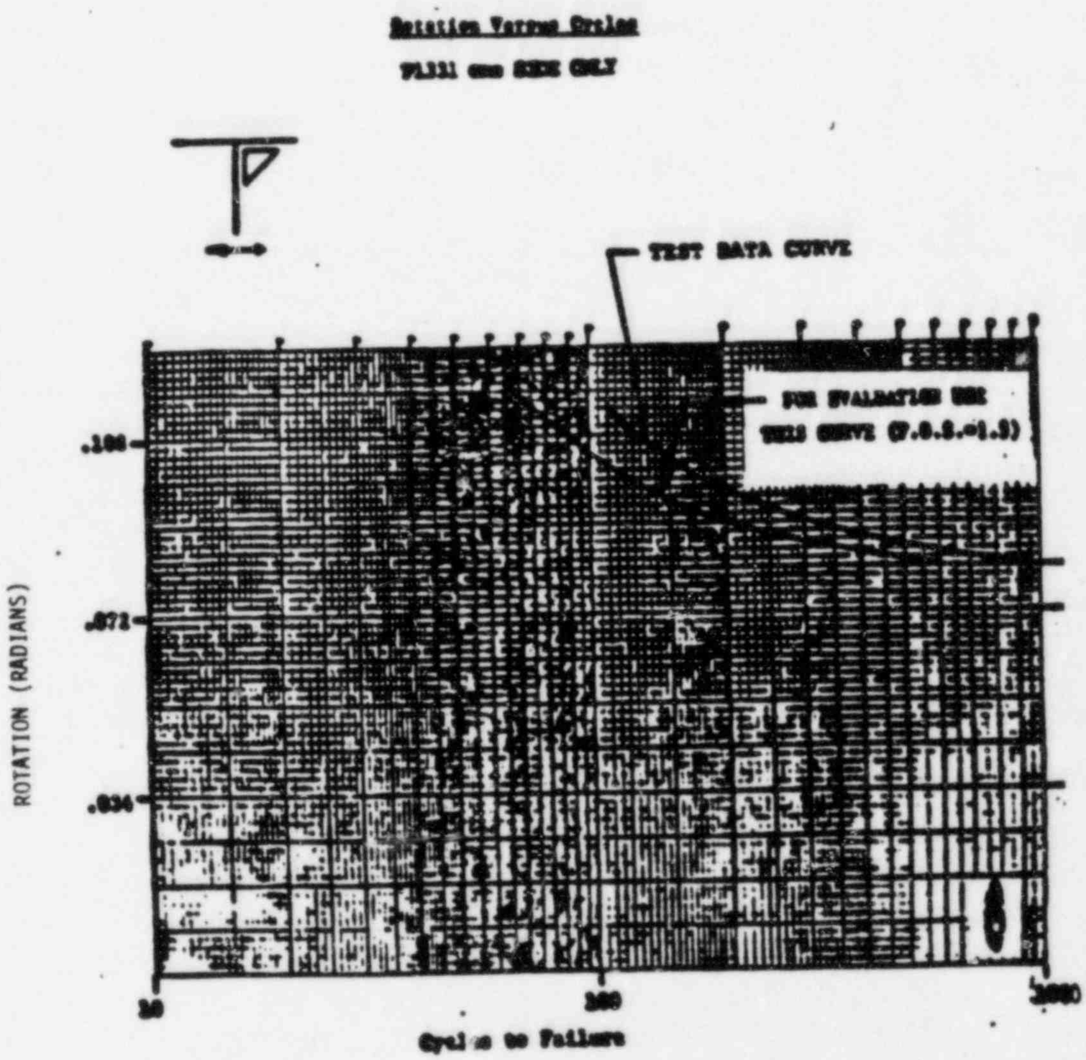
NOTE: No Preload.

FATIGUE CURVE FOR UNISTRUT P1000--Weak Axis

Moment Versus Rotation

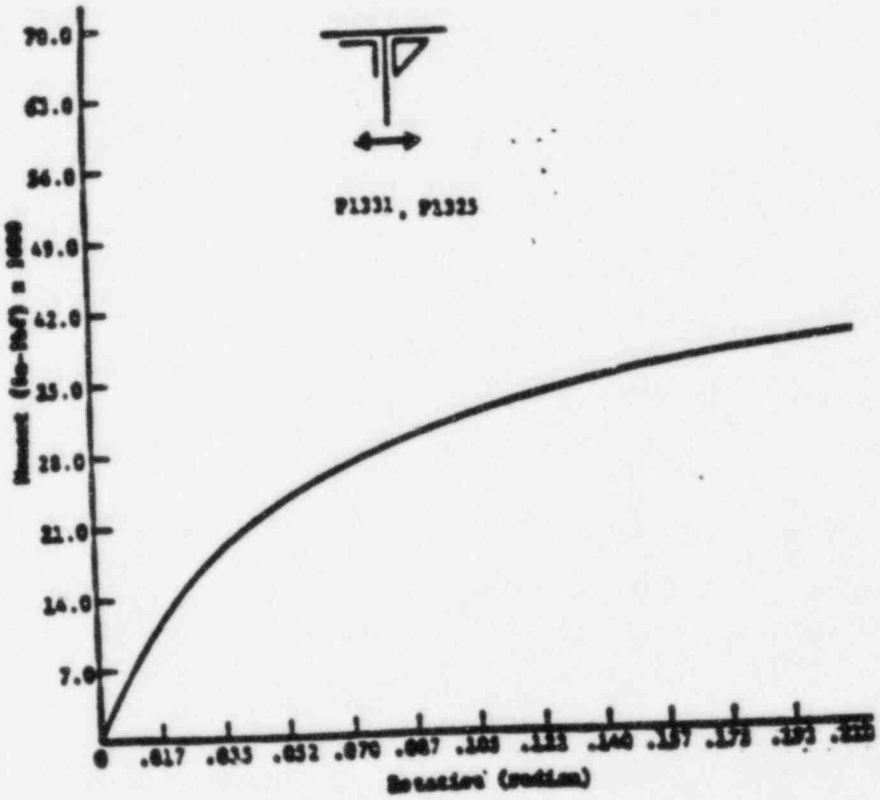


MOMENT-ROTATION CURVE FOR UNISTRUT P1331--One-Side



NOTE: 1,000-pound Preload.

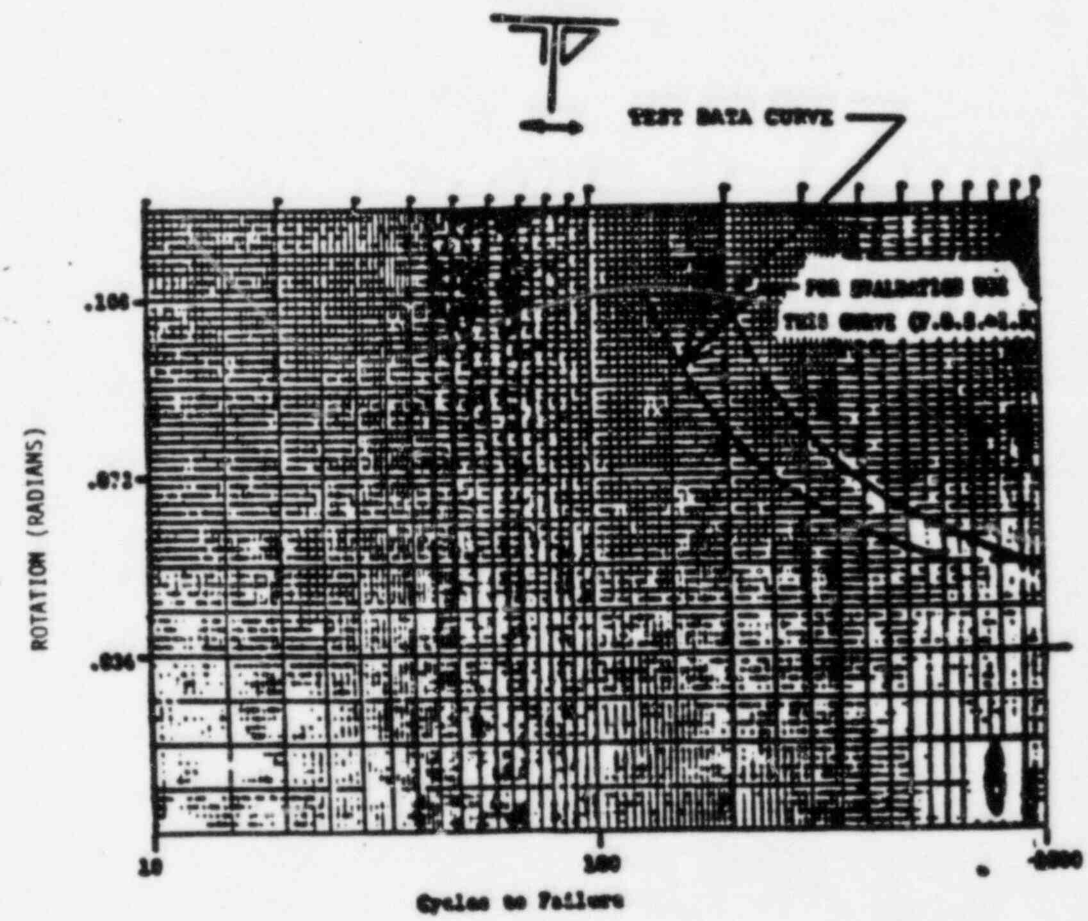
FATIGUE CURVE FOR UNISTRUT P1331--One Side



MOMENT-ROTATION CURVE FOR UNISTRUT P1331, P1325

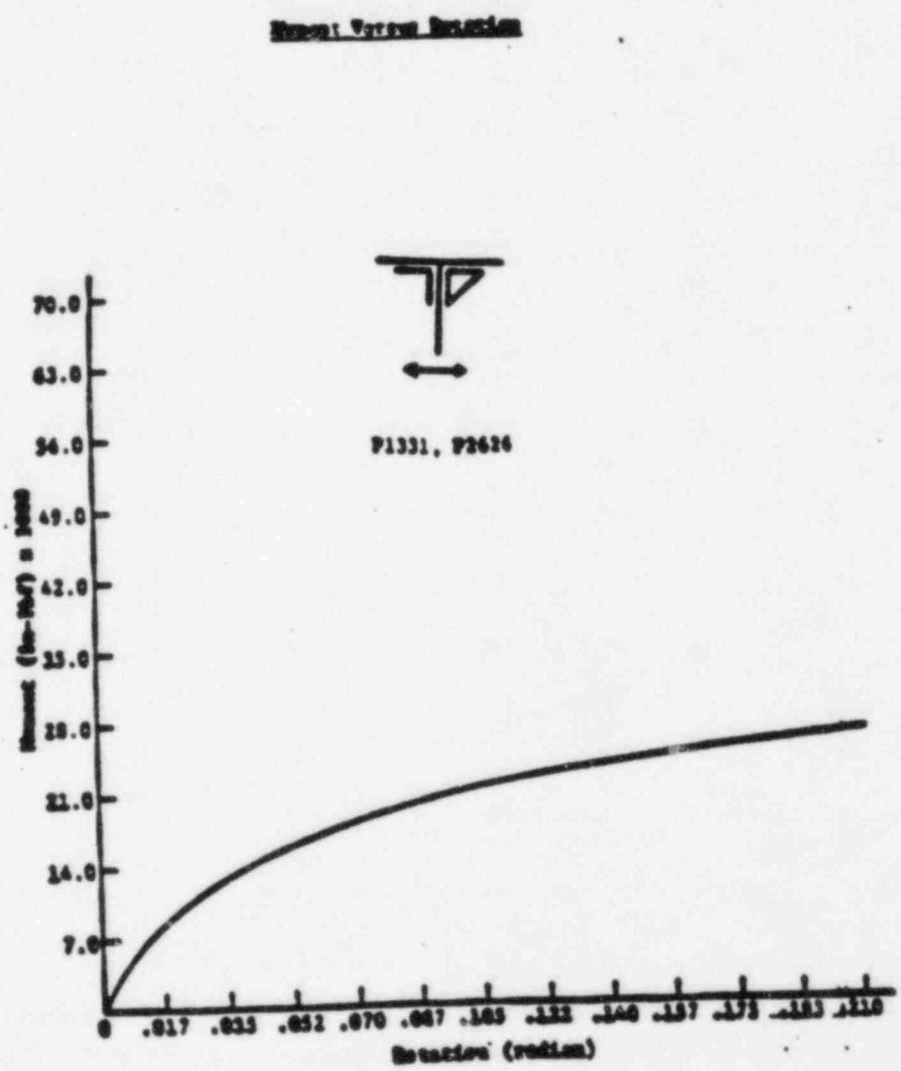
Rotation Versus Cycles

P1325 AND P1331

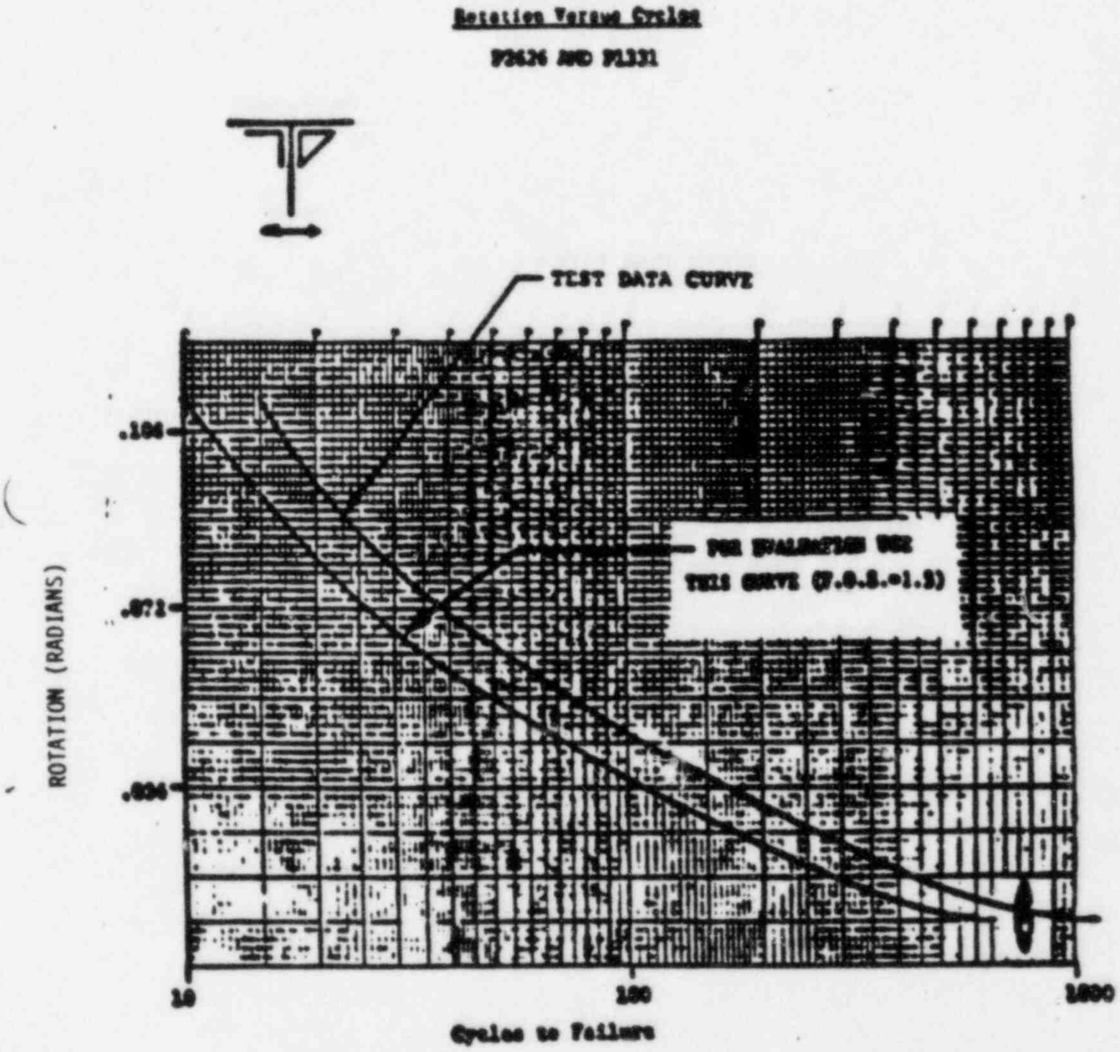


NOTE: 1,000-pound Preload.

FATIGUE CURVE FOR UNISTRUT P1331, P1325



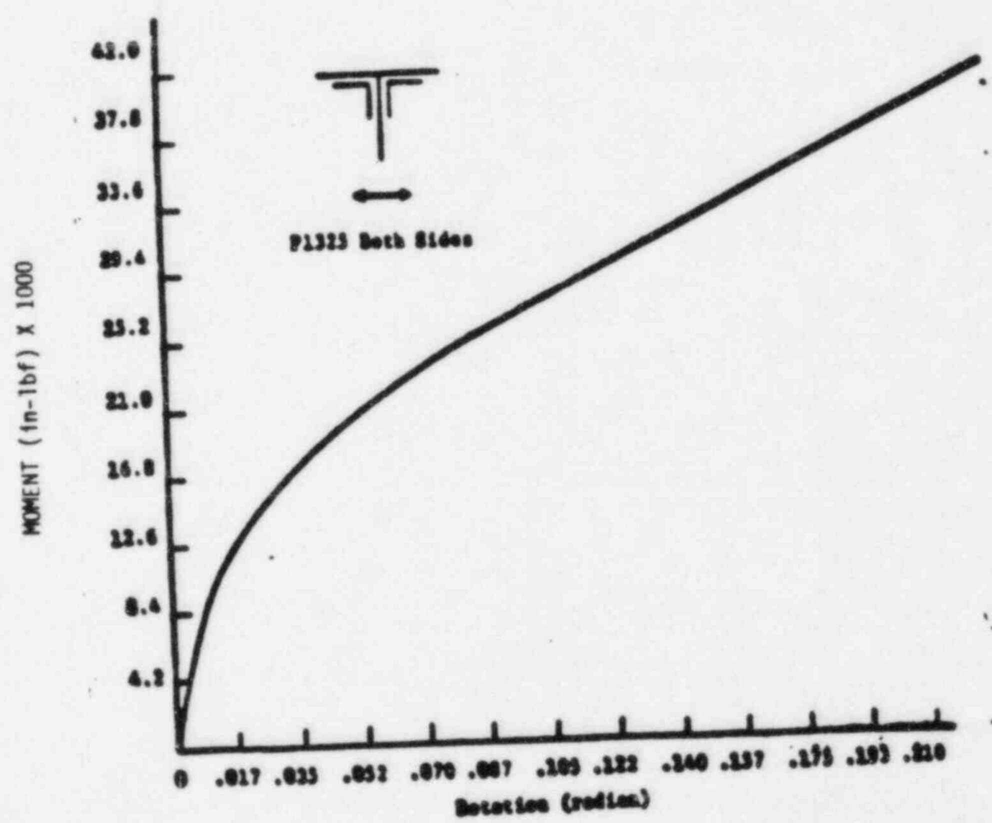
MOMENT-ROTATION CURVE FOR UNISTRUT P1331, P2626



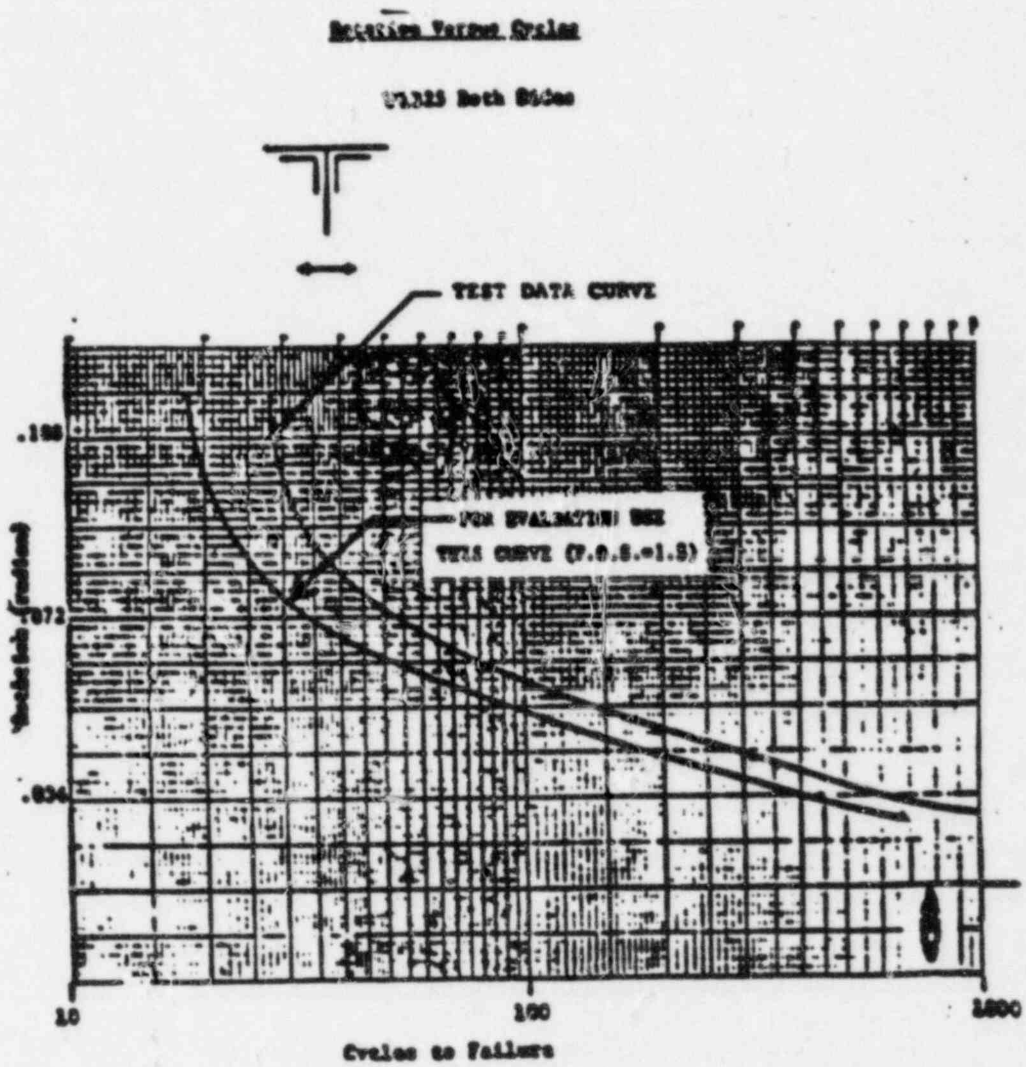
NOTE: 1,000-pound Preload.

FATIGUE CURVE FOR UNISTRUT P1331, P2626

Moment Versus Rotation

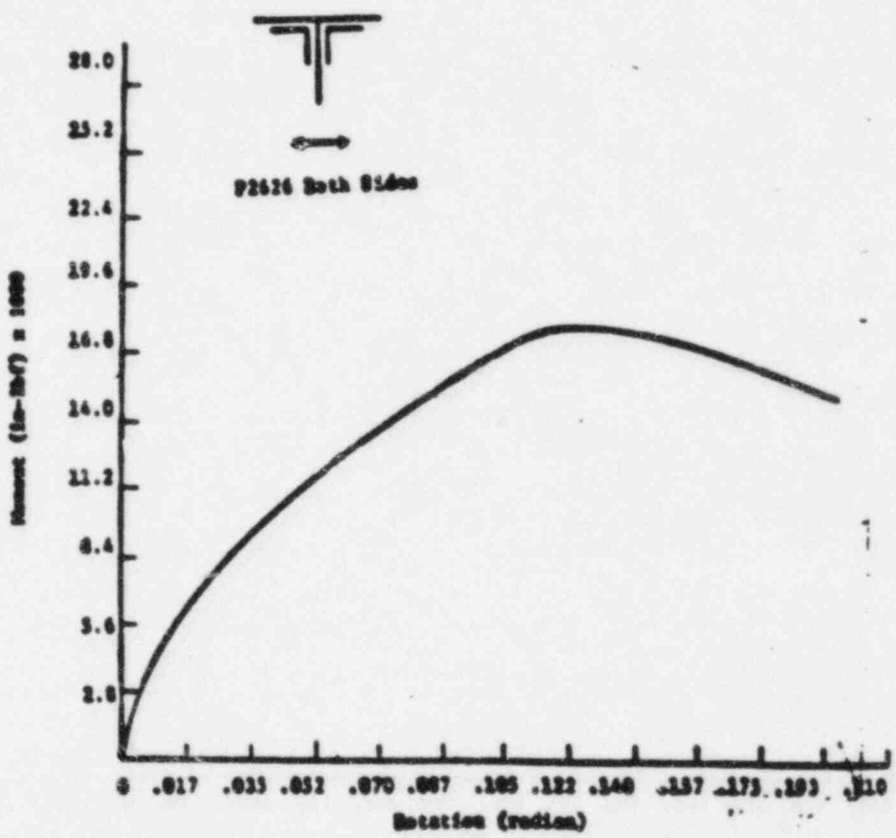


MOMENT-ROTATION CURVE FOR UNISTRUT P1325--Both Sides



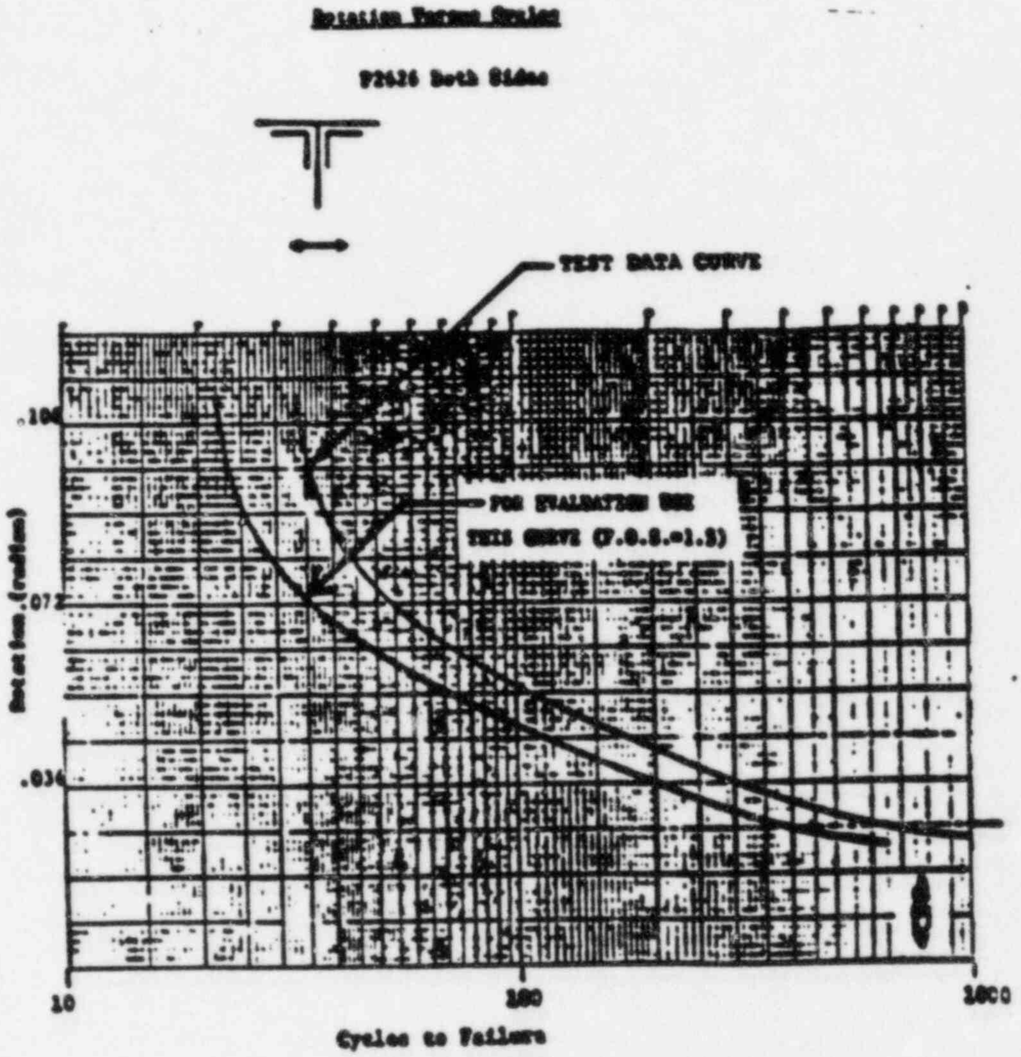
NOTE: 1,000-pound Preload.

FATIGUE CURVE FOR UNISTRUT P1325--Both Sides



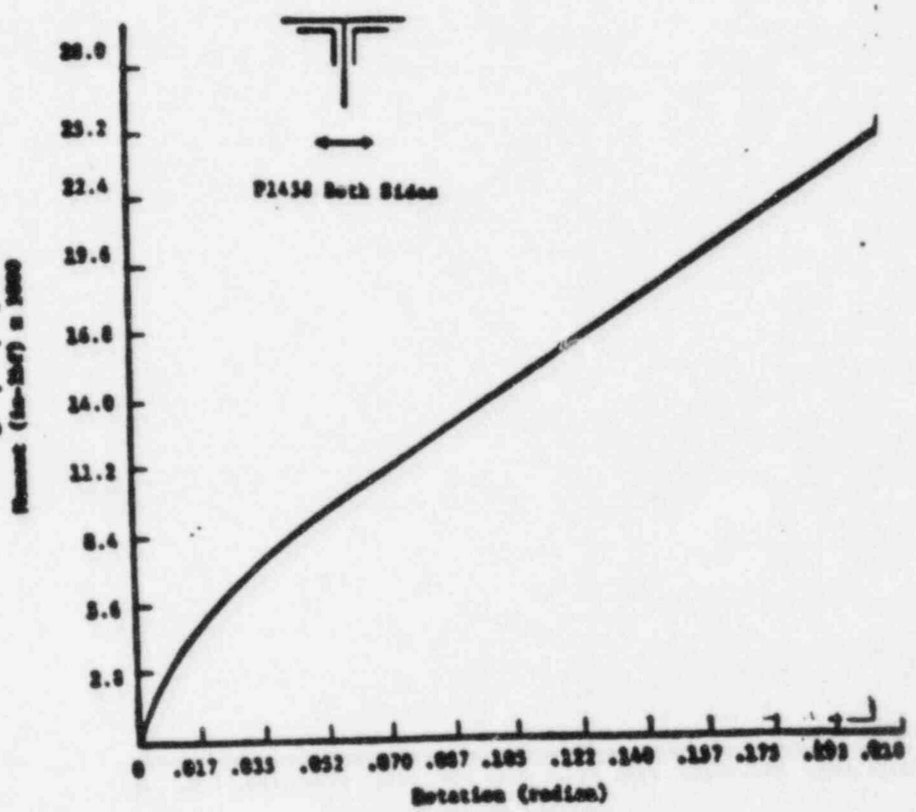
MOMENT-ROTATION CURVE FOR UNISTRUT P2626 OR UNISTRUT P1026

UNISTRUT P2626 Both Sides
or P1026 Both Sides

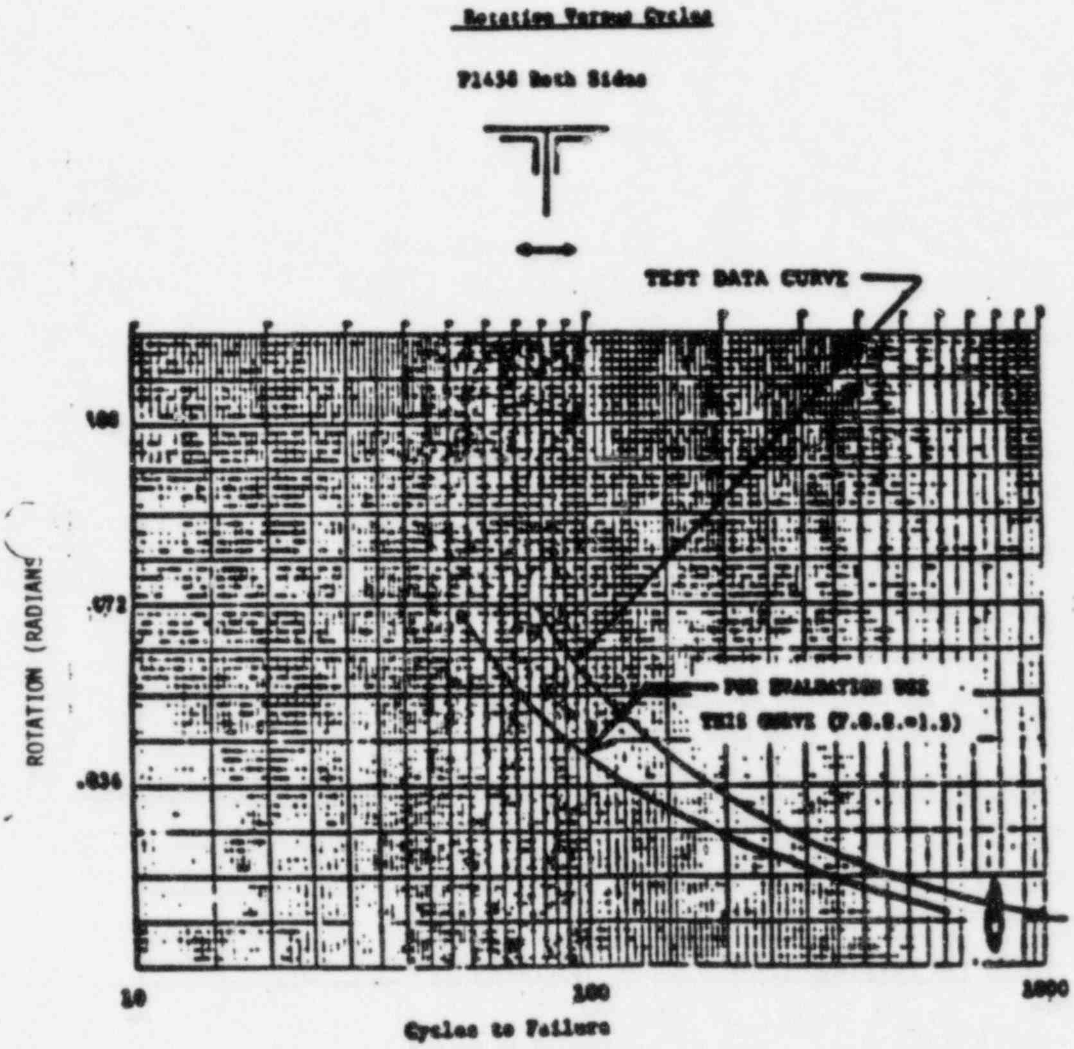


NOTE: 1,000-pound Preload.

FATIGUE CURVE FOR UNISTRUT P2626 AND UNISTRUT P1026

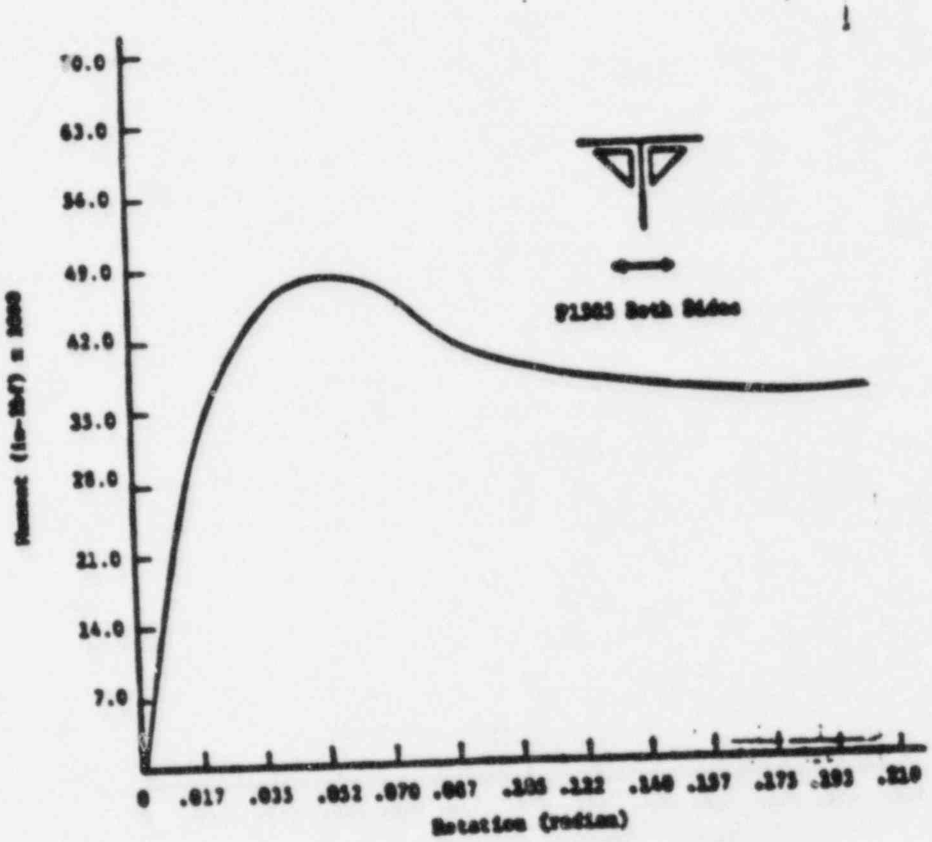


MOMENT-ROTATION CURVE FOR UNISTRUT P1458--Both Sides

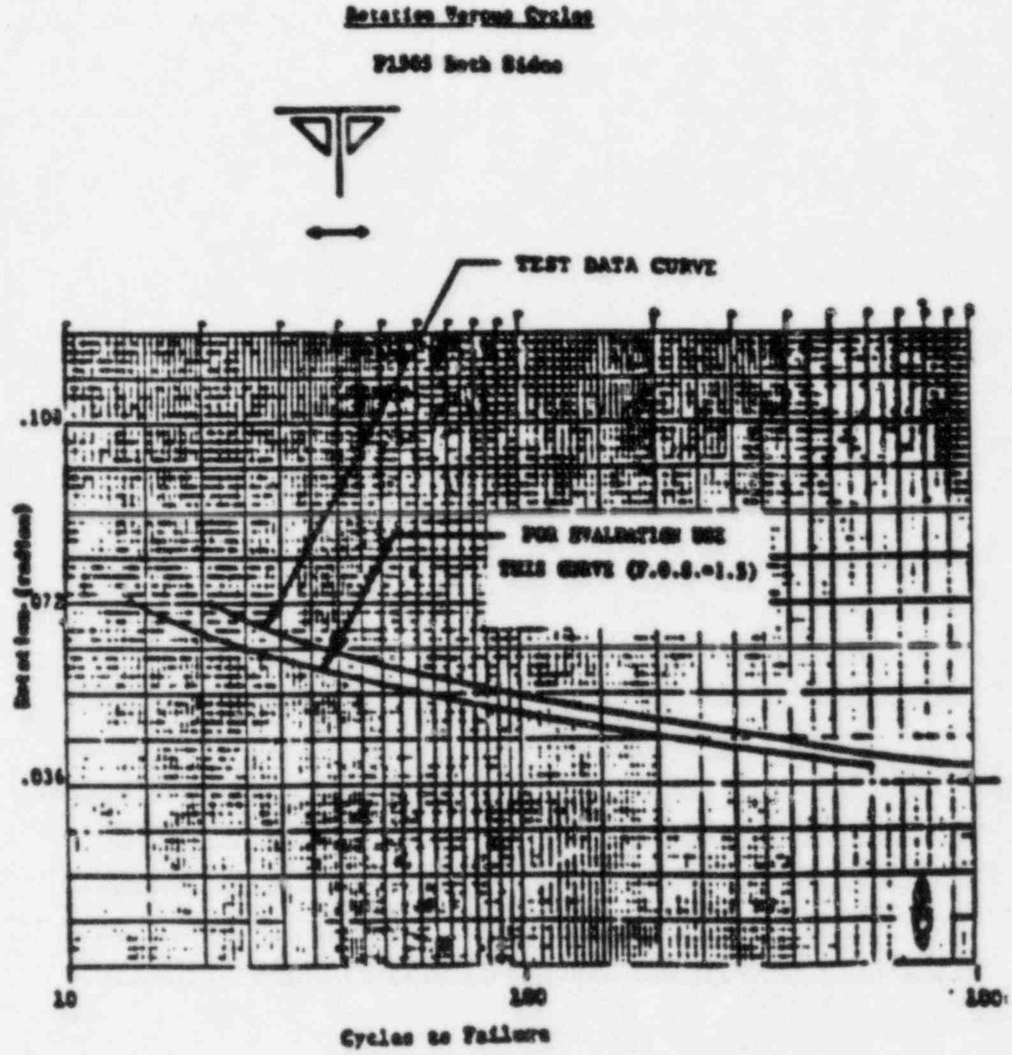


NOTE: 1,000-pound Preload.

FATIGUE CURVE FOR UNISTRUT P1458--Both Sides

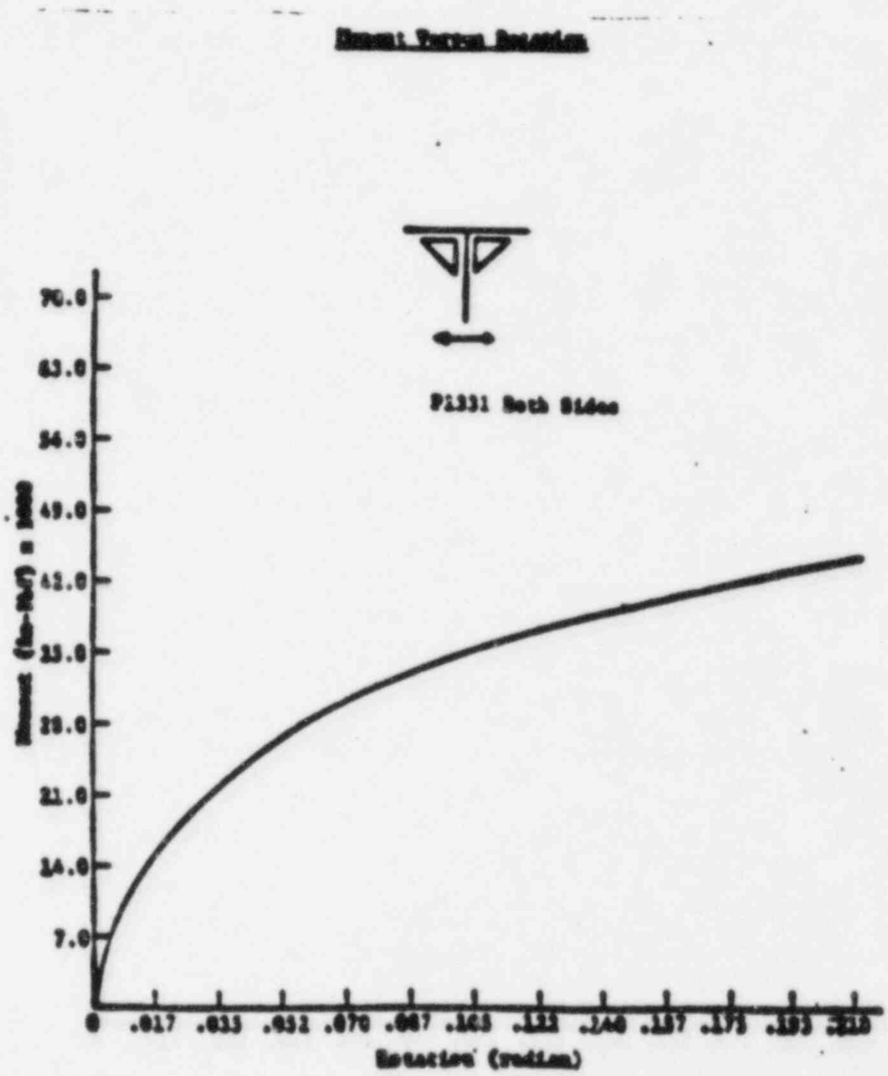


MOMENT-ROTATION CURVE FOR UNISTRUT P1505--Both Sides



NOTE: 1,000-pound Preload.

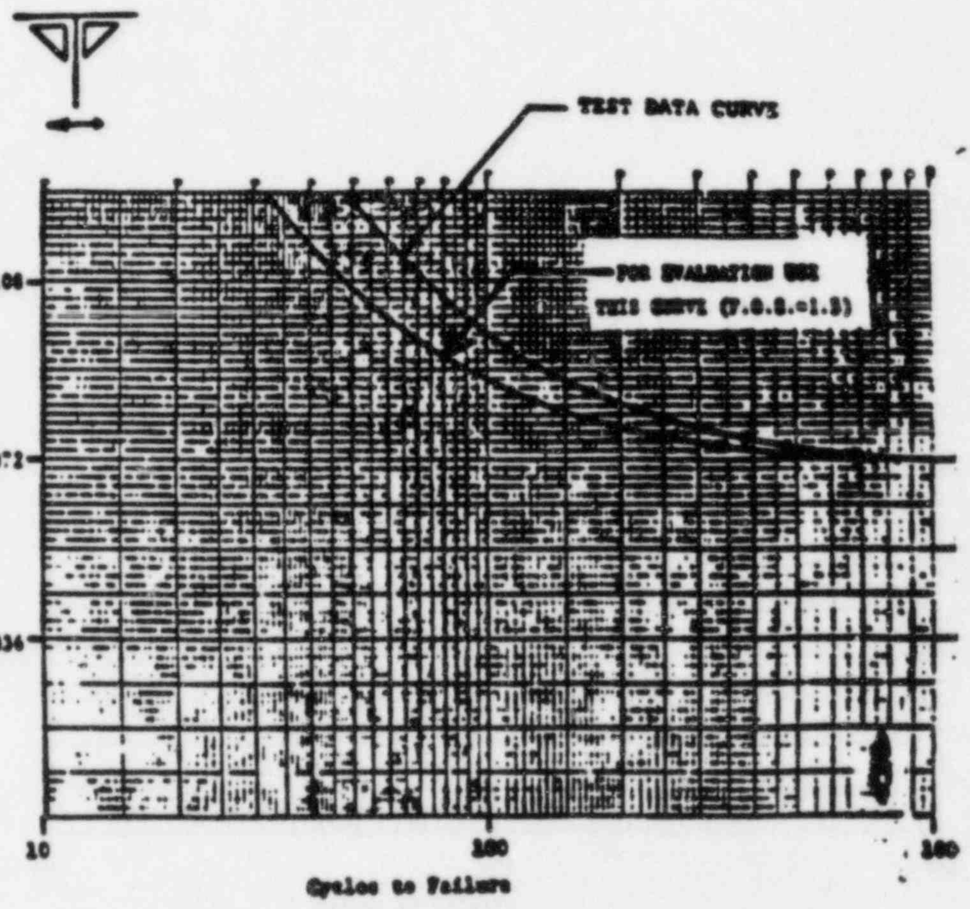
FATIGUE CURVE FOR UNISTRUT P1505--Both Sides



MOMENT-ROTATION CURVE FOR UNISTRUT P1331--Both Sides

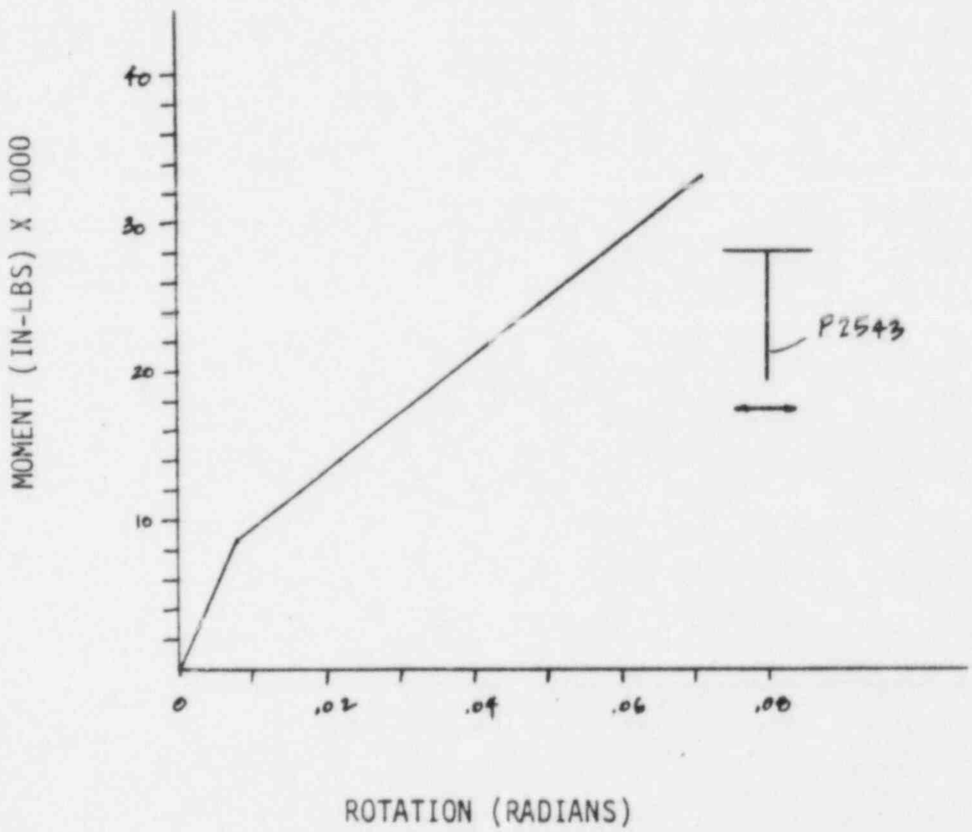
Rotation Versus Cycles

P1331 Both Sides (1331A)

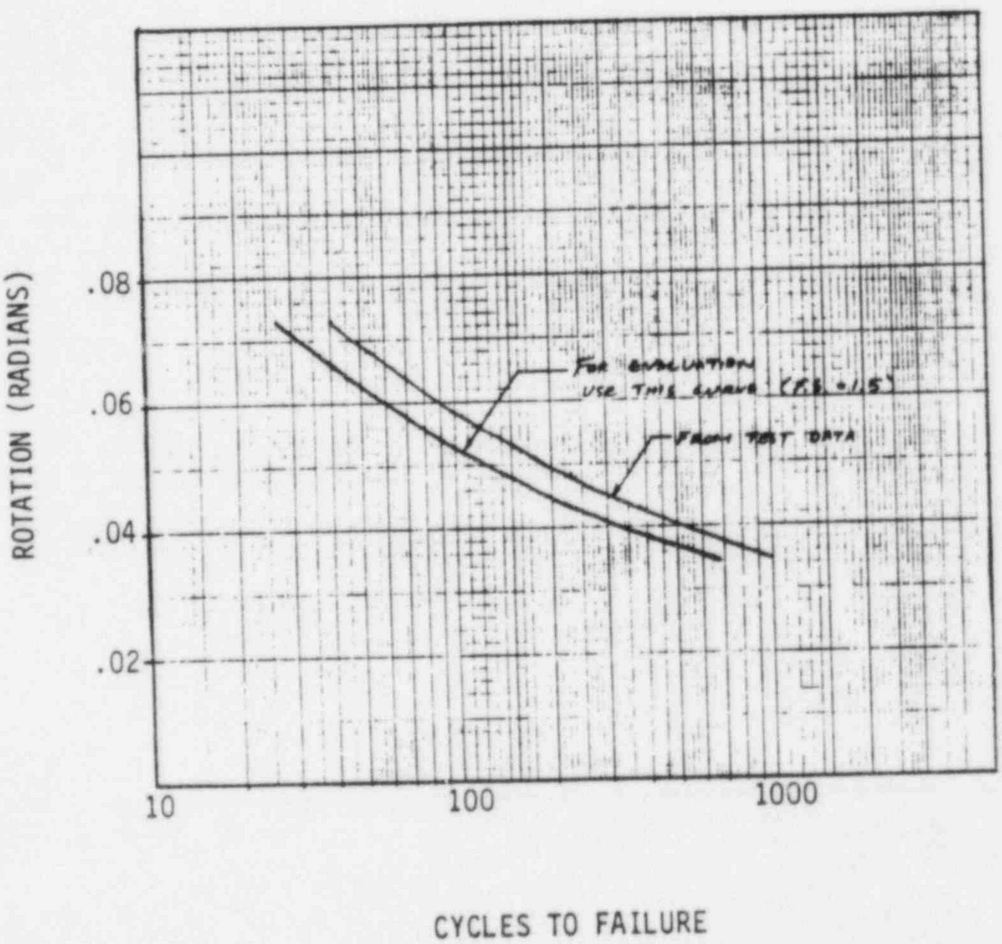


NOTE: 1,000-pound Preload.

FATIGUE CURVE FOR UNISTRUT P1331--Both Sides



MOMENT-ROTATION CURVE FOR UNISTRUT P2543



NOTE: No Preload.

FATIGUE CURVE FOR UNISTRUT P2543

APPENDIX B

NONLINEAR ANALYSES OF TRAIN C CONDUIT SYSTEM

APPENDIX B

Nonlinear Analysis

for

2" Diameter and Smaller Train "C" Conduits

Report no. LIS-1R

Prepared for

TEXAS UTILITIES GENERATING COMPANY

Glen Rose, Texas

Prepared by

GIBBS & HILL, INC.

New York

February, 1986

Table of contents

1. Introduction

In response to Item IC of Technical Review Team (TRT) issues, an action plan was developed in January, 1985 (Ref. 1) to verify the adequacy of the support system installation for non-safety related train "C" conduits less than or equal to 2 inches in diameter. Following this plan, a random sample of 126 runs and an engineering sample of 131 runs were selected for detailed evaluation. Here, a conduit run is defined as a single conduit running between origin and destination (e.g. junction box, panels, etc.). Field as-built information were collected and linear dynamic analysis was performed to compute the earthquake induced forces and moments in the structural members, connections and anchor bolts of the selected samples. The results were compared with the acceptance criteria which limit the member stress to 0.9 of yield stress and a specified factor of safety of 3 to be used for anchor bolt stresses under SSE condition. Based on this comparison, it is found that 14 runs can not be passed due to overstresses of supports.

2. Objective

The conduit system will have some fatique life, even though the range of stress intensity exceeds yield stress and plastic deformation occurs. To predict the fatigue life failure, a simple plastic fatigue analysis can be used following ASME code (Appendix F) utilizing results of an elastic system analysis.

To demonstrate that the simple plastic fatigue analysis is a conservative approach, a detailed plastic fatigue analysis was performed on one of the 14 overstressed conduit runs. The result from both analyses was compared in section 4.

3. Analysis procedure

The general procedure for fatigue analysis can be divided into two parts, simple fatigue analysis and nonlinear fatigue analysis.

I. Simple fatigue analysis :

The support load obtained from linear dynamic analysis is used to evaluate fatigue failure of the support. The stress in each component or the load on the support are compared with the test results and the factor of safety against fatigue failure can be determined. If the support is a statically indeterminate structure, a nonlinear static analysis may be performed to calculate the loads in the components.

II. Nonlinear fatigue analysis (detailed fatigue analysis) :

Nonlinear system dynamic analysis is performed in this step in order to reduce excessive conservatism associated with the simple analysis. The purpose of this analysis is twofold :

a. verify that the simple fatigue analysis is a conservative approach.

b. verify the adequacy of supports.

A flow chart of this task and the detailed description of both analysis procedures are provided in the following pages.

2" Diameter and Smaller Train "C" Conduit Systems
Flow Chart for Qualification of Conduit Systems by Fatigue Analysis

1. Perform system linear analysis for seismic and dead loads. (Response spectrum approach used for seismic analysis)

2. Qualify system
   - pass → 3
   - system fails due to support overstress → 4

3. Conduit system with adequate supports

4. Model overstressed supports considering nonlinear material behaviors

5. Perform nonlinear static analysis to determine nonlinear springs
   - for slightly overstressed supports → 6
   - for highly overstressed supports → 11

6. Perform nonlinear static analysis on supports for combined support seismic and dead loads (from linear analysis item 1)

7. Qualify supports by comparing results with test data
   - fail → 11
   - pass → 8

8. Check redistribution of support loads by performing system nonlinear static analysis using equivalent static load approach. Supports are represented by nonlinear springs (item 5)

9. Verify support which have been qualified in item 2.
   - fail → 11
   - pass → 10

10. System qualified

11. Perform system nonlinear time history analysis using floor accel. time histories as input. Supports are represented by nonlinear springs (item 5)

12. Perform support analysis using load obtained from item 11

13. Qualify supports by comparing results with test data
   - pass → 14
   - → 15

14. System qualified

15. Remove failed supports and check system stabilityiled supports
   - yes → 11
   - fail → 16

16. system failed

1. Perform system linear analysis for the conduit system subjected to seismic and dead loads. This step of analysis has already been completed in accordance with the previous action plan (Ref. 1). In this analysis, the conduits and supports are represented by a finite element model consisting of beam and spring elements, respectively. The seismic analysis is performed using the response spectrum approach and the SSE, 7% damping floor response spectra at appropriate elevation as input. The dead load analysis is performed using the standard finite element static analysis method. The results of these analyses are the loads acting on the supports (support loads) at the connecting points between conduits and support members.

2. Calculate stresses in the support members and components using the support loads obtained from item 1 above. The stresses are then compared with the allowable stresses. The conduit system is acceptable if all stresses in the support members and components are within the allowable limits. If some supports are found to be overstressed they will be removed from the model and the analysis in item 1 is repeated until stresses in all the remaining supports fall within the allowable limits. Otherwise, the fatigue analysis as described in item 4 to 15 below will be required.

3. The conduit system is acceptable if stresses in all the remaining supports are within the allowable limits.

Items 4 to 15 described below are for the fatigue analysis only.

4. Model overstressed supports to account for nonlinear behaviors of UNISTRUT, anchor bolt, connections, etc. The overstressed supports are determined from the results of the linear analysis in items 1 and 2. Elastic-perfectly plastic stress strain curve is assumed for UNISTRUT without considering the strain hardening part. For the connections and anchor bolts, the load-deflection curves obtained from static load tests are used to described the nonlinearity.

5. Perform nonlinear static analysis on the support (item 4) to determine load-deflection curves (nonlinear spring constant) representing global behaviors of the support in all applicable directions.

From the qualification of supports in item 2 described above, if the supports are slightly overstressed, a simple fatigue analysis will be performed (items 6 to 10). Otherwise a less conservative approach based on nonlinear time history analysis will be used (items 11 to 15). The procedure for the simple fatigue analysis is described below (items 6 to 10).

6. For a simple support, the results obtained from linear analysis will be used to evaluate fatigue failure directly. For a complicated support, nonlinear static analysis is preformed on the support model (item 4) using combined support loads of seismic and dead loads from the linear analysis results (items 1 and 2) as input to compute the load in the components of the support.

7. Qualify the supports by comparing the support loads with test

data. The support is acceptable if all the stresses in the bolts, members and connections meet the allowable requirements. Otherwise, a less conservative approach in items 11 to 15 is required.

8. To assure that all supports which have been qualified in items 1 and 2 are not overstressed, it is necessary to investigate the support load redistribution due to nonlinear effects. The nonlinear static analysis will be performed on the conduit system model using an equivalent static load as input. The conduit system model consists of conduits and nonlinear springs (item 5) to represent supports. The equivalent static load can be derived from the response spectrum analysis results. This load will produce at least the same support loads as obtained from the linear analysis using response spectrum and dead load analysis as input. This approach is relatively conservative. The support loads for the overstressed supports are expected to decrease from this nonlinear static analysis while the load in the previously qualified supports may increase due to nonlinear redistribution of the load.

9. The previously qualified supports are re-evaluated as described in item 2. If some supports indicate overstresses then a less conservative approach described in items 11 to 15 is required.

10. The conduit system is acceptable if the results from the nonlinear static analysis (items 8 and 9) are satisfied. This is the end of the simple fatigue analysis.

The following procedure is less conservative and is to be used for highly overstressed supports or for those which can not be qualified by simple analysis (items 6 to 10). The nonlinear fatigue analysis (nonlinear time history analysis) will be used to qualify the supports. The procedure is described below.

11. Perform system nonlinear analysis on the conduit system model which is represented by conduits and nonlinear springs (item 5). The acceleration time histories at the applicable support elevation are applied to the base of the conduit supports. The results of the analysis are the force time histories at the connecting points between the conduits and the supports. The input acceleration time histories are obtained from the previous seismic analysis of the building for the SSE case at the lumped mass location closest to the highest conduit support elevation.

For the nonlinear time history analysis, the acceleration time histories produced by the 3 perpendicular earthquake components are applied simultaneously. For each earthquake component, the rotational time histories at the lumped mass location are transformed into translational component at the floor extreme location of the conduit system and then combined with the major translational time histories. The effects of the other coupling components are included through the use of scaling factors. The gravity load is also included in the analysis.

Since the analyses of the building have been performed for 3 sets of foundation spring constants, lower bound, best estimate and upper bound soil springs, (FSAR section 3.7B.2.1.6), 3 sets of nonlinear analyses shall be performed accordingly. The response spectral peak widening effects shall be considered by shifting frequency of input time history.

The induced force time history at the supports will be used to establish the total number of equivalent load cycles by using linear damage rule.

12. Perform nonlinear dynamic analysis on the support models (item 5) using the force time history obtained from item 11. The induced stress or strain time history in the support component such as anchor bolt can be determined.

13. Qualify the supports according to item 7.

14. System is qualified if all of the conditions in item 7 are met.

15. The failed support will be removed from the model in item 11 and the analysis is repeated from from item 11 through 13 until the conduit system becomes stable.

16. The system is determined to be unstable if it fails to pass steps 11 to 15.

4. Summary of results and conclusion

The random sample no. 44 was chosen for the fatigue analysis. The fatigue analysis was based on the nonlinear time history analysis as shown in the flow chart in section 3. The analysis utilized the computer program NASTRAN version 64. The detailed calculation was summarized in reference 2 and the result of the analysis was summarized in the appendix.

The result of the elastic system analysis indicated that only the anchor bolt of support number 2 was overstressed under SSE condition. Thus only support number 2 was analyzed in detail and the comparison of the results are presented below for SSE condition.

Support no. 2 - Hilti Kwik Bolt 3/8" by 4" embd.

Simple analysis : max. tension = 2943 lbs > 1/3 of ultimate tension
max. shear = 247 lbs << 1/3 of ultimate shear
Ultimate tension (static) = 4750 lbs
Ultimate shear (static) = 5419 lbs

Nonlinear analysis : max. tension = 1469 lbs, max. shear = 194 lbs

According to Reference 4, the Hilti Kwik Bolt can withstand load of less than 1/3 of the ultimate static load for up to 813 cycles without fatigue failure. It is concluded that the simple fatigue analysis is more conservative than the nonlinear fatigue analysis.

5. References

1. Comanche Peak Response Team (CPRT) Action Plan, Item CPRT 1.c, Revision 1, January 11, 1985

2. Gibbs & Hill calculation book no. DMI-17c, Set 10, rev. 0

3. Comanche Peak Steam Electric Station, "2 Inches and Under Train "C" conduit Criteria Document," prepared by Impell, Gibbs & HIll and Ebasco, February, 1986.

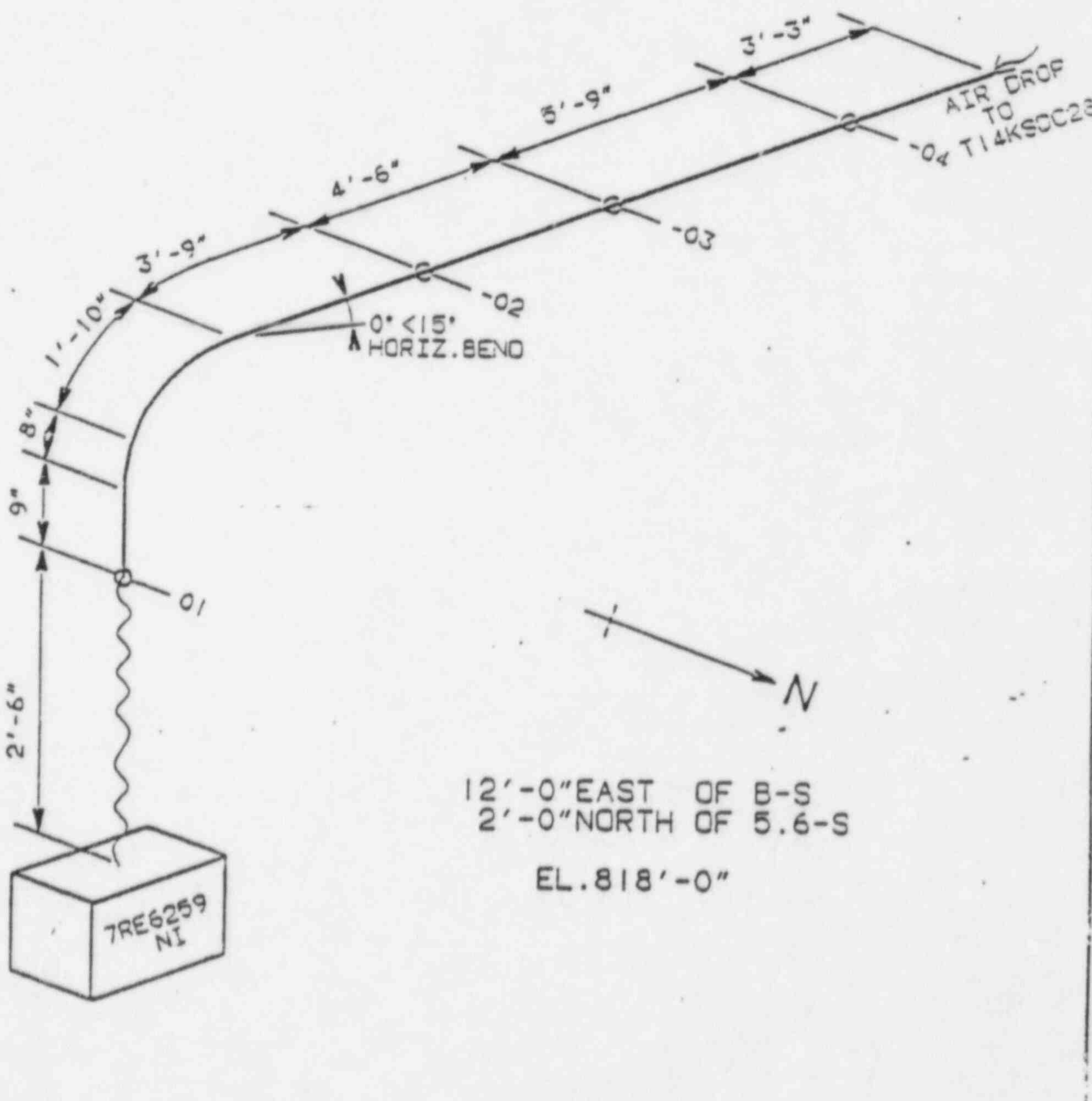
4. Hilti Kwik Bolt Fatigue Tests, Impell Report no. 01-0210-1490, February, 1986.

6. APPENDIX

Figure no.	Description
1 to 5	Isometric sketch and support details
6	Summary of support stiffness used in linear analysis
7 to 10	Plots of nonlinear support stiffnesses
11 to 17	Plots of support load v.s. time for all supports
18, 19	PLots of tension and shear v.s. time in Hilti bolt for support no. 2
20	Typical S-N curve
21	Summary of support loads

FIGURE 1: SAMPLE #44

SCHEMATIC



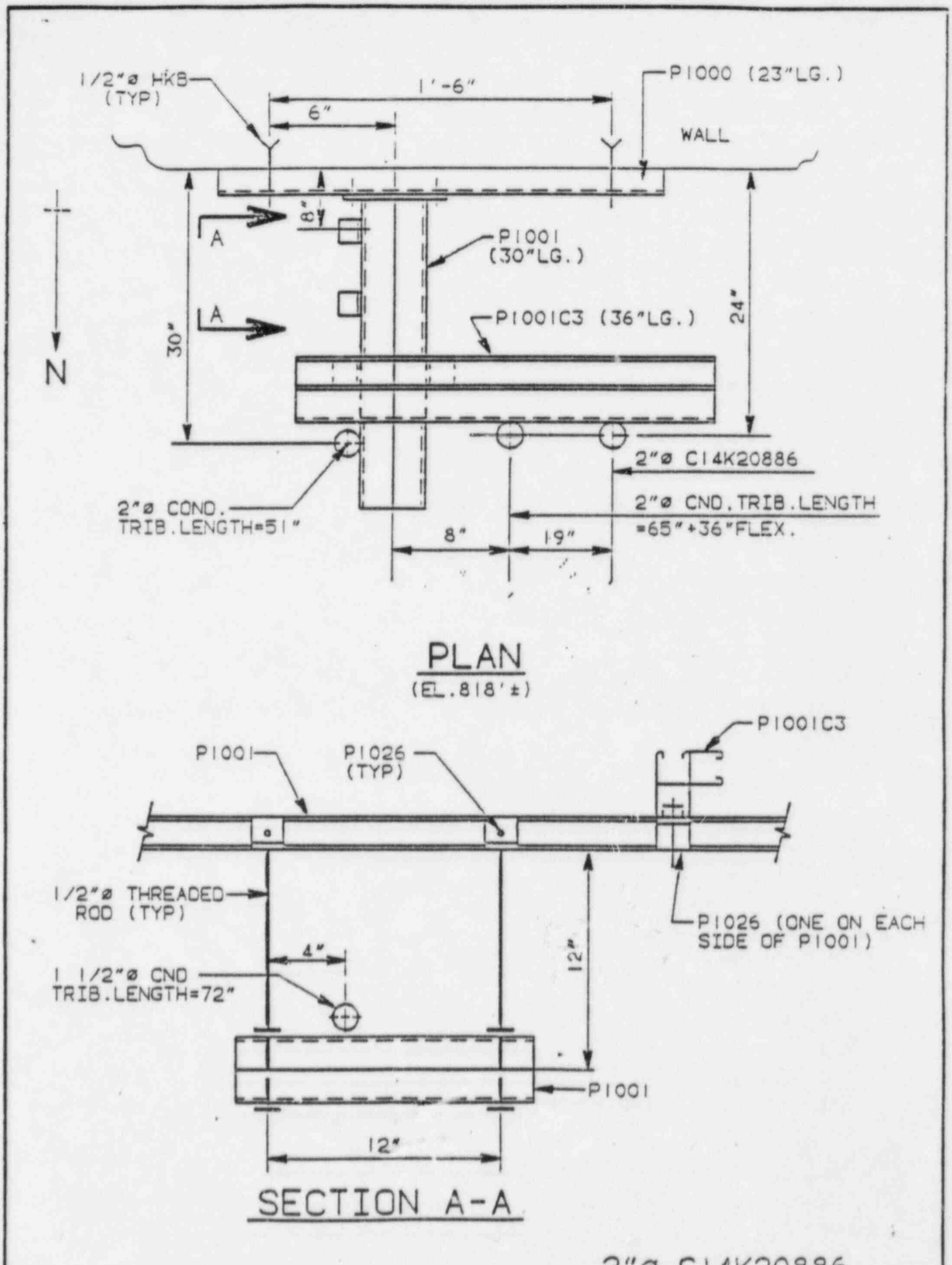
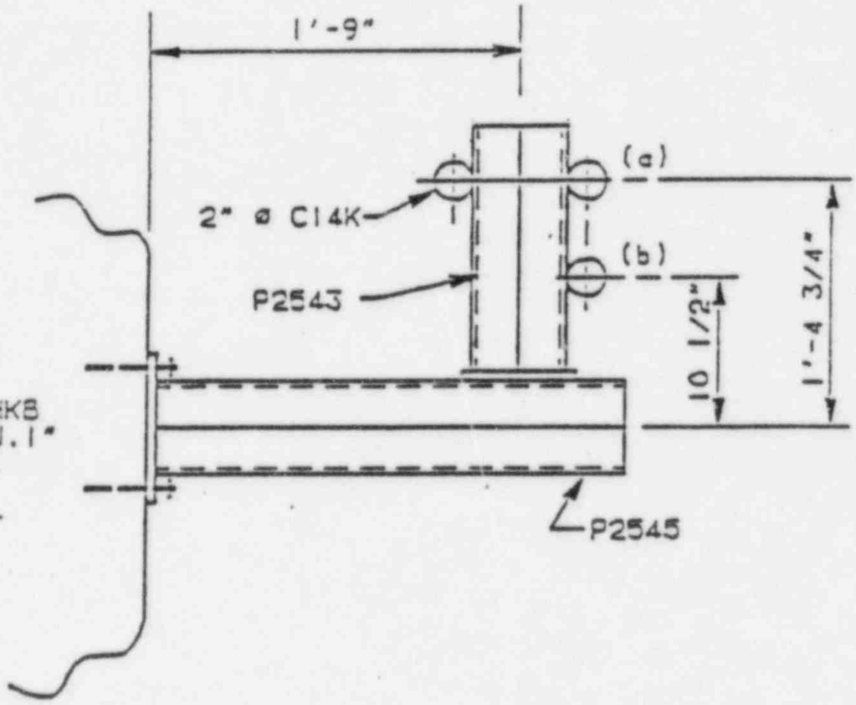


FIGURE 2: 2"ø C14K20886 SUPPORT #01

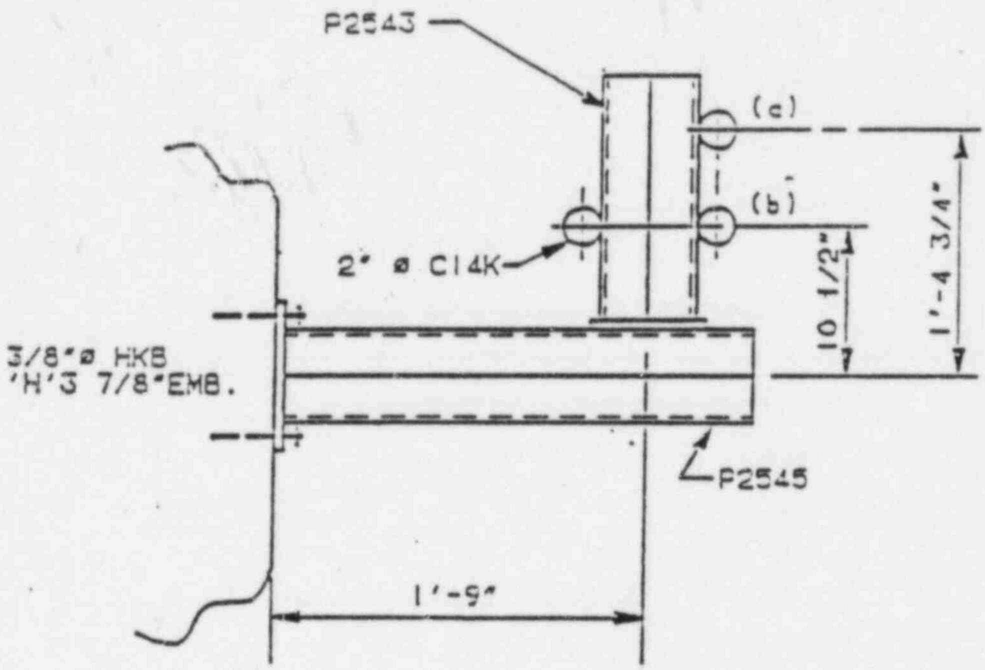


ELEVATION VIEW

COND.	TRIB. LENGTH
(a) 2"ø	6'-3"
(b) 2"ø	5'-6 1/2"

2"ø C14K20886

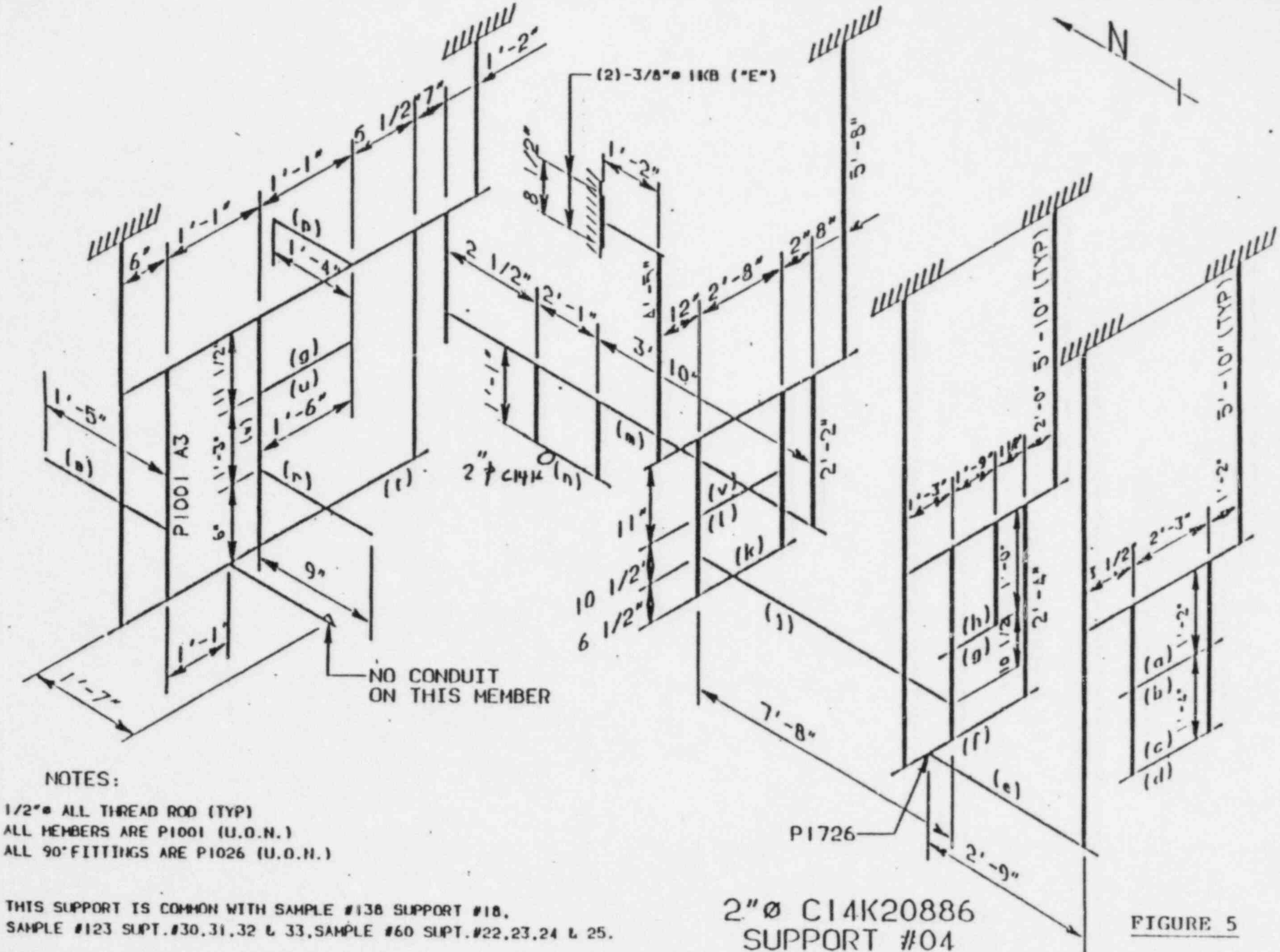
FIGURE 3: SUPPORT #02



ELEVATION VIEW

COND.	TRIB.LENGTH
(a) 2"ø	6'-3"
(b) 2"ø	5'-6 1/2"

2"ø C14K20886
FIGURE 4: SUPPORT #03



NOTES:

1/2"ø ALL THREAD ROD (TYP)
ALL MEMBERS ARE P1001 (U.O.N.)
ALL 90° FITTINGS ARE P1026 (U.O.N.)

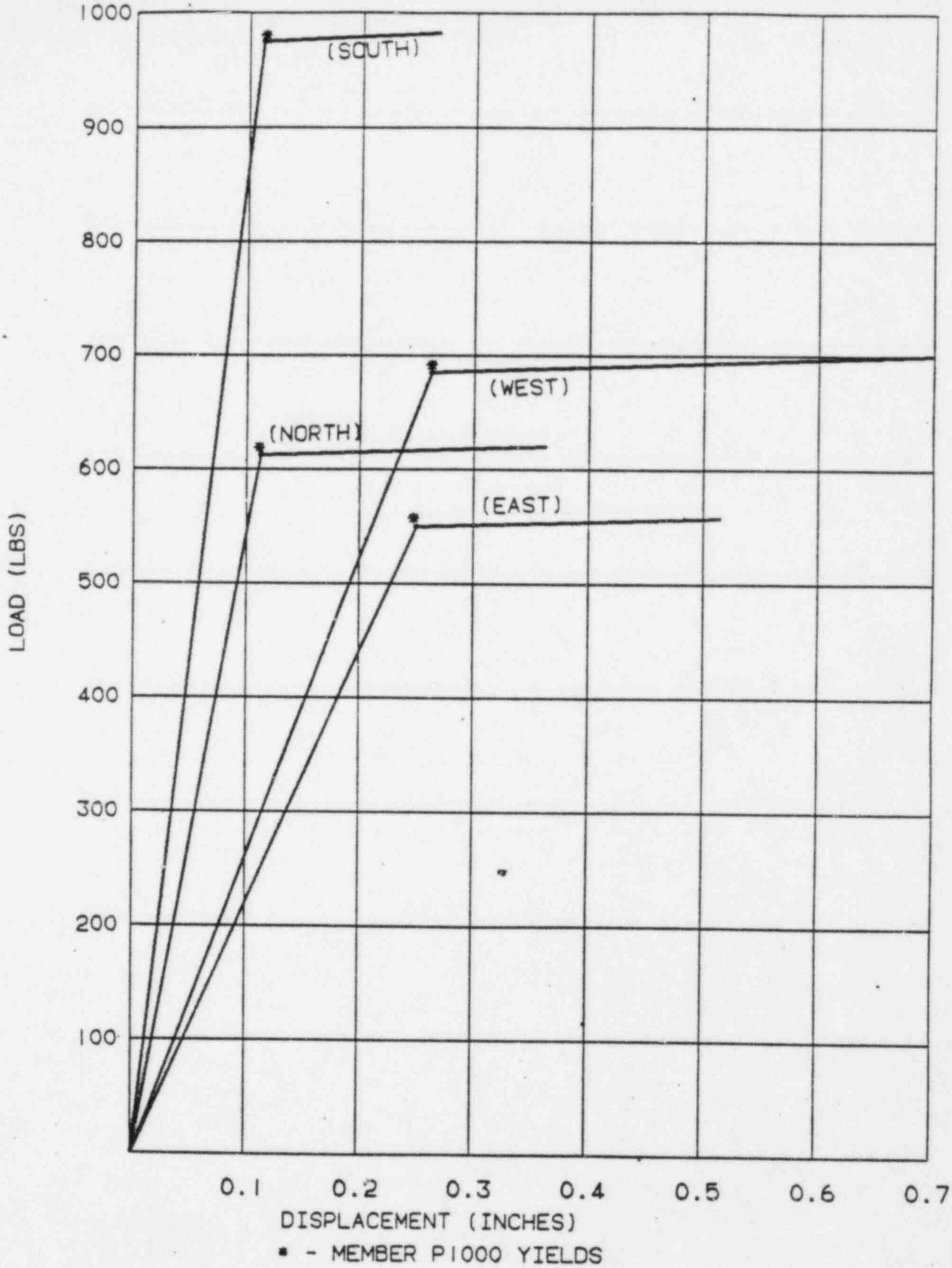
THIS SUPPORT IS COMMON WITH SAMPLE #138 SUPPORT #18.
SAMPLE #123 SUPT.#30,31,32 & 33, SAMPLE #60 SUPT.#22,23,24 & 25.

2"Ø C14K20886
SUPPORT #04

FIGURE 5

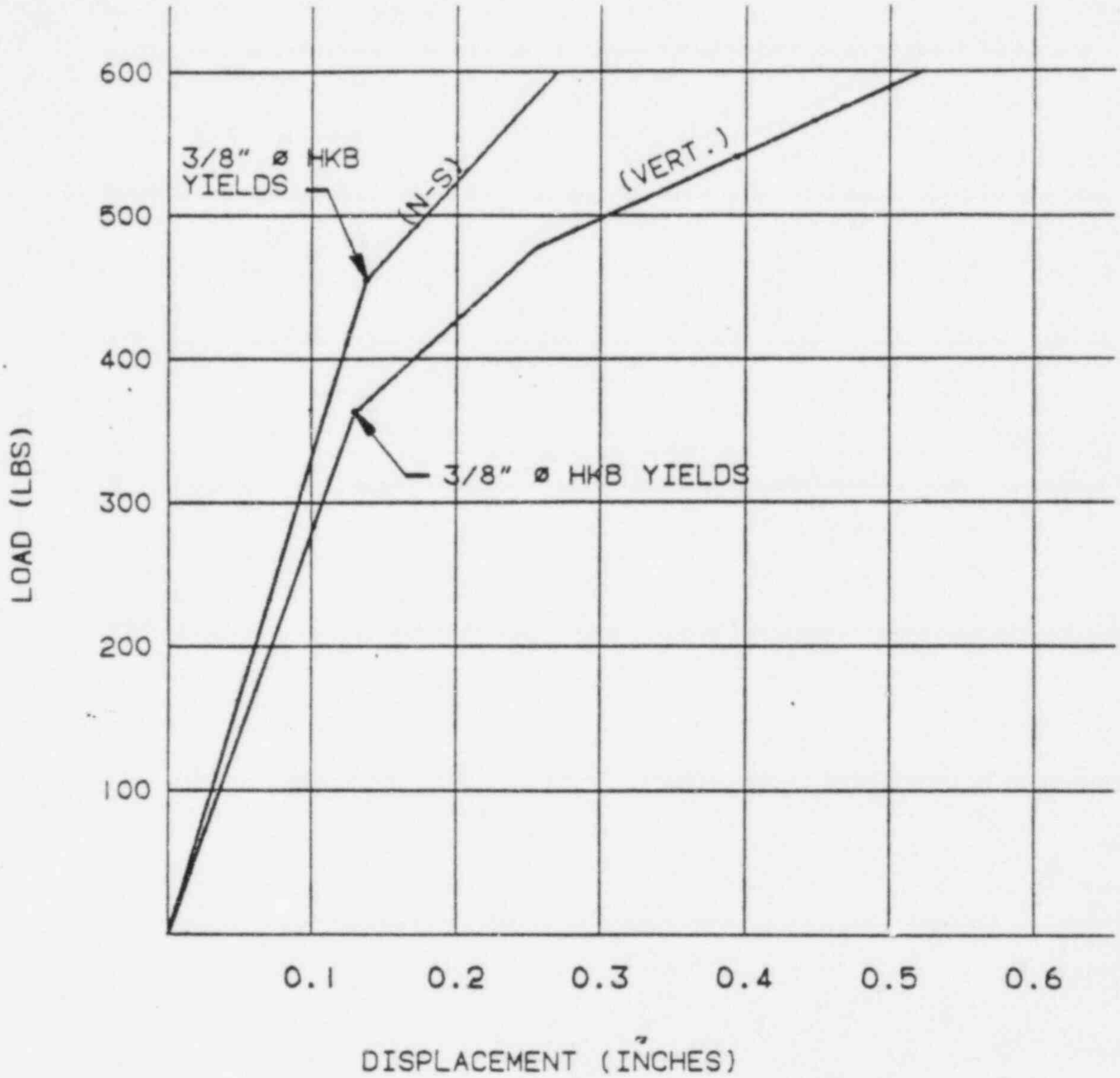
FIGURE 6:
SUMMARY OF SUPPORT STIFFNESSES

SUPPORT NO.	NODE NO.	STIFFNESS (lb/in)			REMARKS
		Kx	Ky	Kz	
1	2	5207	-	2655	
2	5	3026	2829	-	
3	6	4102	2829	-	
4	8	-	7000	-	



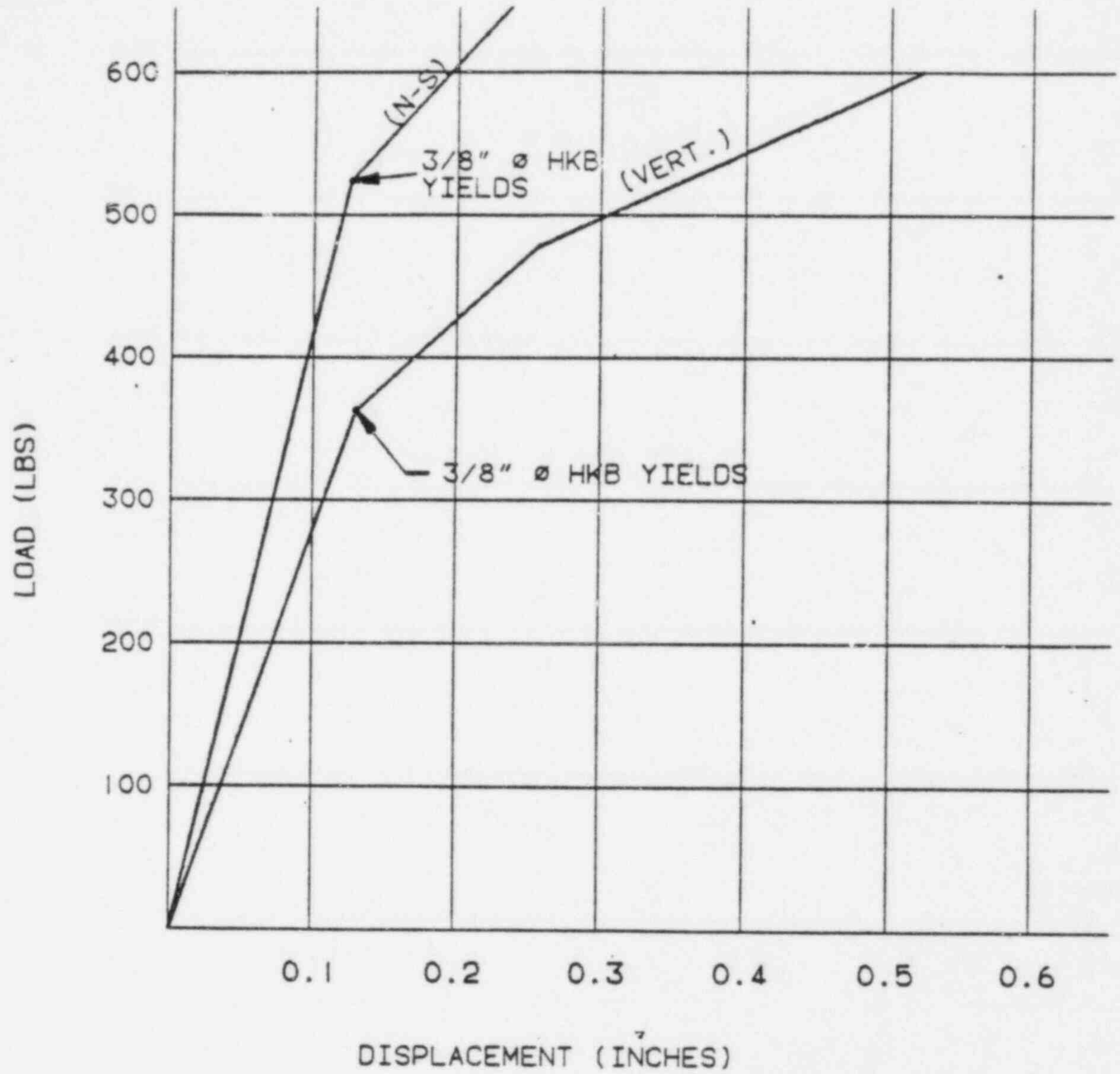
SUPPORT 1 : LOAD - DISPLACEMENT CURVES

FIGURE 7



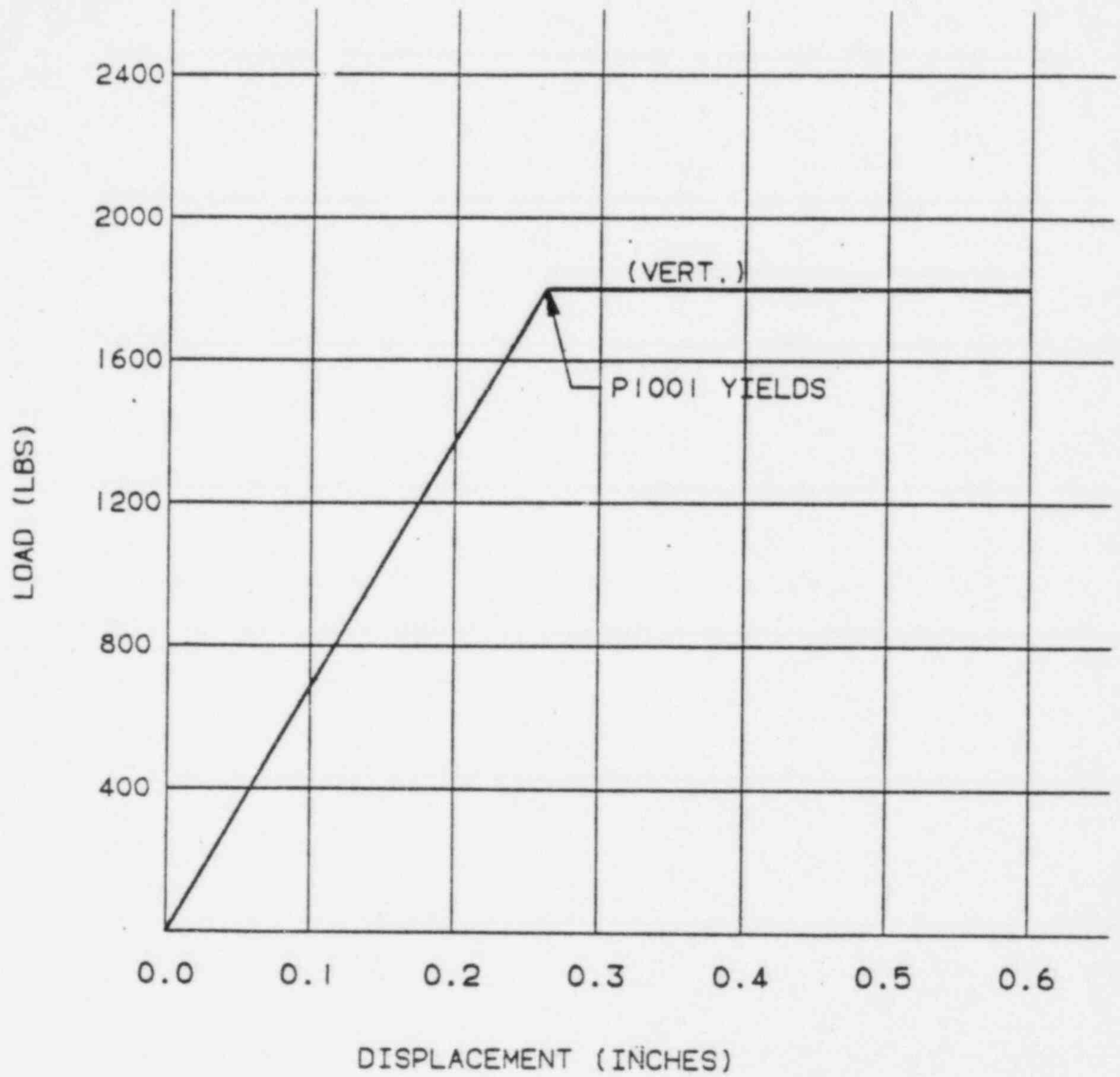
SUPPORT 2 - LOAD - DISPLACEMENT CURVES

FIGURE 8



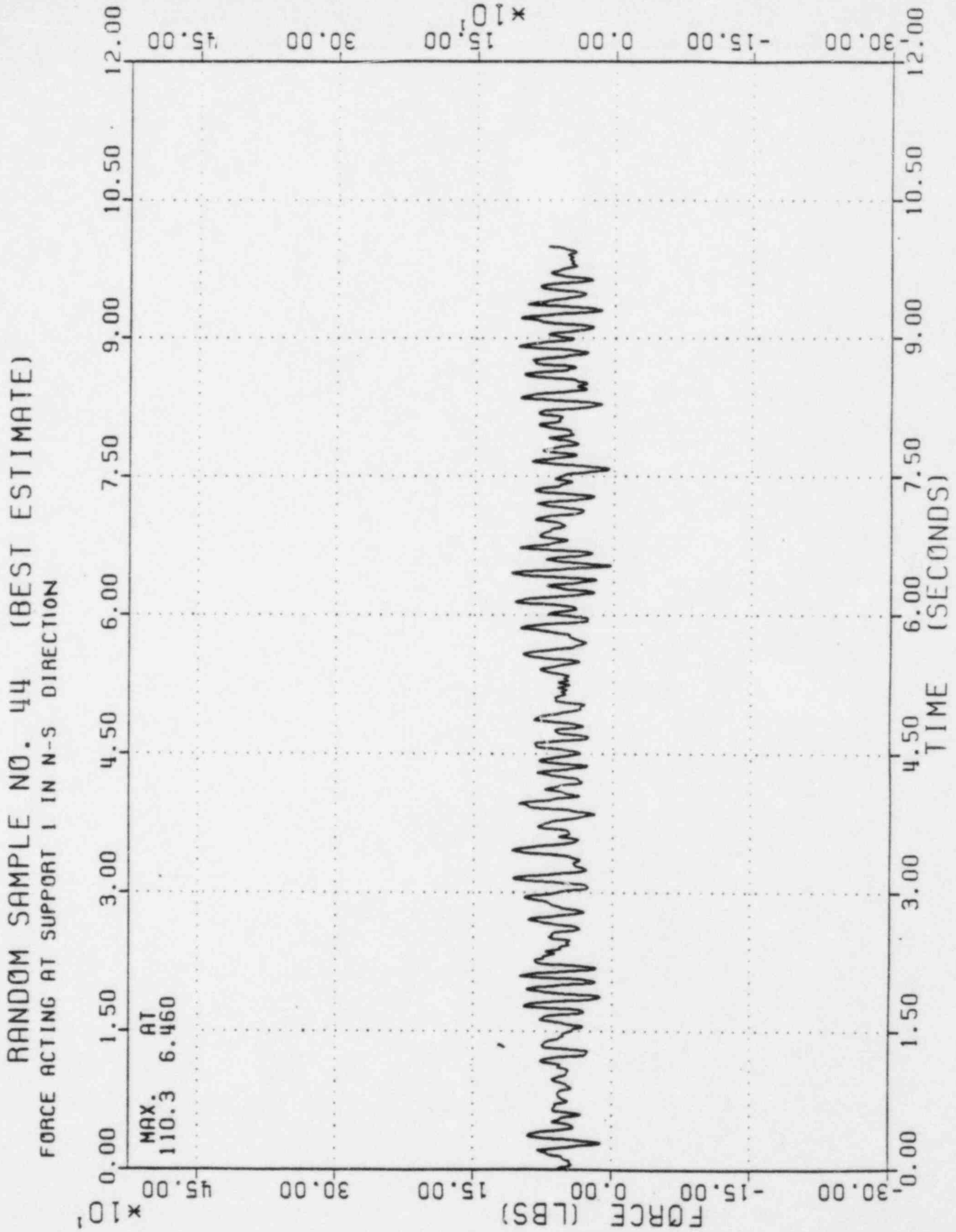
SUPPORT 3 - LOAD - DISPLACEMENT CURVES

FIGURE 9



<u>SUPPORT 4 : LOAD - DISPLACEMENT CURVE</u>

<u>FIGURE 10</u>



TUGCO - 2" CONDUITS

RANDOM SAMPLE NO. 44

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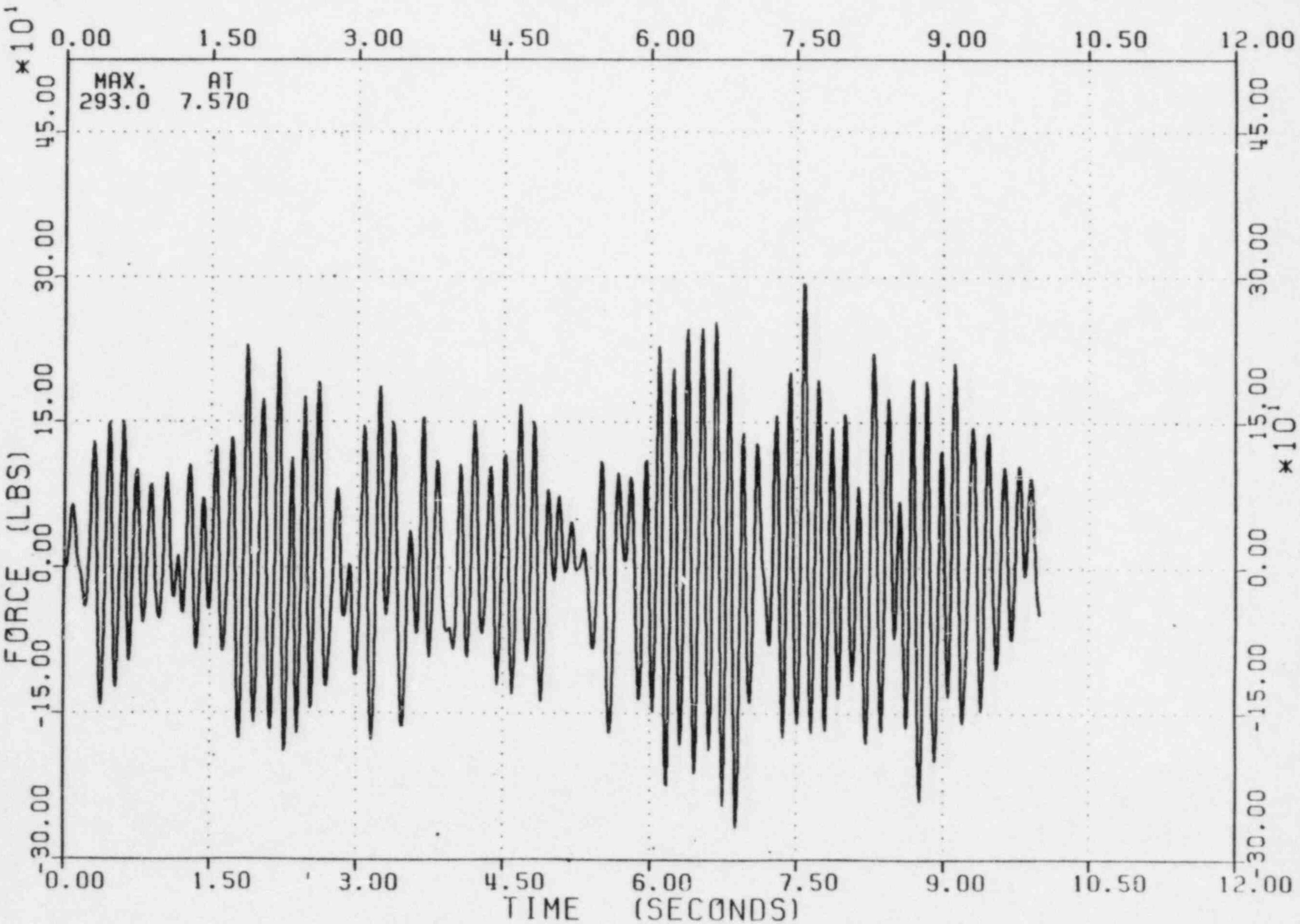
2323-006-1003

FIGURE- 11

ISSUE NO.	DATE	PLTD.	CHKD.	SUP. LDR.	APPROVALS	ISSUED FOR
			1/24/86			
	01/23/86	ADP	WT			

RANDOM SAMPLE NO. 44 (BEST ESTIMATE)

FORCE ACTING AT SUPPORT 1 IN E-W DIRECTION



TUGCO - 2" CONDUITS

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FIGURE- 12

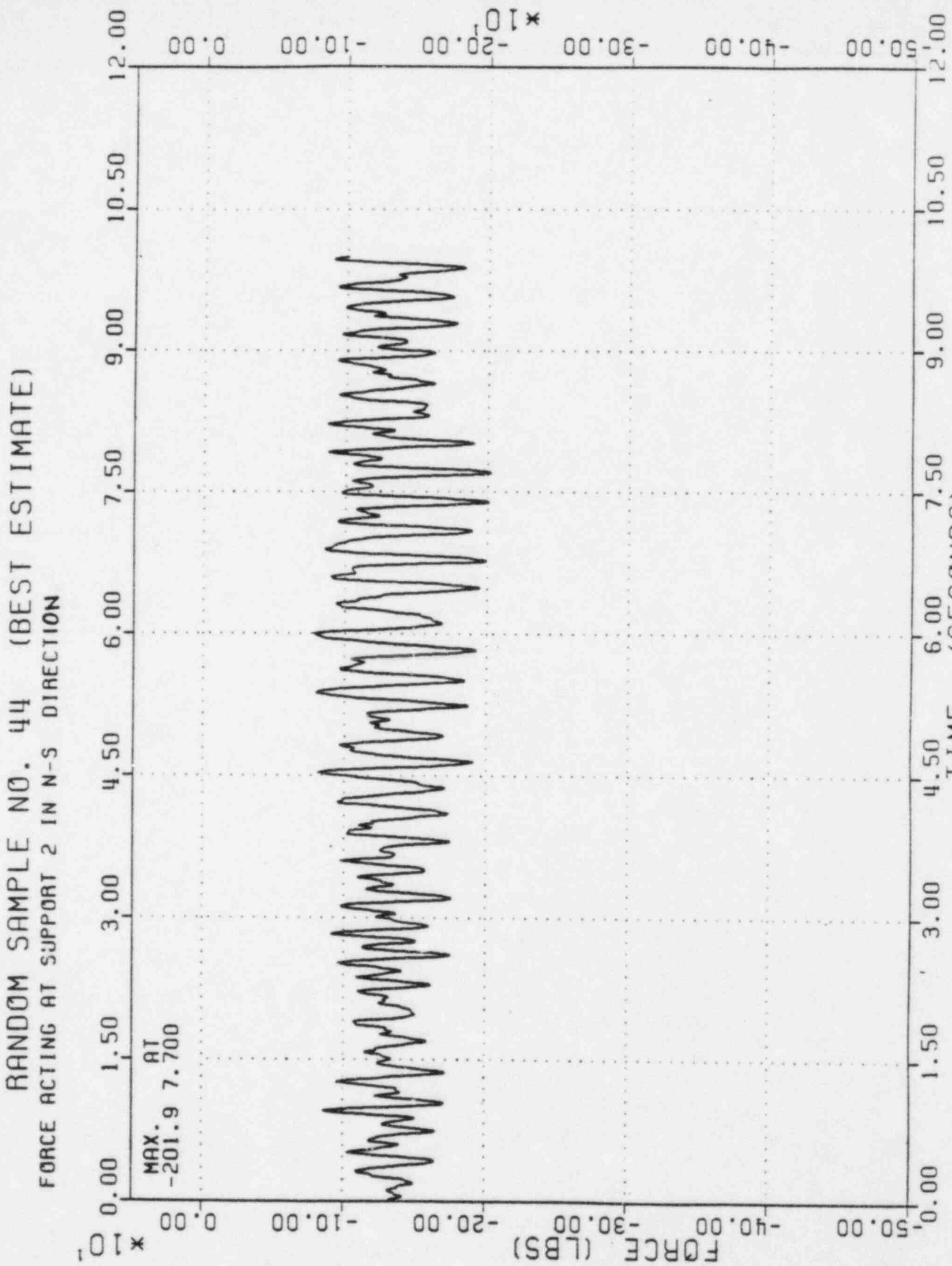

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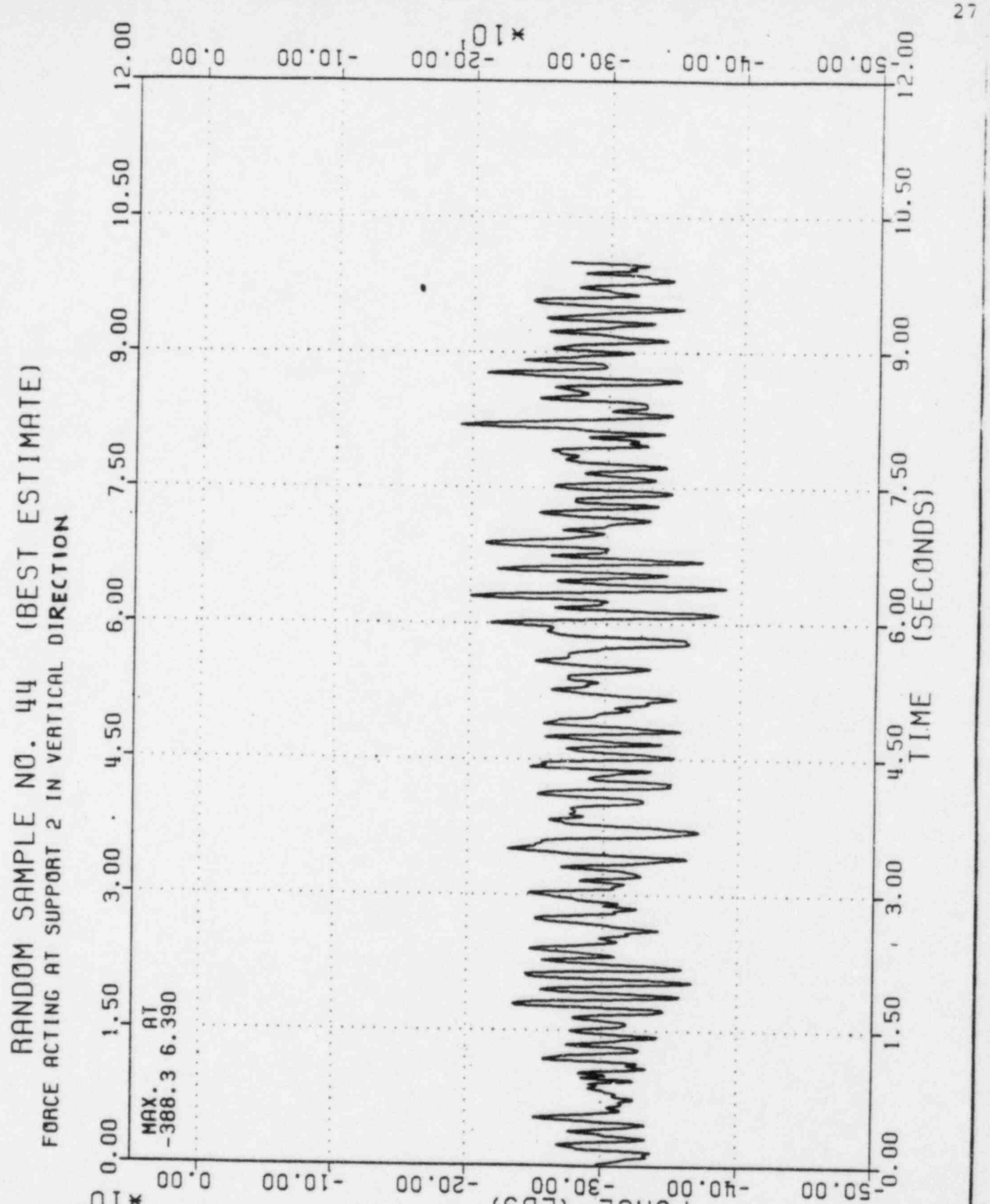
FIGURE- 13

ISSUE NO.	DATE	PLTD.	CHKD.	SUP. LDR.	APPROVALS	ISSUED FOR
	01/23/86	ADP	WT			
		1/24/86				

MECH. STRCT ARCH. ELEC. GEN. & SPECS. CPP. P.S.



		1/24/86		
01/23/86	ROP	WT		
ISSUE NO.	DATE	PLTD.	CHKD.	SUP. LDR.

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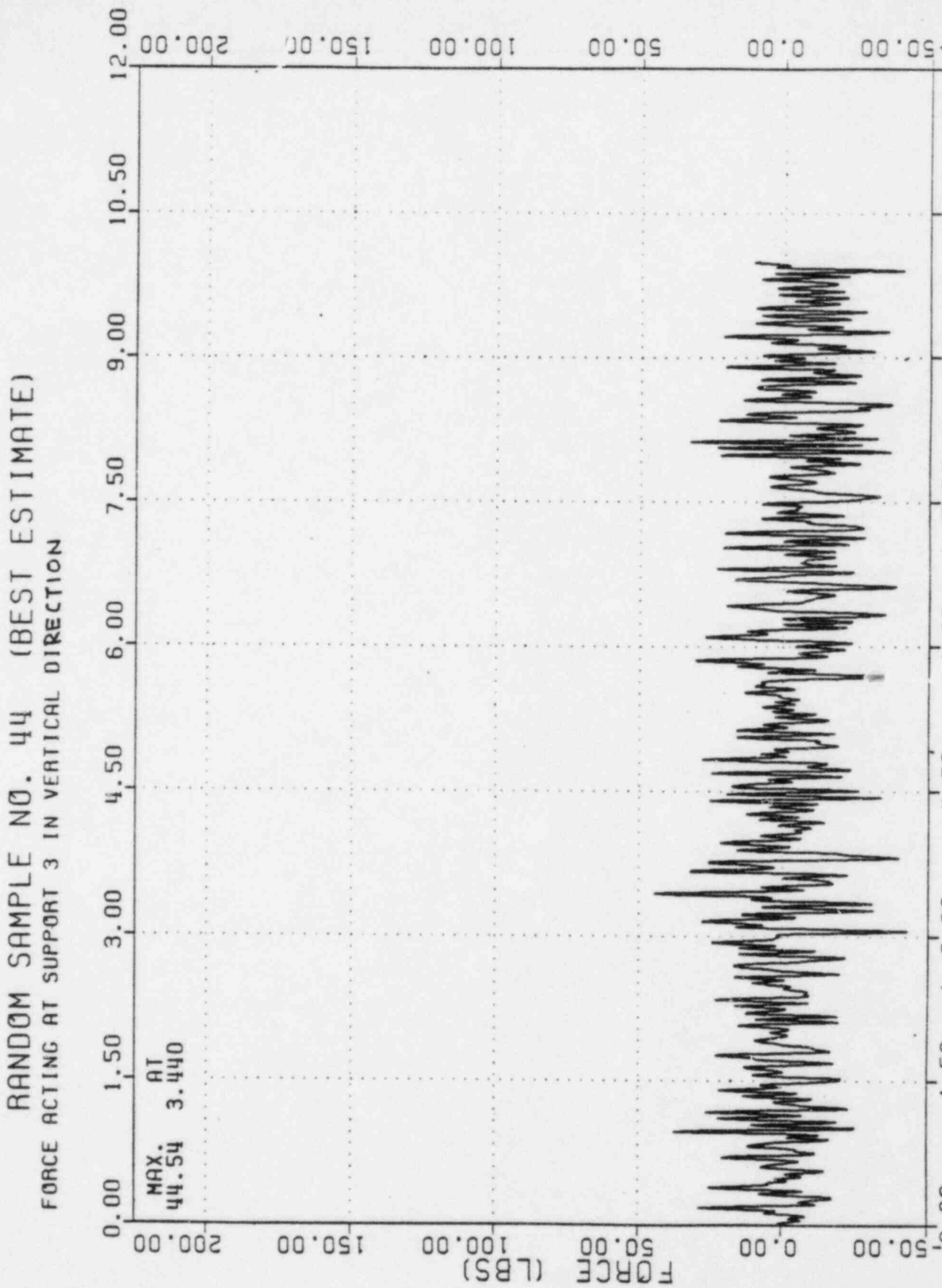
FIGURE- 14



TUGCO - 2" CONDUITS		
RANDOM SAMPLE NO. 44		
GIBBS & HILL, INC. ENGINEERS, DESIGNERS, CONSTRUCTORS NEW YORK	2323-006-1003	
JOB NO. 2323	FIGURE- 15	

ISSUE NO.	DATE	PLTD.	CHKD.	SQD. LDR.	APPROVALS	ISSUED FOR
	01/23/86	ROP	WT			
			1/24/86			

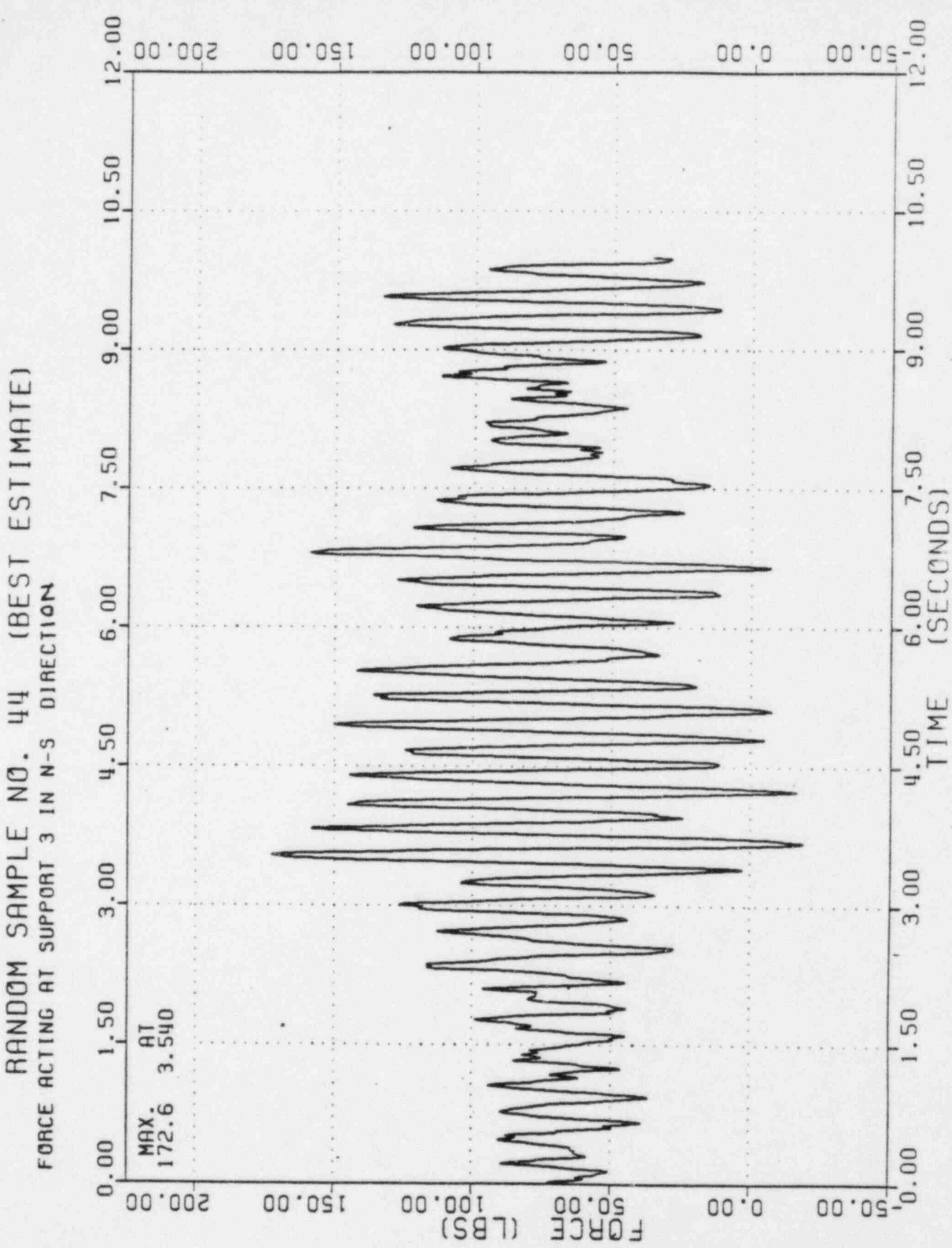


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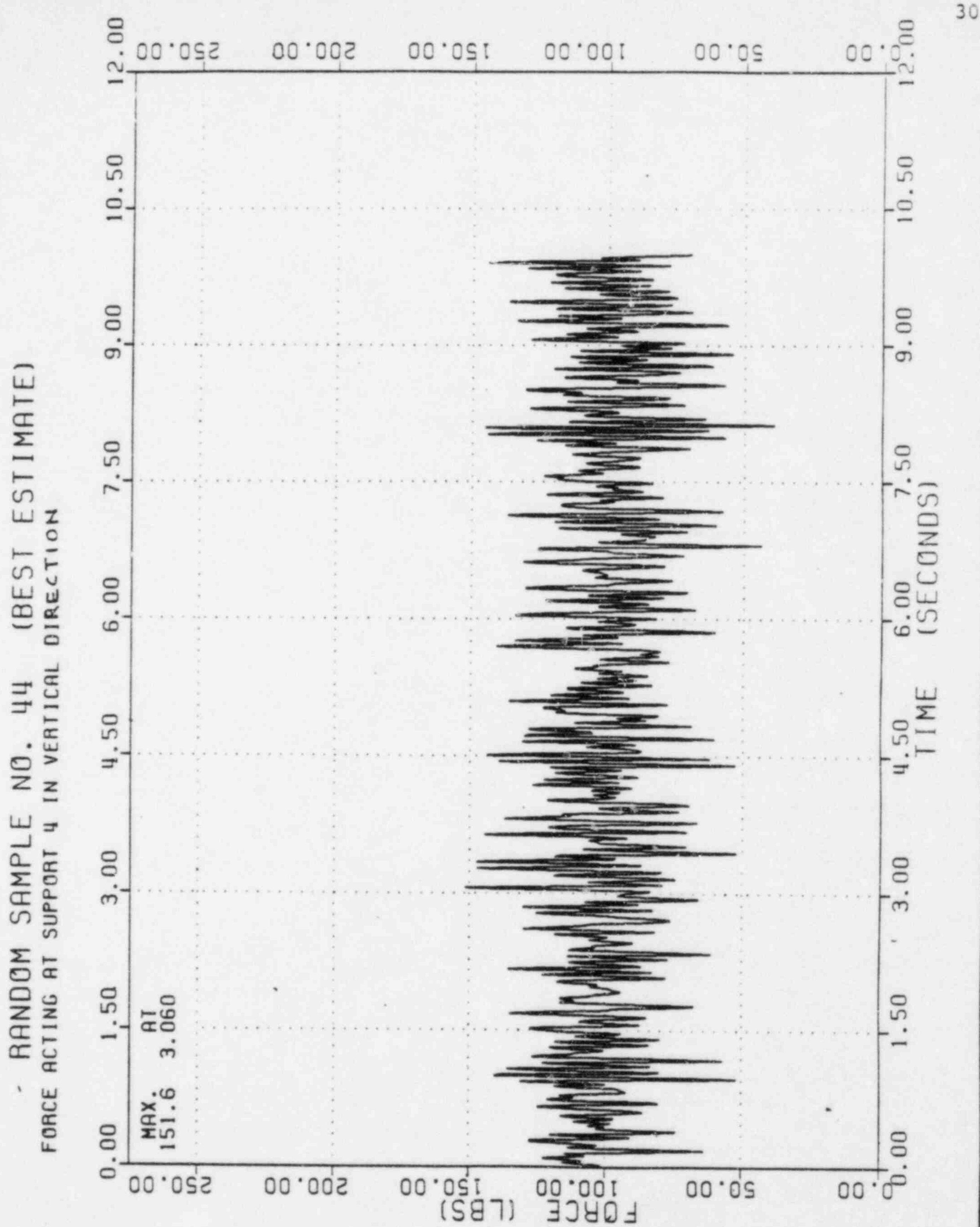
FIGURE- 16

ISSUE NO.	DATE	PLTD.	CHKD.	SUP. LDR.	APPROVALS	ISSUED FOR
			4/24/86			
	01/25/86	ROP	WT			



TUGCO - 2" CONDUITS

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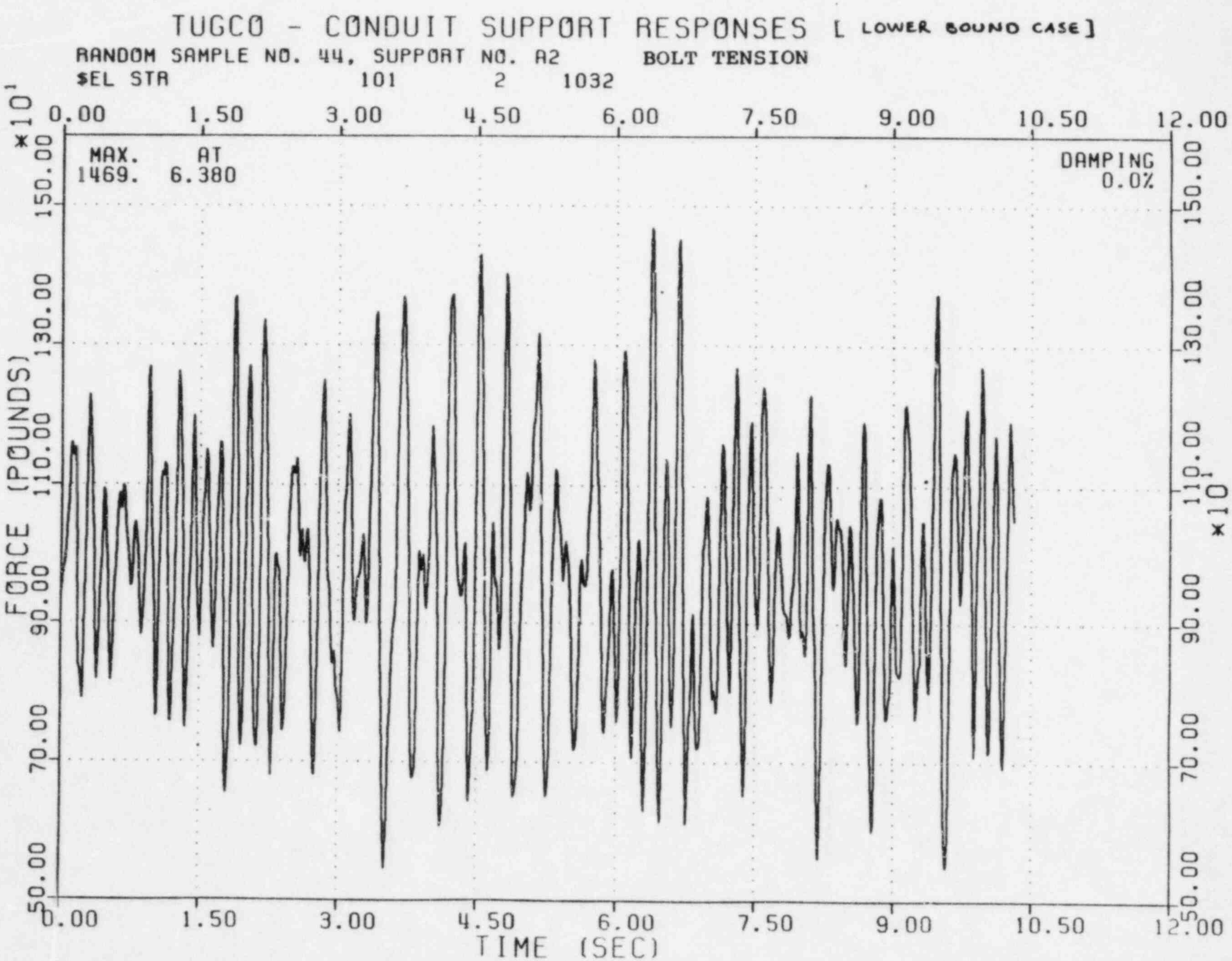
2323-006-1003

FIGURE- 17

ISSUE NO.	DATE	PLTD.	CHKD.	SUP. ENG.	APPROVALS	ISSUED FOR
	01/23/86	RDP	WT 1/24/86			

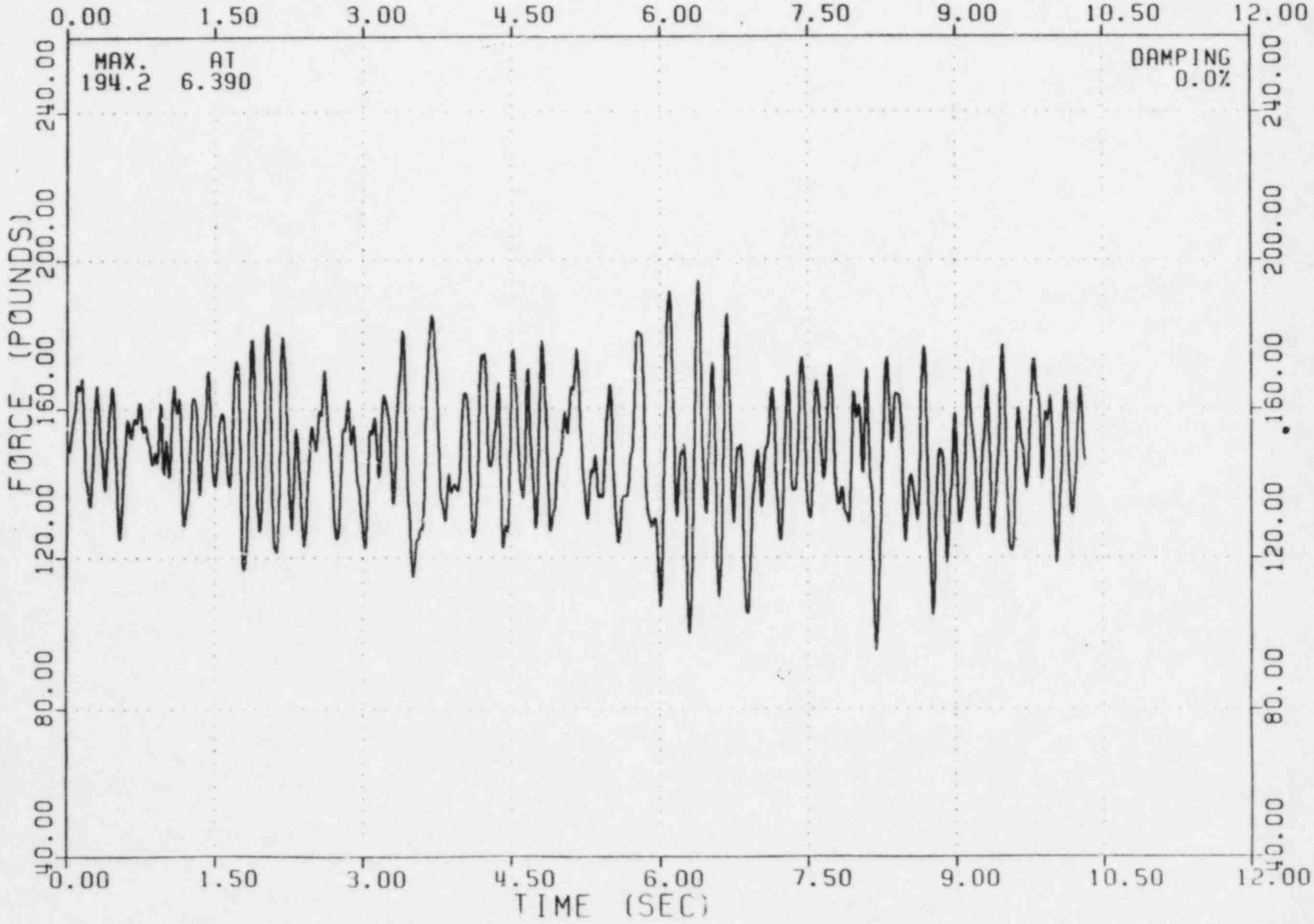


TUGCO-CONDUIT SUPPORT
RESPONSE TIME HISTORY
GIBBS & HILL, INC.
ENGINEERS, DESIGNERS, CONSTRUCTORS
FIGURE- 18

TUGCO - CONDUIT SUPPORT RESPONSES [LOWER BOUND CASE]

RANDOM SAMPLE NO. 44, SUPPORT NO. A2 BOLT SHEAR

$EL' STA 102 2 1032



TUGCO-CONDUIT SUPPORT

RESPONSE TIME HISTORY

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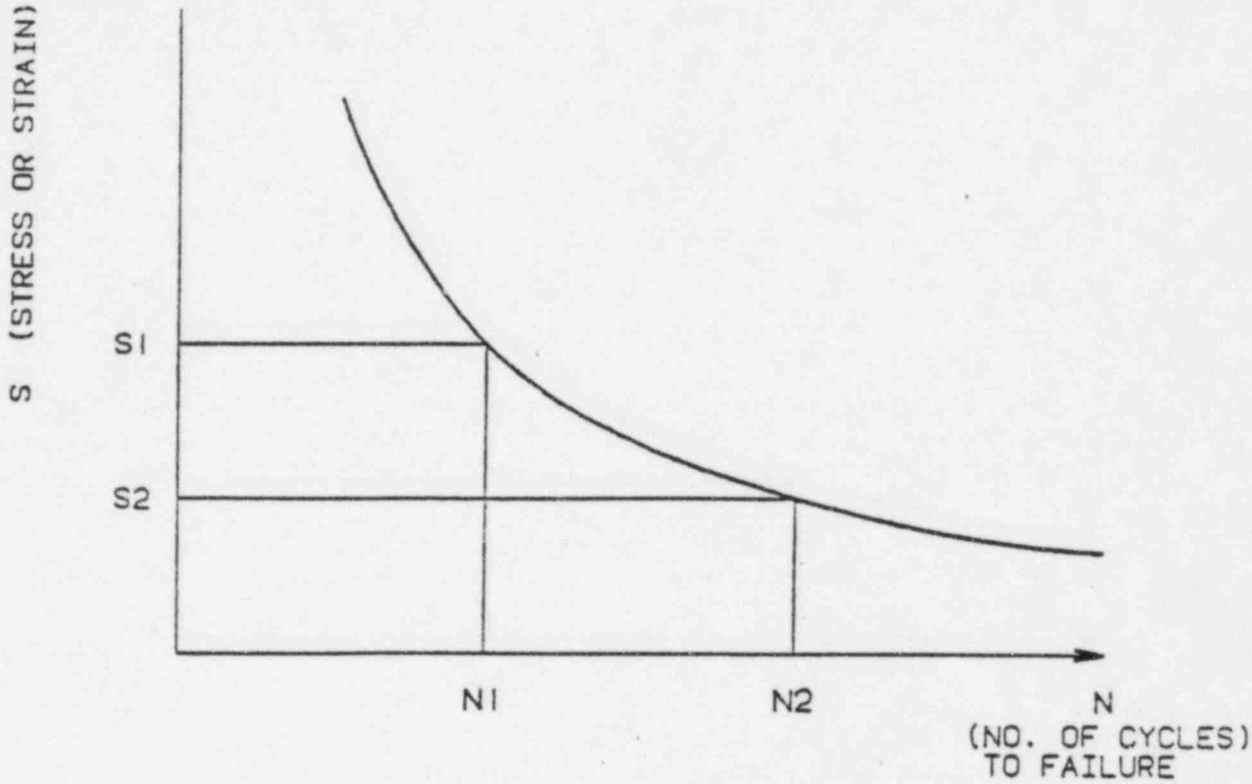
FIGURE- 19

ISSUE NO.	DATE	PLTO.	CHKD.	SUP. LDR.	APPROVALS	ISSUED FOR
	01/23/86	TTF	WT 1/27/86			

ARCH. STRCT MECH. ELEC. GEO.& BALD. ENV. P.M.

3.



ANALYSIS RESULTS: n_1 cycles at s_1

n_2 cycles at s_2

COMBINED DAMAGE EFFECT = $n_1/N1 + n_2/N2 + \ldots\ldots$

FIGURE 20

FIGURE 21: SUMMARY OF SUPPORT LOADS (LBS)

SUPPORT NO.	LINEAR ANALYSIS (DL + SEISMIC)			NON LINEAR ANALYSIS (DL + SEISMIC)					
				BEST ESTIMATE			LOWER BOUND WITH FREQUENCY SHIFTED		
	N-S	VERT.	E-W	N-S	VERT.	E-W	N-S	VERT.	E-W
1	166	-	396	110	-	293	172	-	232
2	356	494	-	202	388	-	325	449	-
3	326	90	-	173	45	-	325	100	-
4	-	260	-	-	152	-	-	146	-

APPENDIX C

HILTI KWIK BOLT FATIGUE TESTS

COMANCHE PEAK STEAM ELECTRIC STATION

HILTI KWIK BOLTS
CONCRETE EXPANSION ANCHORS

JUSTIFICATION OF FACTOR OF SAFETY

Prepared for:

Texas Utilities Generating Company

Prepared by:

Impell Corporation

Impell Report No. 01-0210-1483
Revision 0

February, 1986

COMANCHE PEAK STEAM ELECTRIC STATION
HILTI KWIK BOLTS CONCRETE EXPANSION ANCHORS

JUSTIFICATION OF FACTOR OF SAFETY

Impell Corporation
350 Lennon Lane
Walnut Creek, California 94598

Prepared for:
Texas Utilities Generating Company
Post Office Box 1002
Glen Rose, Texas 76043

This work was performed in accordance with the Impell Quality Assurance Program. The signatures below verify the accuracy of this report and its compliance with the applicable quality assurance requirements.

Authors: J. M. Eidinger

M. D. Ratiu

Reviewed by: T. P. Desmond

Approved by: M. S. Swatta

TABLE OF CONTENTS

1.0 INTRODUCTION

This report provides justification for use of a Factor of Safety of 3 for the Hilti Kwik Bolt concrete expansion anchors used for Train C conduit at Comanche Peak.

In Section 2.0, a detailed technical discussion is provided, describing all factors that relate to Hilti Kwik Bolt strength. In Section 3.0, a report is made concerning the results of a Hilti Kwik Bolt test program specific to Comanche Peak. The technical discussion of Sections 2.0 and 3.0 are summarized in Section 4, Conclusions.

Overall there is a large amount of evidence that a Factor of Safety of 3 provides a high confidence that Train C conduit will not fail due to the SSE event.

2.0 CONCRETE EXPANSION ANCHORS FACTOR OF SAFETY = 3 FOR SSE.

This section describes the justifications for using a factor of safety (F.S.) of 3 for the Hilti Kwik Bolts used in the Train C conduit supports. This Factor of Safety of 3 is implemented in evaluations of Train C Supports [1].

2.1 Normal Practice for Non-Safety Related Structures

For Comanche Peak, the OBE is a severe environmental load condition, and will be encountered infrequently during the plant life. It is reasonable and conservative to call this OBE severe environmental condition a "working" load condition. The Hilti manual states that a F.S. = 4 should be used for maximum "working loads". [2]. The manual further states that the "actual factor of safety to be used depends on the application and should be selected by the designer on this basis."

The criteria used on the Train C evaluation [1] requires that a F.S. = 3 be shown for the SSE load faulted condition on the non-safety related Train C conduit. By using this F.S. = 3 for SSE, it is implied that the F.S. is greater or equal to 4 for the OBE load condition.

Since Train C conduit is non-safety related, the appropriate F.S. for this application should parallel that suitable for non-nuclear (conventional) construction. The AISC manual [3, Section 1.5.6] does not include seismic loads with normal dead and live loads. The AISC manual allows an increase in allowables of 1.33 for seismic loads. It is reasonable for the designer of non-safety related Train C conduit to follow the AISC recommendations and use this same 1.33 factor for Hilti bolt application. By using this AISC increase factor for the Hilti Kiwk Bolts, the F.S. equals 3 for the SSE load condition.

This interpretation of the 1.33 increase factor, for non-nuclear (conventional) application of Hilti Kwik Bolts, is well documented, as described in [4]. In this reference, the City of Los Angeles, Department of Building and Safety, specifically allows a 1.33 increase from the Hilti factor of safety of 4, thereby requiring a F.S. = 3, when designing for wind or seismic loads.

Thus, TUGCo is in full compliance with the Hilti manual when using a F.S. = 4 for OBE (defined as a "working" load), and a 1.33 increase for faulted SSE loads, leading to a F.S. = 3 for SSE.

2.2 Technical Factors Affecting Hilti Kwik Bolt Strength

The ultimate strength of Hilti Kwik Bolts depends upon several parameters. This section describes these factors in regards to their possible relevance to the design factor of safety of 3 for the SSE load conditions.

As a preface, it is noted that the key design assumptions used in the Train C Sampling study [1] are that the compressive strength of concrete is 4000 psi, and that the embedment depth of the Hilti Bolts is taken as the minimum possible, considering the Hilti length tolerances. Both these assumptions are conservative. By considering these conservatisms, the actual criteria F.S. is between 3.45 and 4.1, as described in the following paragraphs.

The technical factors are described as follows:

FACTOR

Embedment Depth
Concrete Strength
Air Content in Concrete
Edge Distance
Bolt Spacing
Aggregate Size and Hardness
Tolerance on Hole Size
Anchor Pre-load
Static or Cyclic Loading
Existing Stresses in Concrete

2.2.1 Embedment Depth

The depth of embedment is an important factor determining Hilti Kwik Bolt ultimate strength: the greater the embedment, the larger the concrete shear stress cone necessary to cause a concrete failure. To account for this, the Hilti manual [2] gives different ultimate loads for all sizes of Hilti Kwik Bolts as a function of embedment length. These ultimate loads are used by TUGCO, with a F.S. = 3, in all evaluations of Train C conduit supports.

Embedment lengths are obtained in the walkdown process by reading the stamped letter on the end of the Hilti Kwik Bolt. This letter denotes the total length of the bolt. The Comanche Peak Hilti Kwik Bolt installation specification [5] specifies that this length can vary by 0.5-inches. To be conservative, the Sampling Study procedure [1] is to always assume the shortest length tolerance for any Hilti Kiwk Bolt. For example, a 0.375 inch diameter Hilti Kwik Bolt that is stamped "D" is between 3 and 3.5-inches long. The engineer then subtracts the length of bolt (say 1-inch) protruding from the concrete surface. Thus, the embedded length of bolt is between 2 and 2.5 inches. For conservatism, the engineer uses the allowable load for a 2-inch embedment and not the 2.5-inch embedment.

For the above example, the amount of conservatism introduced by this procedure is as follows:

- ultimate tension load [2], 2-inch embedment: 3025 pounds
- ultimate tension load, 2.5-inch embedment: 3900 pounds

The actual lengths of the Hilti Kwik Bolts are unknown, but between 3- and 3.5-inches. On average, it may be 3.25-inches. At this length, then the ultimate load is:

(3025 + 3900)/2 = 3462 pounds

Since the allowable load at F.S. = 3 is:

3025/3 = 1008 pounds,

then the actual factor of safety, for the average bolt is:

3642/1008 = 3.43.

Therefore the embedment length factor is conservatively applied. Further analysis of actual embedment lengths could increase the average F.S. from 3 to 3.43.

A similar increase in factor of safety will occur for shear loads.

2.2.2 Concrete Strength

The holding power of wedge expansion anchors depends on the pressure exerted by the expanding part of the bolt on the sides of the concrete hole. One failure mode for the Hilti Kwik Bolts could be initiated by local crushing of the concrete near the wedges. Another failure mode, concrete, shear cone pull out, also depends on the compressive (and thereby shear) strength of concrete. Therefore, the higher the concrete compressive strength, the greater the anchor capacity.

Concrete for Comanche peak safety related structures can be characterized by the following parameters:

- the 28 day specified compressive design strength is 4000 psi.
- air entrainment of 3.5% to 5%
- unit weight of 145 pounds per cubic foot
- cement type II
- maximum aggregate size of 0.75 inch

A recent study tabulated the average and variance of the concrete compressive strength for Comanche Peak [6]. This study included two series of 28 day concrete compressive strength samples. The first sample had 509 cylinder tests; the second sample had 372 cylinder tests. The key results of this study are presented below:

	Sample 1	Sample 2
Number of Cylinders	509	372
Mean Value	5158 psi	5441 psi
Standard Deviation	475 psi	383 psi
Tenth Percentile	4457 psi	4913 psi

From the above data, we can estimate what the actual acceptable strength is according to the ACI code [7]. Section 4.3.1 of ACI 349-80 requires that the average compressive strength must exceed the required compressive strength by 700 psi, if the standard deviation is between 400 and 500 psi. Thus, the sample 1 "nominal" achieved compressive strength would be:

> 5159 - 700 = 4458 psi. (Note: this is very close and in good agreement to the tenth percentile requirements of ACI 214-65, "Recommended Practice for Evaluation of Compression Test Results of Field Concrete," [8])

Similiarily, ACI requires that the average compressive strength exceed the required compressive strength by 550 psi, when the standard deviation is 300 to 400 psi. Thus, the sample 2 "nominal" achieved compressive strength would be:

> 5441 - 550 = 4891 psi. (Note: this is again very close to the tenth percentile requirements of ACI 214-65)

Next, we consider the effects of aging on compressive strength behavior of concrete. In Figure 1, a plot is given for the effect of aging (from 3 to 7 to 28 to 56 to 90 days) for three batches of concrete. This concrete was prepared according to TUGCo concrete mix procedures. This concrete was used in the test fixtures used for the tests described in Section 3.0 of this report, and is the same concrete mix design used for Comanche Peak safety related structures. A total of 46 cylinder tests were included in that study. The statistics for the compressive strength at 28 days and 90 days for these cylinders are:

	28 days	90 days
Number of Cylinders	9	10
Mean Value	5229 psi	6480 psi
Standard Deviation	108 psi	100 psi
Minimum	5010 psi	6300 psi

Since the mean 28 day compressive strength for the test fixture concrete (5229 psi) compares closely with that seen in the larger sample [5158 psi to 5441 psi, Samples 1 and 2], and both samples are based on the CPSES concrete mix design, it is reasonable to expect that the strength increases over time from Figure 1 would be similarily achieved from the above larger sample, and for the plant as a whole. This strength increase is 24%. Further, since the Train C Hilti Kwik Bolts are required to withstand the SSE, at the earliest, in 1986, and the concrete pours were completed by 1982, then the extra strength due to this longer aging period is appropriate. From [9] the typical concrete compressive strength increase from 90 days to 4 years is 10%.

To summarize the above key numbers (Sample 1):

- specified 28 day compressive strength = 4000 psi
- achieved 28 day compressive strength = 4458 psi
- strength increase, 28 days to 90 days = 24% = 1069 psi
- strength increase, 90 days to 4 years = 10% = 553 psi
- achieved 4 year compressive strength (minimum) = 6080 psi
- ratio, average to minimum compressive strength = 1.157
- achieved 4 year compressive strength (average) = 7034 psi.

Based upon Sample 2 data the achieved minimum 4 year compressive strength is 6671 psi, and the average 4 year strength is 7421 psi.

TUGCo has designed the Train C conduit Hilti Kwik Bolts with a F.S. = 3, based upon the 28 day concrete compressive strength. At the earliest possible occurrence of the SSE during operation of Comanche Peak, the minimum concrete strength is 6080 psi. It is acceptable to base the ultimate strength of the Hilti Kwik Bolts based upon this higher concrete strength.

A review of the Hilti catalog shows that for 0.375-inch and 0.5-inch diameter bolts (those are the sizes used for the Train C conduit), the minimum strength increases from 4000 psi to 6000 psi concrete are:

- 0.375-inch bolts: 12.5%(tension); 15.6%(shear)
- 0.50-inch bolts: 24.2%(tension); 12.3%(shear)

For 6080 to 6671 psi concrete, we can expect a further small increase in these values. Therefore, a 15% increase is a reasonable lower bound estimate of the total increase in the Hilti Kiwk Bolt ultimate strength, considering bolt diameters, tension and shear loads, and 6080 to 6671 psi concrete.

Thus, considering the effect of higher strength concrete alone, the actual F.S. is:

$$F.S. = \frac{\text{Catalog Ultimate adjusted for 6080 (to 6671) psi concrete}}{\text{Working Allowable at 4000 psi concrete}} \; 3.45$$

Considering the average concrete strength (7034 psi to 7421 psi), the 15% increase noted above for 6080 psi to 6671 psi concrete would increase to 20%. Then, the F.S. is:

$$F.S. = \frac{\text{Catalog Ultimate adjusted for 7034 (to 7421) psi concrete}}{\text{Working Allowable at 4000 psi concrete}} = 3.60$$

Combining this concrete strength parameter with the fact that the average embedment length is longer than that used for design, there is an additional increase in the F.S. for tension loads, as follows

F.S. = 3.94 (Minimum concrete strength at 4 years, average embedment length)

F.S. = 4.11 (Average concrete strength at 4 years, average embedment length.

A similar increase in Factor of Safety will occur for shear loads.

2.2.3 <u>Air Content in Concrete</u>

The amount of air entrained in concrete can have an effect of Hilti Kwik Bolt strength [10]. Expansion anchor performance is relatively unaffected by air entrainment up to about 7%. Since the concrete mix design for Comanche Peak keeps air entrainment below 5%, then air entrainment has no effect on the F.S. = 3.

2.2.4 <u>Anchor Bolt Spacing</u>

Anchor spacing is an important factor determining ultimate strength of Hilti Kwik Bolts. This is because if the bolts are spaced too close together, the overlap in concrete stress cones can reduce the capacity of individual bolts.

Hilti bolts have been installed per CEI-20 [5], which incorporates minimum bolt spacing requirements. This procedure assures the correct strength calculation of Hilti Kwik Bolts, considering anchor bolt spacing. This factor therefore has no effect on the F.S. = 3.

2.2.5 <u>Edge Distance</u>

Edge distance can affect the ultimate strength of a Hilti Kwik Bolt if the minimum edge distances recommended by Hilti [2] are not met. The Hilti bolts have been installed per CEI-20 [5], which has requirements on minimum edge distances.

The above procedure assures the correct strength calculation of Hilti Kwik Bolts, considering edge distance. This factor therefore has no effect on the F.S. = 3.

2.2.6 Aggregate Size and Hardness

Aggregate size and hardness influences the slip behavior of Hilti Kwik Bolts. The greater the aggregate hardness, the greater the anchor capacity, due to the larger force required to crush the aggregate and permit the expander to slip through the wedging device.

A set of tests to measure the slip of Hilti Kwik Bolts set in the concrete mix used at Comanche Peak was performed [see Section 3.0]. In these tests, three bolts were cyclically loaded up to a minimum load corresponding to a factor of safety of 3 (assuming 4000 psi concrete). The maximum accumulated slip in any of the three tests, after 813 full load cycles, was less than 0.023 inches. The maximum slip in 10 full load cycles (maximum expected in any single earthquake [11]), was 0.011 inches. The maximum slip for one SSE and five OBEs, with a bolt loaded to the F.S. = 3 limit, and each earthquake having 10 full load cycles, is under one-sixteenth of an inch.

Therefore, since aggregate size and hardness has insignificant influence on Hilti Kwik Bolt slip, it is concluded that the Comanche Peak concrete aggregate size and hardness have no adverse effect on the F.S. = 3.

2.2.7 Tolerance on Hole Size and Hole Smoothness

An oversize or smooth hole reduces the capability of the Hilti Kwik Bolt wedging device to exert the lateral force required to hold the anchor in the concrete. The hole size for Hilti Kwik Bolts is specified in CEI-20 [5], and is governed by the diameter of the drill bit. The installation procedure for Comanche Peak requires that this drill bit be a carbide tipped drill bit, which produces a rough hole. The installation procedure allows for the use of diamond bits or water cooled core drills, which could create smooth holes, only when rebar are cut, and then only with the approval of the engineer. Therefore it is concluded that the hole smoothness has no effect on the F.S. = 3.

2.2.8 Anchor Pre-load

The installation of the Hilti Kwik Bolts at Comanche Peak is performed using the procedures of CEI-20 [5]. This procedure requires that the bolt be torqued to a specified limit. This torque produces a tensile pre-load in the bolt to insure that the wedges are properly bearing against the sides of the hole. Over time, the pressure of the wedges against the concrete will cause the concrete to creep, and this will cause a gradual loss of pre-load. This may cause a small amount of slip once the bolt is loaded during the SSE.

In a series of cyclic Hilti Bolt Tests [Section 3.0], pre-load was applied to the bolts. Then, 9 days were allowed to give the concrete time to creep. Creep of concrete is known to occur mainly following application of pre-load. The subsequent SSE load tests showed that the maximum slippage of a Hilti Kwik Bolt in the first SSE load cycle, up to a F.S. = 3 load, was 0.006 inches. Subsequent cycles of full SSE load resulted in much less additional slippage, typically under 0.0005 inches of slip per cycle.

Further, test results discussed in [12] show that the ultimate strength for both static and dynamic tests are relatively unaffected by anchor pre-load.

It is therefore concluded that anchor pre-load has no effect on the F.S. = 3.

2.2.9 Static or Cyclic Loading

In the past, about 90% of all anchor bolt tests have been static tests. Of the cyclic tests that have been performed, the results indicate that properly installed anchors have similar performance under static and cyclic loads. However, the tests have typically not been performed for loads more than one-quarter the manufacturer's ultimate load.

To ensure that cyclic loading does not adversely affect the Hilti Kwik Bolts used for Train C conduit supports, a test program was conducted [Section 3.0]. This program tested the cyclic strengths of 0.375 inch Hilti Kwik Bolts, placed in concrete of mix design typical for Comanche Peak, and loaded up to a F.S. = 3 for a substantial number of cycles, and including on out-of-straightness installation defect.

These tests conclusively show that the Hilti Kwik Bolts used for Train C conduit supports at Comanche Peak can easily accommodate cyclic loading up to a F.S. = 3, for more than 813 load cycles. The tests also showed no cyclic degradation of the bolts even for loads near nominal-rated ultimate strength (F.S. = 2 or F.S. = 1.25). Also, as described in Section 3.0, the slippage of even the smallest anchor bolt embedments found at Comanche Peak (2 inches) are less than one sixteenth inch, for the SSE and five OBEs. Similarly results have been shown during cyclic tests described in [12].

It is therefore concluded that cyclic loads have no effect on the F.S. = 3.

2.2.10 Existing Stresses in Concrete

Most bolt tests have been conducted in concrete slabs which are not stressed. Although compressive stresses should have no effect, or may in fact increase strength, tensile stresses may adversely effect anchor bolt strength. In [10], it is suggested that low level tensile stresses that could occur in nuclear plant structures are not expected to be significant on anchor bolt strength.

2.3 Actual Factor of Safety Versus Criteria Factor of Safety

The criteria used in the Sampling studies was F.S. = 3. However, most Hilti Anchor Bolts for Train C conduit are not loaded right up to the criteria F.S. = 3 limit.

The Sampling study reviewed 2413 Train C conduit supports. Of these, 2370 support pass the acceptance criteria of F.S. = 3 (using 4000 psi concrete, minimum embedment length).

For these 2370 supports, a statistical study was performed to determine the actual Hilti Kwik Bolt F.S. Preliminary results from this study are that:

- total 2370 supports
- total 2200 supports have Hilti Kwik Bolts
- the average F.S. for these 1894 Hilti Kwik Bolts = 10
- over 90% (1980) supports have F.S. over 4, assuming 4000 psi concrete and minimum embedment lengths
- over 96% (2110) supports have F.S. over 4, assuming minimum obtained 6080 psi concrete
- 99%+(2190) supports have F.S. over 4, assuming average 7034 psi concrete and average embedment lengths. (Likely 100% if refined analyses performed)

2.4 Statistical Review of Anchor Bolt Failures

The following paragraphs describe the available statistical data concerning the failure rates of concrete expansion anchors, loaded to various Factors of Safety. This section is provided as there does not yet exist a formally accepted industry-wide standard for the factor of safety for concrete expansion anchors. With this statistical data, we can develop a recommended factor of safety, by using the principles embodied in other widely accepted design Codes, like AISC and ACI, and the principle that Train C conduit supports should have a probability of under one in one million of failure during the plant's lifetime.

This section is organized into two parts. In Section 2.4.1, a failure rate for an individual Hilti Kwik Bolt is established, which is low enough in order to ensure that the overall one in one million failure rate is maintained. In Section 2.4.2, the available test data on concrete expansion anchors is reviewed, in order to establish the failure rate of Hilti Kwik Bolts at various Factors of Safety.

2.4.1 Required Hilti Kwik Bolt Failure Rate

A common assumption used in Codes is that the higher the Factor of Safety, the less likelihood that a failure can occur. Based on a strength concept, the typical Code mandated factor of safety is about 3 for working loads, and about 1.6 for infrequent loads, like earthquake motions.

Embodied in these factors of safety is that there is an acceptable number of components, taken from a large sample of components, that will fail under the code mandated factors of safety. For example, the ACI 349 code allows that ten percent of samples taken for concrete compressive strength fall below the required design compressive strength. Similarly, ASTM testing specifications allow a small percentage of steel coupons to fail to meet minimum design yield stresses.

The exact percentage of specimens taken from a sample that are allowed to fail code criteria depends upon the importance of the specimen in the overall design. Also, the percentage of specimens allowed to fail depends on the consequences of failure for the structure being designed.

For Hilti Kwik Bolts used in Train C conduit, it is suggested that an acceptable "failure" rate of individual bolts is 1%. This failure rate is determined based on recognition of the following factors:

- Train C conduit supports are allowed to have large distortions, as there are no functional requirements for Train C electrical systems. All Train C conduit are non-safety related.

- The failure of a single Hilti Kwik Bolt does not cause collapse of the Train C conduit system. This is because Train C conduit is a multiply supported system. The typical Train C conduit is supported at five to seven foot intervals, and has ten supports over its length. In order to cause a Train C conduit to actually fall, a "zipper" effect must occur - ie., multiple (at least two) Train C supports must fail. The failure of a single Hilti Kwik Bolt can almost always be accommodated by the extra capacity in the adjacent support. It is assumed that the probability of two adjacent supports, both having Hilti Kwik Bolts, both failing during an earthquake, is independent, and is the multiplication of each individual bolt's probability of failure, namely 0.01 * 0.01 = 0.0001.

- For nuclear plant design, it is a common assumption that if a postulated event cannot occur with a frequency higher than one in one million, during the lifetime of the nuclear plant, then that event need not be explicitely designed for. The SSE motion can be assumed to have a probability of one in ten thousand of occurring during the plant's lifetime. As described in the previous paragraph, the probability of a single Hilti actually falling is assumed to be 0.01; or 0.0001 to cause a zipper type failure of an entire conduit run. Therefore, the chance that both the SSE occurs and one Hilti Kwik Bolt connection fails is less than one in one million. Further the chance that one entire conduit run actually fails is one in one hundred million.

  Therefore, a failure rate of 0.01 for an individual Hilti Kwik Bolt is acceptable for Train C conduit supports.

2.4.2 Actual Hilti Kwik Bolt Failure Rate

There have been prior studies to examine the failure rates of concrete expansion anchors. One statistical study [13] came to the following conclusions:

- number of tension tests = 152
- number of tension failures at F.S. of 4 = 1 (0.7%)
- number of tension failures at F.S. of 3 = 1 (0.7%)
- number of tension failures of F.S. of 2 = 4 (2.6%)

- number of shear tests = 113
- number of shear failures at F.S. of 4 = 0 (0.0%)
- number of shear failures at F.S. of 3 = 0 (0.0%)
- number of shear failures at F.S. at 2 = 0 (0.0%)

Based upon this data, the study extrapolates the probability of failure, by using the mean and standard deviation statistics of the test data, assuming normal distribution of bolt strengths. This statistical evaluation suggests that at F.S. = 2, there is a probability of failure of 2.3%, and at F.S. = 4, there is a probability of failure under 0.1%. Note, however, that the actual test data above suggests lower failure rates for F.S. = 2 or 3, than does the statistical extrapolation. This is either because the sample is too small, or because the physical strength of concrete expansion anchors does not conform to a "normal" distribution.

A more recent study [10] has gathered more test data than used in the statistical evaluation done in [13]. This study includes approximately 2900 tension tests and 1600 shear tests. About 20 to 30 percent of the test were static tests on bolts with intentional installation defects. The statistical analysis of this test data has shown that approximately 1% of the tested bolts fail at or below F.S. = 2. Since this study [10] uses a larger sample than [13], this 1% failure rate is a more likely representation of true failure rates, than the 2.3% reported in [13]. Further statistical review of the test data should show about a 0.1% to 0.5% failure rate at F.S. = 3, and a failure rate much below 0.1% at F.S. = 4.

An additional set of tests, described in Section 3, has been performed to determine the fatigue life of cyclically loaded Hilti Kwik Bolts, at loads for F.S. = 3, = 2, and = 1.25, considering nominal Comanche Peak concrete strength. A total of nine bolts were tested, all including an installation defect. All bolts were able to take a minimum of 813 cycles of load up to the tested factor of safety.

Based upon the above test data, the failure rate for Hilti Kwik Bolts, at F.S. = 2, is likely 1% [10]. At a F.S. = 3, the failure rate of Hilti Kwik Bolts is likely about 0.1% to 0.5%.

3.0 COMANCHE PEAK TEST RESULTS

A set of nine tests of Hilti Kwik Bolt anchorage specimens was conducted to determine their suitability for cyclic loadings at loads at or above a factor of safety of 3. The testing program has been conducted by Corporate Consulting and Development Company (CCL) [14, 15]. The testing program substantially follows the fatigue test procedure described in ASTM E488-84. [16].

The 3/8" Hilti Kwik Bolt anchorage specimens, with 2 inch embedment, were prepared with the maximum permissible out-of-plumbness of 6°. This deviation is the maximum allowed per [5]. This misalignment will produce additional bending stresses in the bolt, affecting the material fatigue endurance.

A uniaxial tensile load is applied with a frequency of 1 Hertz for a total of 813 cycles. For most specimens, the cyclic testing is repeated with another 813 cycles since no failures occured. One test was run at 5 Hertz, with no significant difference in results from the 1 Hertz tests. One test was repeated twice, for a total of 2439 cycles.

The results of all tests are summarized in Tables 1 and 2. All 9 specimens have passed the first program phase of 813 cycles. The subsequent phase with increased loads, up to the static (failure) design load (F_u = 3050 lbs) [2 and 17] reveals that the anchorage strength is limited by the fatigue strength of the bolt steel.

The bolt steel is a free cutting steel AISI 11L41, with high contents of sulfur (0.13%) and lead (0.15% to 0.35%). This steel has a high notch sensitivity and, therefore, the bolt failure occurance at the load of 3060 lbs after 733 cycles is expected.

The cummulative loading cycles by different load between 1050 lbs and 3060 lbs, are presented in the following load-life diagram (Figure 2). The solid thick line shown in this curve is the lower bound Load-Cycles curve from test data. The two tests marked by diamonds are shown for information only, as the tests were not completed. The thick dottes lines represent extrapolations of the data, and should not be used without further test results.

The test results indicate the cyclic loading suitability of the Hilti Kwik bolt anchorages for service conditions up to 3000 lbs and 733 cycles. The cyclic loading at smaller loads (F.S. = 3) will use a negligable portion of the fatigue life of Hilti Kwik bolts.

4.0 CONCLUSIONS

The conclusions from the above discussions are as follows:

- F.S. = 3 for non-nuclear conventional structures, with precedant

- All engineering parameters affecting the strength of Hilti Kwik Bolts have been reviewed, and found not to degrade strength. By considering actual minimum concrete strengths, the criteria F.S. is 3.45. By also considering average concrete strengths and bolt embedments, the criteria F.S. is between 3.94 and 4.11.

- Not all Hilti Kwik Bolts are loaded up to their criteria allowable F.S. = 3. A statistical study of over 2200 Train C conduit supports with Hilti Kiwk Bolts showed an average F.S. = 10. By considering average concrete strengths and embedments, and actual loads, likely 100% of the supports in the sample have F.S. over 4.0.

- Statistical evaluations of the required F.S. to maintain nuclear plant safety suggest that at a F.S. = 2, a 1% failure rate for individual Hilti Kwik Bolts can be expected, including workmanship defects. This 1% failure rate leads to a 0.01% failure rate for a collapse of a multiply supported Train C conduit run, and less than one in one million probability of such a failure during the plants lifetime. At a F.S. = 3, the probability of failure goes up by another order of magnitude. Considering that two adjacent Hiltis would have to fail to cause a conduit collapse then the probability of this occuring goes up by an additional one c· two orders of magnitude.

- A series of 9 cyclic load tests performed using Hilti Kwik Bolts typically used in Train C supports, including a workmanship defect, all showed capacity to take 813 full load cycles at a F.S. = 3. No significant slip occurs in these tests, and less than one-sixteenth inch accumulated slip would occur after five OBEs and one SSE.

It is concluded from the above that the Criteria of F.S. = 3 is acceptable and safe for Train C applications.

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Table 1

TESTING CONDITIONS OF HILTI-KWIK BOLTS

Test Sample Number	Testing Frequency: 1 Hertz Alignment Slope: 6°
1-C-0.33H-02	Test was run to 813 cycles. Subsequent testing at a higher load was terminated after 274 cycles due to load system oscillation. Approximately 20 cycles of the subsequent testing data are free of the effects of the system oscillation.
1-C-0.33H-04	Test was run to 813 cycles. Subsequent testing at a higher load was run to 813 cycles.
1-C-0.33H-05	Test was run to 813 cycles. Subsequent testing at the same load, but with an input frequency of 5 Hertz, was run to 813 cycles. Subsequent test at a higher load and an input frequency of 5 Hertz was run to 813 cycles.
1-C-0.50H-01	Test was run to 813 cycles. Subsequent testing at a higher load was run to 813 cycles.
1-C-0.50H-02	Test was run to 813 cycles. Subsequent testing at a higher load was run to 813 cycles.
1-C.0.50H-03	Test was run to 813 cycles. Displacement data was erratic due to test fixture rotation. Subsequent testing at a higher load was run to 813 cycles. Displacement data was again erratic due to LVDT slippage.
1-C.0.80H-01	Test was run to 813 cycles. Subsequent testing at a higher load was run to 813 cycles.
1-C.0.80H-02	Test was run to 813 cycles. Subsequent testing at a higher load was run to 813 cycles.
1-C.0.80H-03	Test was run to 813 cycles. Subsequent testing at a higher load was terminated after 733 cycles when the HILTI bolt fractured.

TABLE 2

HILTI-KWIK BOLT TESTING RESULTS

3/8"--AISI 11L41 BOLTS--2" EMBEDMENT

Test Sample Series	Test Sample Number	Applied Tensile Load Pounds	Resulting Slip (Pull-out Plus Bolt Extension) After 1 Cycle (10^{-3} inches)	10 Cycles	25 Cycles	100 Cycles	813 Cycles
1-C-0.33	H-02	1095	5.8	10.7	13.5	20.0	23.0
		1950	31.1	41.7	53.5	Test Dropped	
	H-04	1100	2.8	3.9	4.8	6.1	8.5
		2400	47.3	77.7	90.4	100.0	160.0
	H-05	1050	4.3	6.5	7.4	8.6	10.9
		1050	--	Frequency		5 Hertz	
		2400	31.2	71.0	99.6	126.5	170.0
1-C-0.50	H-01	1530	7.9	12.8	16.4	24.4	35.6
		2410	40.0	54.3	63.3	77.9	106.0
	H-02	1560	6.0	11.4	15.3	21.3	30.2
		2470	38.2	53.2	63.7	80.0	99.7
	H-03	1530	9.3	26.9	35.2	51.2	LVDT SLIPPAGE
		2600	16.1	37.9	49.7	73.4	LVDT SLIPPAGE
1-C-0.80	H-01	2520	42.9	67.6	78.3	88.2	106.7
		3090	113.3	114.3	116.7	124.7	165.2
	H-02	2520	27.2	65.6	83.7	111.3	131.9
		3040	132.4	136.2	138.7	150.3	161.8
	H-03	2510	50.3	71.4	79.3	90.9	115.7
		3060	109.9	110.4	111.9	117.5	BOLT FAILURE

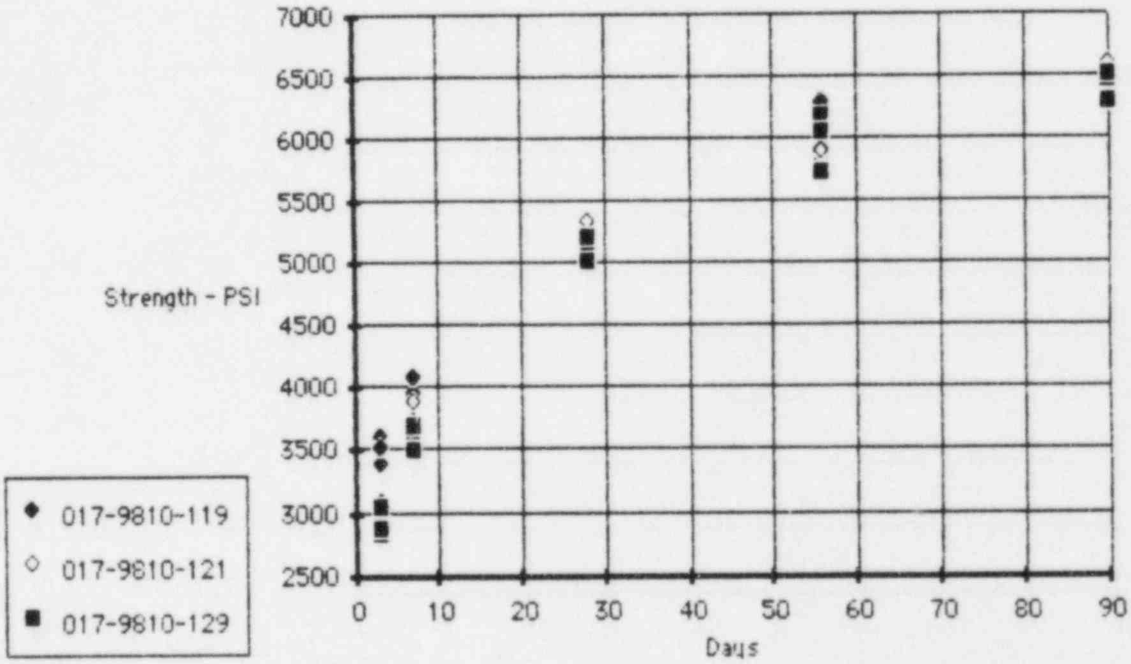

Figure 1 TEST FIXTURE CONCRETE STRENGTH OVER TIME

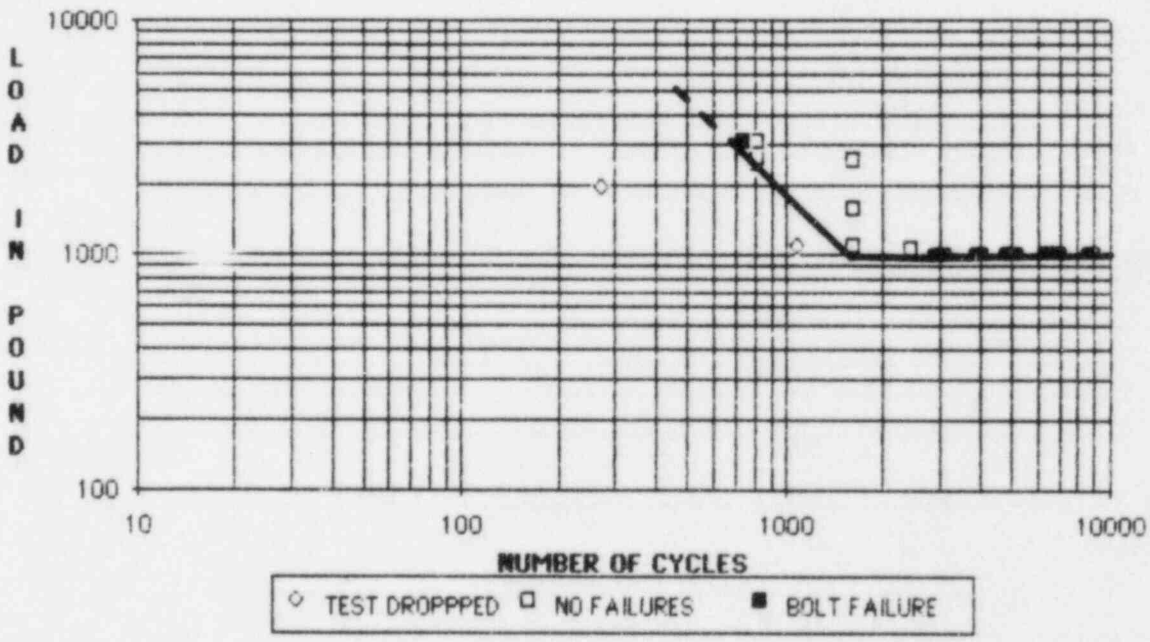

Figure 2 FATIGUE CURVE FOR 0.375 INCH HILTI KWIK BOLT (2-INCH EMBEDMENT) WITH MAXIMUM OUT-OF-PLUMBNESS--tension loads only

APPENDIX D

TARGET EVALUATIONS

APPENDIX D

DAMAGE STUDY

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PART I

TARGET ANALYSIS

INTRODUCTION

This part of the report describes studies conducted to access the behavior of structural systems and components as it is impacted by conduits during a postulated failure of conduit supports. The studies investigated four types of targets.

(1) Safety and Non-Safety Class Piping Systems

(2) Conduits

(3) HVAC Ducts

(4) Cable Trays and Cable Tray Supports

As will be briefly explained, it is necessary and prudent to allow the target to absorb the energy of the missile by plastic strain energy. The amount of plastically allowed for the target to undergo while absorbing the missile energy was held to very small values defined as the minimum of (1) 10% of the strain at ultimate load or (2) 10 times the strain at yield. Observing these two limits ensure ample safety for both hard and malleable materials. For piping systems energy limited to a bending moment of 70% of the ultimate moment is used.

The structure, having a function to perform, should not be deformed by the impact so as to impair its function. Accordingly, functional capabilities of structures and systems will be assured by limiting distortions. Such limitations are system dependent and will be stated as each system is addressed in this report.

The approaches taken in investigating various targets are distinctly different, brought about by the distinct and unique behavior of the various structural systems especially during non-linear behavior. The different approaches and computer codes are presented as each system is addressed in this report.

BEHAVIOR OF STRUCTURES UNDER IMPACTIVE LOADS

When a structural is impacted a certain amount of energy is transferred from the missile to the structure (the target) and stored as strain energy in said target. If the strain energy is stored while the target is responding elastically, the target will rid itself of this energy by partially giving it back to the missile and partially dissipating it through damping as it vibrates after impact. This energy stored during the elastic response of the target is known as "Resilience" and it represents an insignificant amount of energy compared to the total energy the structure is able to safely absorb.

Typically missiles carry large amounts of energy. On impact the target safely absorbs a substantial portion of this energy through plastic deformations. The energy absorbed by the target during plastic response is permanently retained by the target and removed from the missiles.

Figure 1, shows a typical load - deformation curve for a structure. The total area under the curve up to failure at maximum deformation is the total energy needed to fail the structure, or conversely, the energy absorbed by the structure up to failure. This total energy is known as Toughness. Toughness is typically 300 to over 1000 times the resilience. If the target after impact is not expected to carry any loads, then practically all the toughness may be used to resist impact. Such a practice is common in design of barricades and shields. However, if impact is incidental to the actual function of the structure, then a healthy margin of safety should be observed to avoid approaching the ultimate load carrying capacity of the target.

In this study, load-deformation diagrams up to and past the ultimate load is generated for each structural system. These diagrams are used to compute the strain energy absorbed by the structure at any desired state of strain or load. For structures other than piping, a strain limit defined as the least of 10% of the strain at ultimate load or 10 times this at yield is used. The area under the load-deformation diagrams is then computed up to the strain limit just stated. For piping systems however, energy absorption is restricted to that up to 70% of the ultimate moment.

It is however acknowledged that the target structure was subjected to some loads prior to impact, and as such, the allowable strain energy computed above is not all available for absorbing the impact energy, but it must be reduced by the amount of energy already existing in the structure prior to impact. The latter being unknown, a conservative assumption is made that the structure prior to impact is stressed all the way to its yield stress or to the highest design stress (or strain) the applicable code will allow, whichever is greater. The energy corresponding to this stress (or strain) is then deducted from the allowable strain energy of the structure the remainder of which will be available for impact.

IMPACT CAPACITY OF PIPING SYSTEMS

I. INTRODUCTION

The target piping systems may be Class I, II or III. However, in the analysis that follows Class I is assumed. This assumption renders the analysis conservative when applied to Class II and III piping.

For each pipe size considered, a range of spans are investigated, the largest being the B31.1 recommended span for water filled pipes, the shortest being ½ that recommend. Five pipe sizes are investigated; 4", 6", 8", 10" and 12", all of which the standard schedule is used.

Limitations

This study is limited to impacting a piping systems with the following constraints.

(1) The impacted span is supported no closer then one-half that required by ANSI B31.1 code. Spans of longer length is permitted with no bound.

(2) The impacted span is that of a straight pipe. (no elbows, or brace).

(3) The span contains no valves or equipment.

(4) The impact occurs in the middle 2/3 of the span (1/6 the span away from each support).

The above limitations can be relaxed or eliminated by further and specialized analysis. However, currently the limitations are not expected to cause undue hardship on the walkdown effort.

Acceptable Criteria

(1) Stress Requirement
Maximum bending moment produced in the piping on impact shall not exceed 70% of the ultimate bending moment capacity of said pipe. (ASME, Section III, Div. 1, Appendix F)

(2) Functional Capability
The distortion of piping cross section shall not cause a reduction of the net flow area more than 5%. However, in this analysis the reduction in the net flow area was found to be less than 4% for all cases.

II. METHODOLOGY

A non-linear finite element analysis was used in which both geometric and material non-linearity was accounted for.

Impact Simulation

A straight rigid bar on the top of the pipe at the center of the span is pressed down on the pipe simulating the impact load.

The pipe is loaded through this rigid bar via enforced displacements which are gradually increased until failure takes place. (Fig. 2)

Model

A simply supported span with the length equal with one-half of that prescribed by ANSI B31.1 code is considered, followed by an analysis for the full span.

The center part of the span is modelled with the shell elements for a length equal with 3 diameters of the pipe in each side, for the remaining length of the pipe span beam elements are used to the supports.

The MSC-NASTRAN Version 63 Code, Solution 66 is implemented in performing the elasto-plastic large deformation analysis.

Boundary Conditions:

For the straight pipe span the following two planes of symmetry are identified.

(1) 0^{o}-180^{o} plane of the cross section containing the centerline of the piping.

2. Vertical plane containing the cross section perpendicular to the centerline of the piping at the center of the span.

Therefore, both conditions of symmetry are utilized to reduce the model to a 1/4 of its original size.

The end of the pipe is simply supported in the two translational directions vertical and lateral.

The same symmetrical boundary conditions are also applied to the rigid bar on the top of the pipe. For the beam elements only the second symmetrical condition applies.

Detailed Description of the Model

Thirteen (13) shell elements are used to describe the 180^{o} segment of the pipe cross section in the circumferential direction. The first two shell elements at the top are smaller to assure a better description of behavior at the top. Along the length of the piping there are 10 subdivisions to account for the 3 diameter of pipe length modelled with shell elements. The first seven of subdivisions are smaller and account for a length of 1.5 diameter of the pipe the remaining 3 divisions account for the remaining 1.5 diameter of the pipe. Therefore, a total of 130 (13 x 10) shell elements, connecting 154 (14 x 11) grid points are used to describe the part of the pipe modelled by shell elements.

The remaining part of the half span is modelled with beam elements. Rigid body elements are employed to connect the grid points of the last segment of the shell elements to the first grid point of the first beam elements.

The detailed model is shown in Fig. 3. To cut down on the computer cost the nonlinear material properties were assigned to the shell elements describing the first length of 1.5 diameter from the center of the pipe. All other shell or beam elements were assigned elastic material properties.

The rigid bar on the top of the pipe is given a very small width of about ½" to approximatively simulate the point loading. The bar is modelled lengthwise with a row of shell elements (5 or 6 elements depending on the size of pipe) and is made exceedingly thick in comparison with the pipe wall thickness. The bar is connected to the pipe with gap type elements to allow undirectional loading. The grid points of the bar elements are connected to the grid points of the top of the pipe with 4 or 5 gap elements along cross section of pipe depending on the size of the pipe. There are 8 or 10 gap elements connecting 16 or 20 grid points, oriented vertically, perpendicular to the centerline of the pipe in the direction of application of the displacement.

Material Properties

A stress-strain curve of elasto-plastic strain hardening properties is used to describe the shell elements material properties for the length equal to 1.5 diameter of pipe. The remaining shell and beam element have elastic properties only. The material considered is Type-304 stainless steel and the stress-strain curve used is that obtained by testing and in the technical paper "Plastic Deformation of Piping due to Pipe Whip Loading" by T. L. Gerber[4], and presented in Fig. 4.

To be able to make a fair comparison between different pipe sizes and length the same stress-strain curve was used throughout the analysis.

For this material the following parameters were used.

$E = 30. \times 10^6$ psi

$Sy = 27{,}000$ psi

$Su = 78{,}500$ psi

$\varepsilon_u = 0.25$

Yield Criterion:

The von-misses yield criterion is used to represent the state of the stresses within each element, the equivalent stress of which is given by:

$$S_{equ} = 1/2 \sqrt{(S_1-S_2)^2 + (S_2-S_3)^2 + (S_3-S_1)^2}$$

where

S_1, S_2, S_3 are the three principal stresses

Yield and failure of the material is postulated when the equivalent stress as computed by von-misses stress criteria respectively exceeds the yield or ultimate stress as defined in the stress-strain curve used.

Large Deformation

A large deformation feature is utilized in order to account for the effect of the ovalization of the pipe cross section and the local deformation due to the loading conditions on the moment carrying capacity of the pipe. The cross sectional ovalization and local deformations at the top of pipe reduces the pipe section modulus, i.e. reduces the value of moment/max stress.

The moment may still be increasing due to the plastic flow which allows greater portions of the cross section to be subject to higher stresses.

Cases Analyzed

The analysis was done for different pipe sizes all of which are standard schedule. Each pipe size is analyzed for two span lengths, that recommended by ANSI B-31.1 for water filled pipes, and half the recommended span. The latter is done to emphasize the ill-effects of local shell deformations.

The pipe sizes and span length considered are:

(1) 4 inch diameter pipe sch. 40 (std) L= 7'-0" & 14'-0"

(2) 6 inch diameter pipe sch. 40 (std) L=8'-6" & 17'-0"

(3) 8 inch diameter pipe sch. 40 (std) L=9'-6" & 19'-0"

(4) 10 inch diameter pipe sch. 40 (std) L=10' -0" & 20'-0"

(5) 12 inch diameter pipe sch. 40 sch. 40 (std) L=11'-6" & 23'-0"

The force-displacement (P-Δ) diagram up to failure was developed for each size and span.

The area under the curve corresponding to the load (P) equal to 70% of the ultimate moment (Mu) was computed, representing the total energy available for the pipe impact.

The 70% factor used above is in accordance with ASME code, Section III, Div. 1, Appendix F. From this total energy the energy existing in the pipe prior to impact such as pressure, seismic, thermal, etc. is subtracted. This energy being unknown at time of impact, a conservative assumption is made whereby it is assumed that the piping system is stressed and strained to the maximum code allowable for design. The energy that exist at such state will be assumed to exist prior to impact and thus deducted from the total energy available at

0.7 Mu, the remainder of which will be available for absorbing impact. For Class 1 piping a maximum of 3Sm is permitted by ASME Section III NB code. This will actually strain the pipe to twice the elastic strain. Consequently, an energy equal to that at twice the elastic strain is assumed to exist prior to impact. This assumption is adopted also for class 2 and 3 piping though it is conservative.

Figures 5 thru 9 show the P-Δ diagram for the piping sizes 4", 6", 8", 10" and 12", respectively. It is apparent that the ultimate load of the half spans are decisively larger than the full spans, yet toughness (area under the curve) of the half spans are much less than that of the full span. On the other hand, while the ultimate load of the full span is less than that of the half span, their toughness is quite high and thus the ability to absorb impact energy is quite large. The finding, although expected deserves an explanation. Figures 10 and 11 show the effect of different span length on the deformation of the mid-span pipe cross-section, displayed at M ultimate (Mu) and 0.7 Mu respectively. On inspection of Figure 10, the following is concluded; for short spans, the pipe does not act as a flexural beam in any significant way, and accordingly, the bending stresses in the longitudinal direction (axial direction) are small and will not cause instability as the cross-section deforms out of a true circle. The failure scenario would be that of local crushing and large sectional distortion. In other-words, short spans resist the imposed loads through local deformations by which archs and catenary are developed to carry the load. The weakness of the axial stress (due to weak beam action) will not accelerate instability of the deformed section allowing the imposed loads to reach fairly high values. However, the excessively deformed section will fall prey to instability even at small bending levels, brought about by the large eccentricity of the deformed section at load point and the bending stresses in the outer fibers of the full section just outside the local area of the load application. This instability will occur rapidly with little deformations.

For long spans on the otherhand, the flexural stiffness is much less than the stiffness of the cross-section. Consequently, a load will cause the pipe to displace as a beam with little deformation in the cross-section. The failure scenario is that of plastic hinge development where the axial stresses due to bending is quite high and beam deflections will give rise to strains that will grow until the maximum strain of the material is reached at which point failure will occur. Figures 12 and 13 shows the effect of spans on moment-rotation and load-deflection diagrams respectively. Evident from these load-deflection diagrams is that short spans carry higher loads compared to long spans, however, failure is rapid with little deformation, while the long span shows substantial deformation prior to ultimate load. The moment-rotation diagram on the other hand shows the ability of the long span to both withstand moment and undergo deformation much larger than short span. Accordingly, a conclusion can be made that the longer the span, the better the pipe is for absorption of impact.

Figure 14 through 18 show the Weight-Height relationship of allowable missiles for the respective pipe sizes studied, presented for two span lengths; that recommended to ANSI B31.1 for water filled pipes (shorter than air filled pipes), and one-half that span. It would be conservative to use these curves for spans longer than the ones stated on the grabs, the converse however, is not true.

FIGURE 1
TYPICAL LOAD-DEFORMATION CURVE

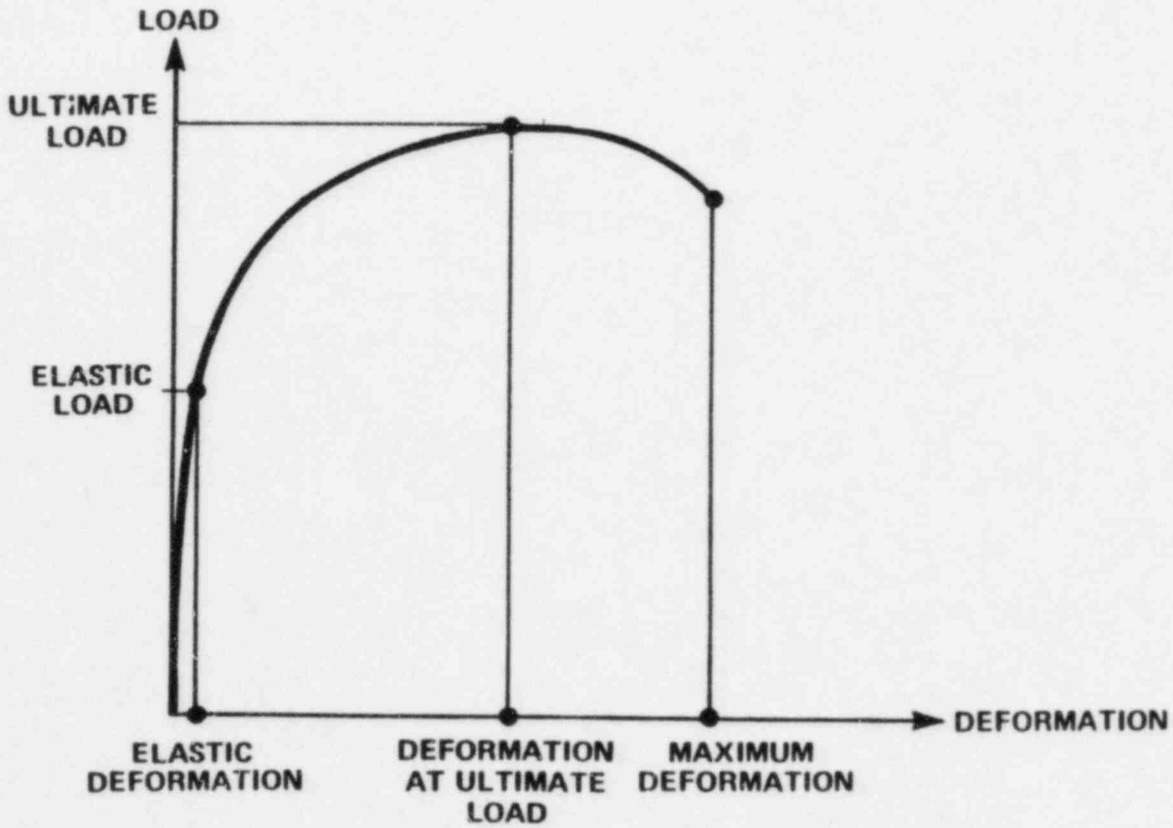
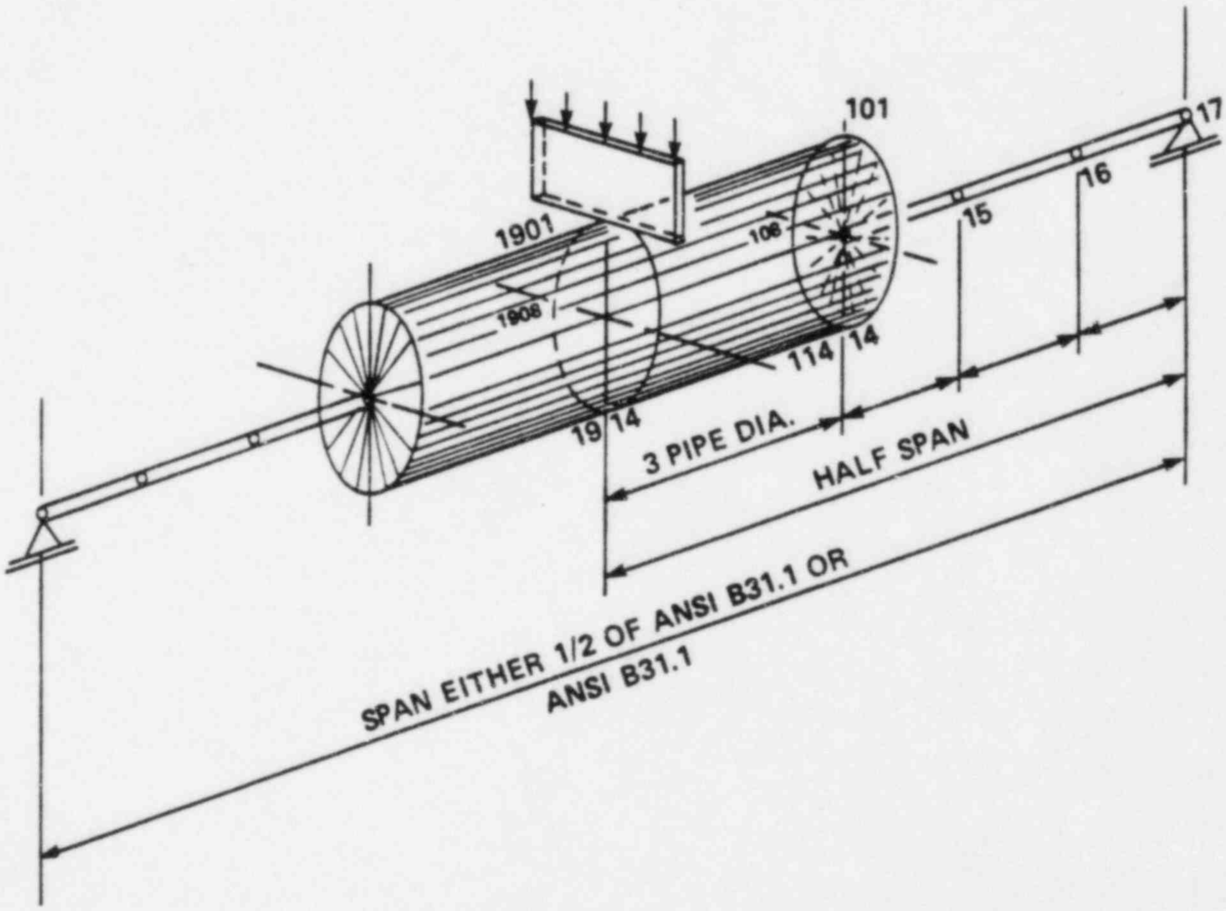


FIGURE 2
PIPE MODEL USED FOR IMPACT SIMULATION



FINITE ELEMENT: • MODEL ¼ MODEL
• CODE MSC NASTRAN, VERSION 63
SOL. 66
NONLINEAR GEOM AND
MATERIAL

FIGURE 3
¼ MODEL FOR CONCENTRATED LOAD
(Typical)

PIPE, STAINLESS STEEL, SCH 40

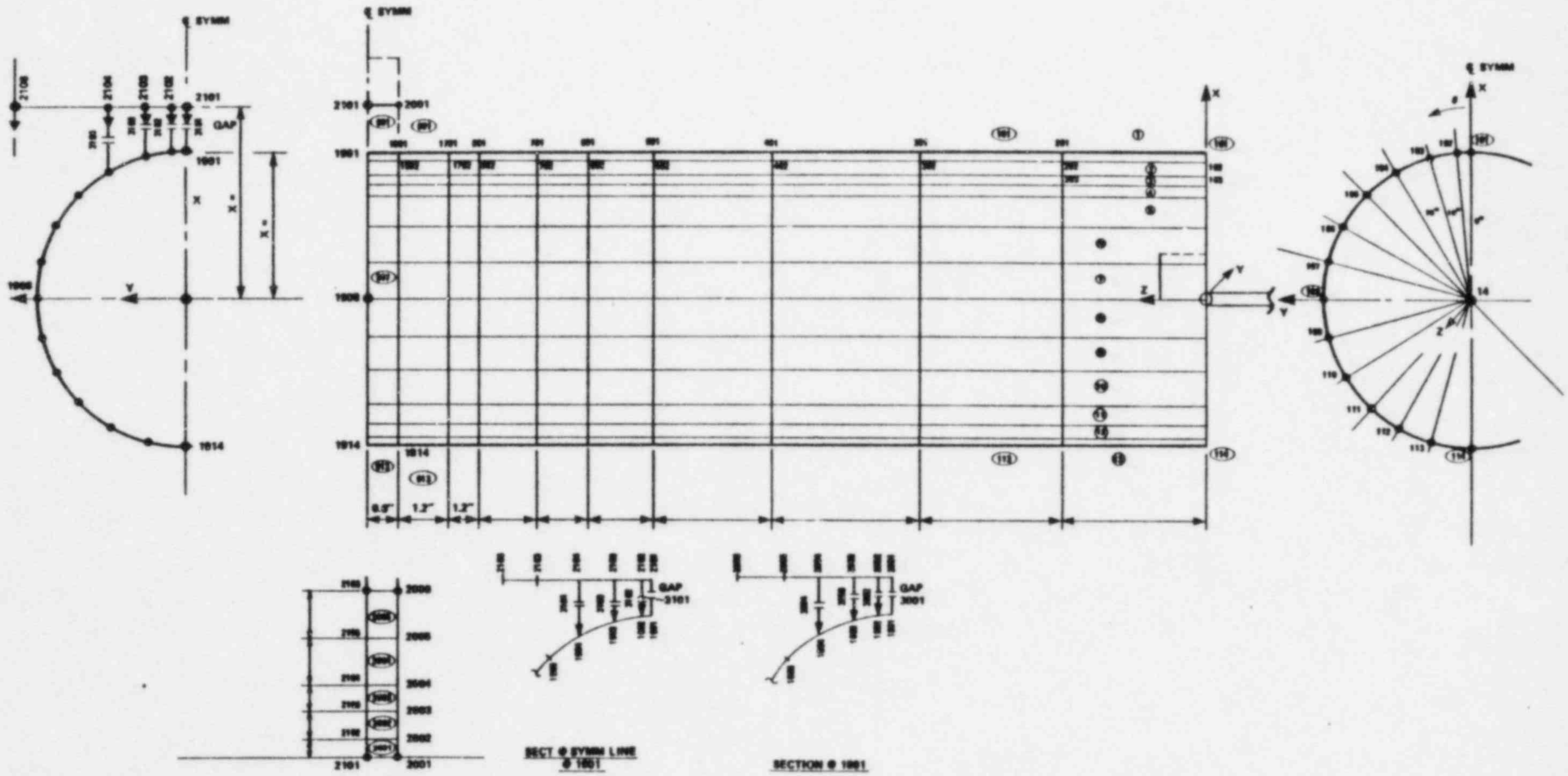


FIGURE 4
STAINLESS STEEL PIPES STRESS-STRAIN PROPERTIES

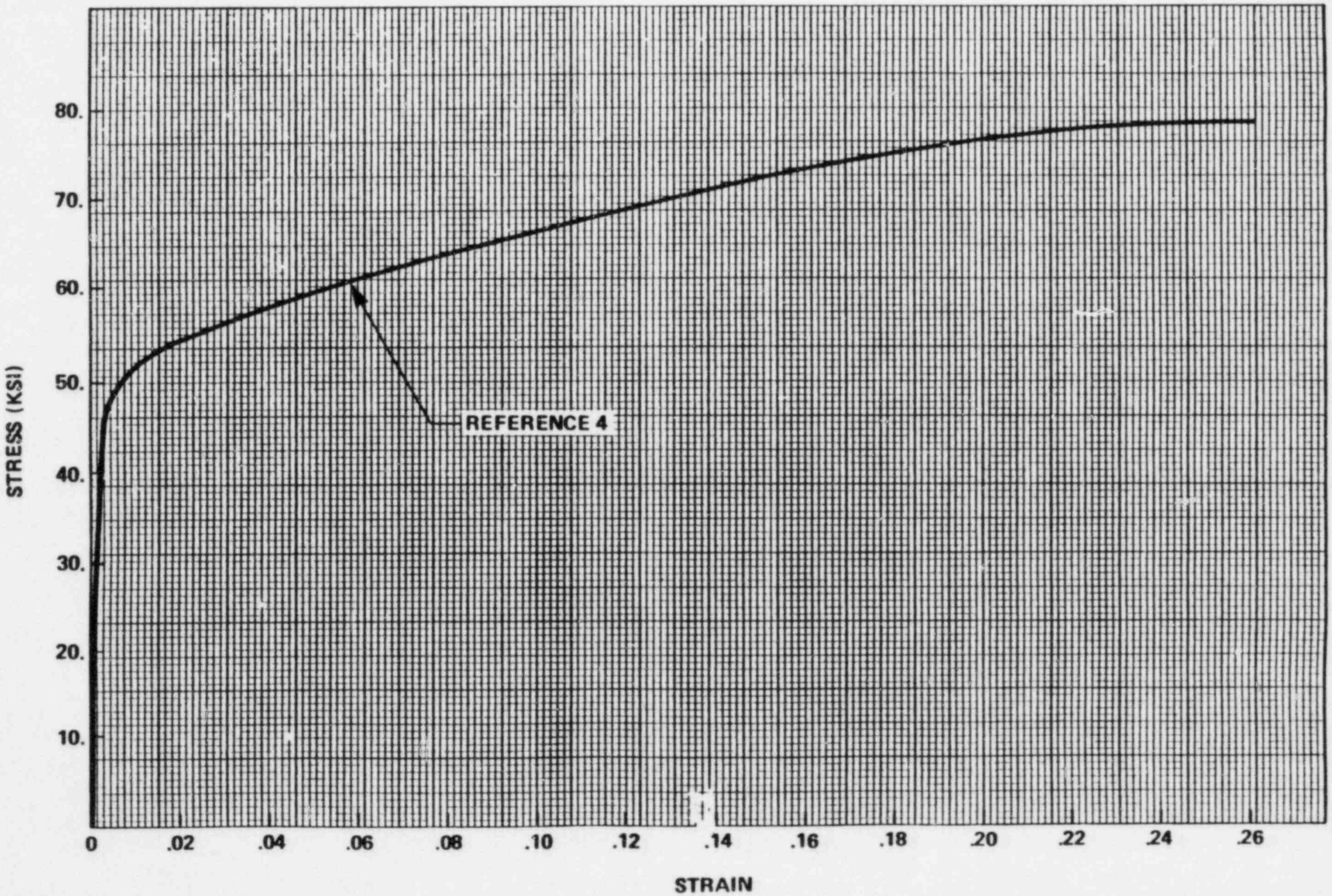


FIGURE 5

P - Δ DIAGRAM FOR 4" DIAMETER PIPE

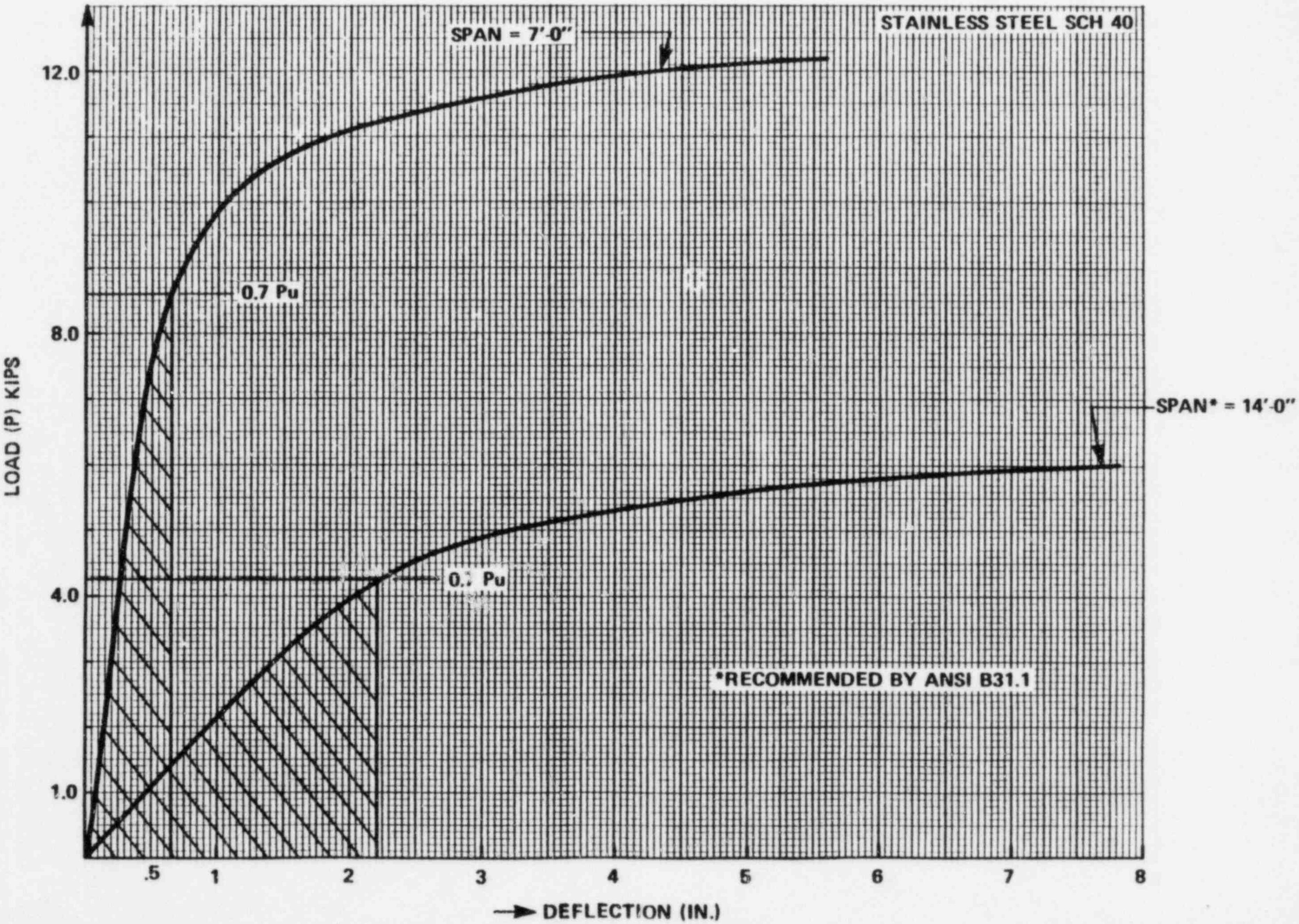


FIGURE 6

P-△ DIAGRAM FOR 6" DIA. PIPE

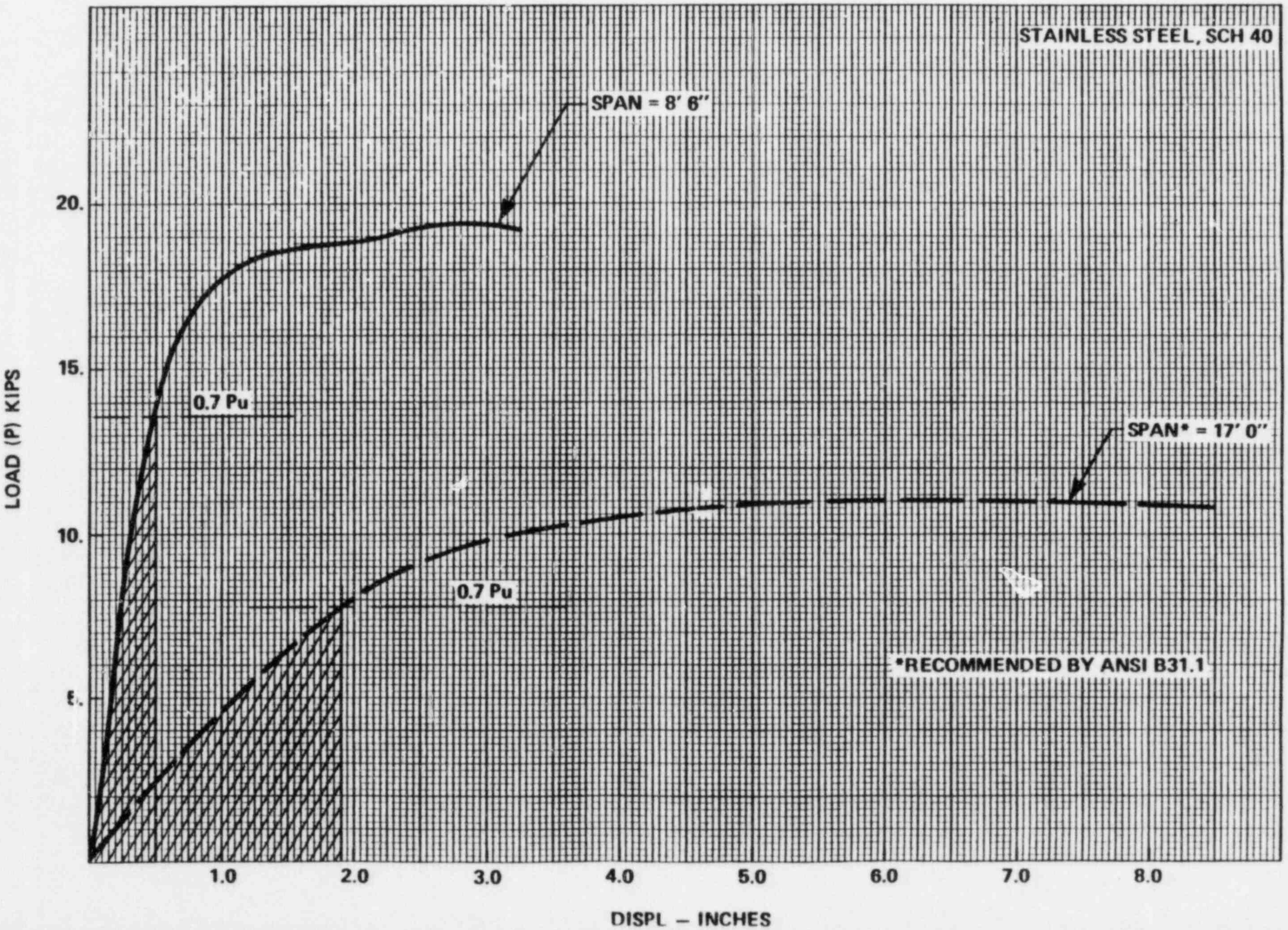


FIGURE 7

P -△ DIAGRAM FOR 8" - DIAMETER PIPE

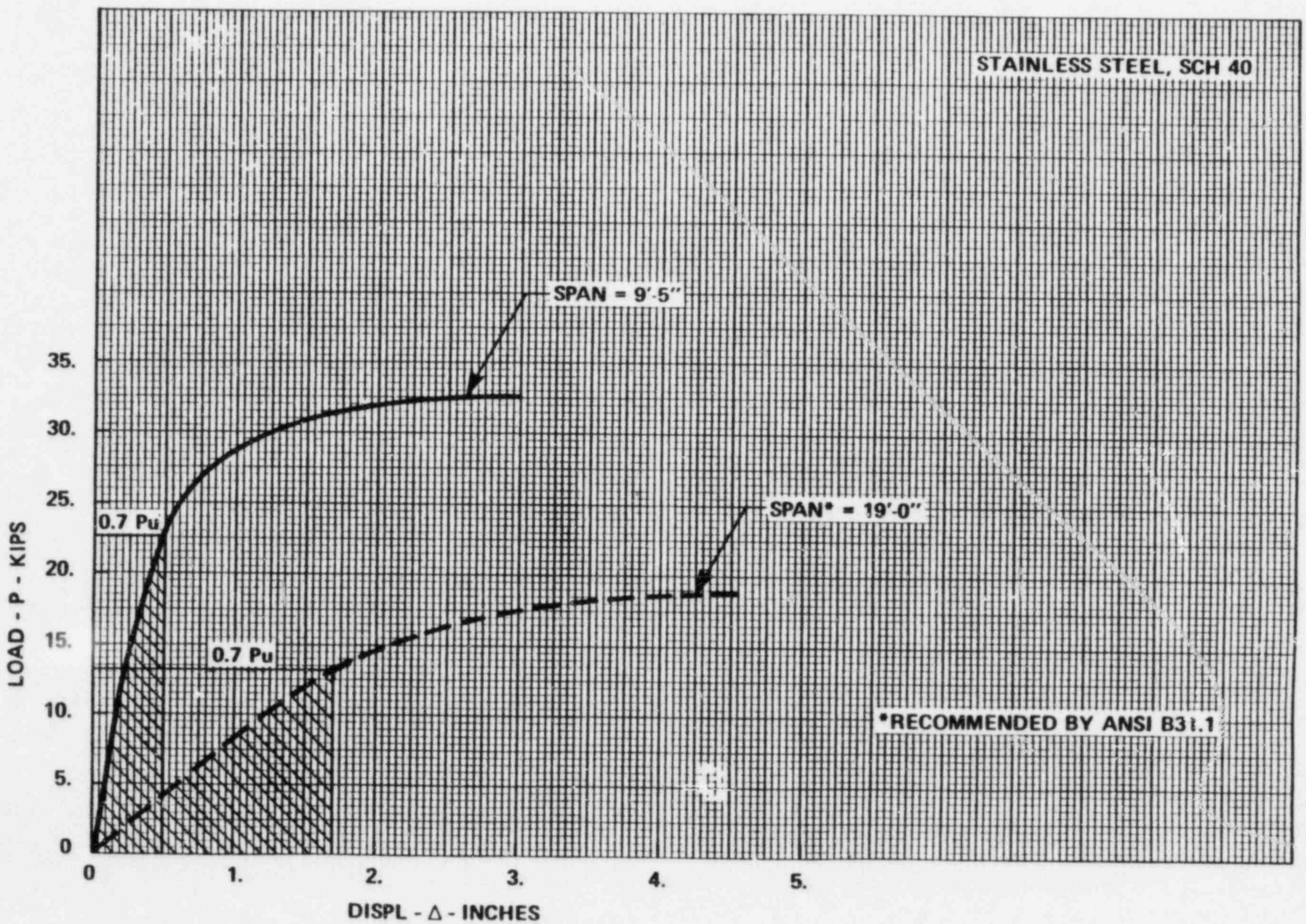


FIGURE 8

P - △ DIAGRAM FOR 10" DIA PIPE

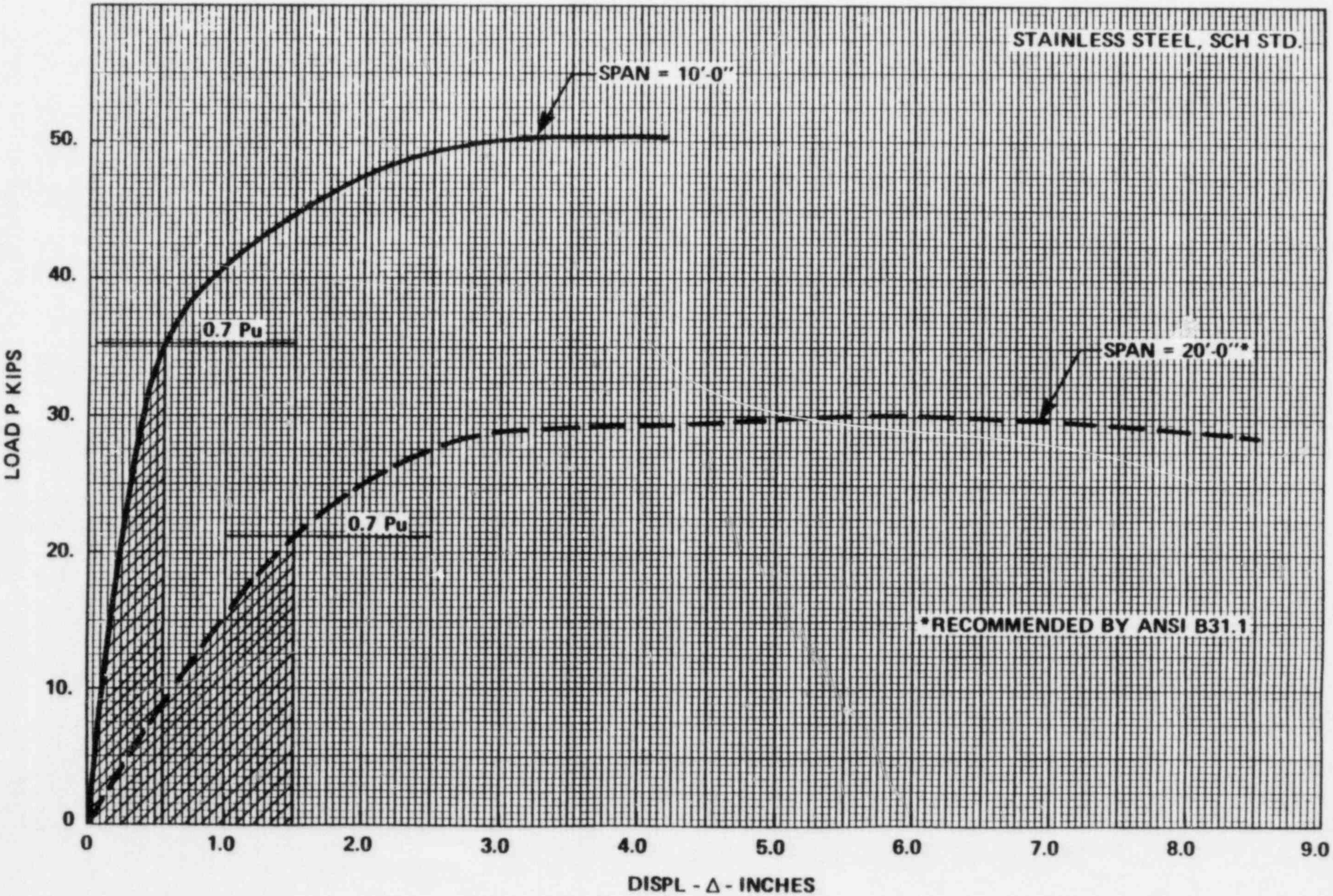
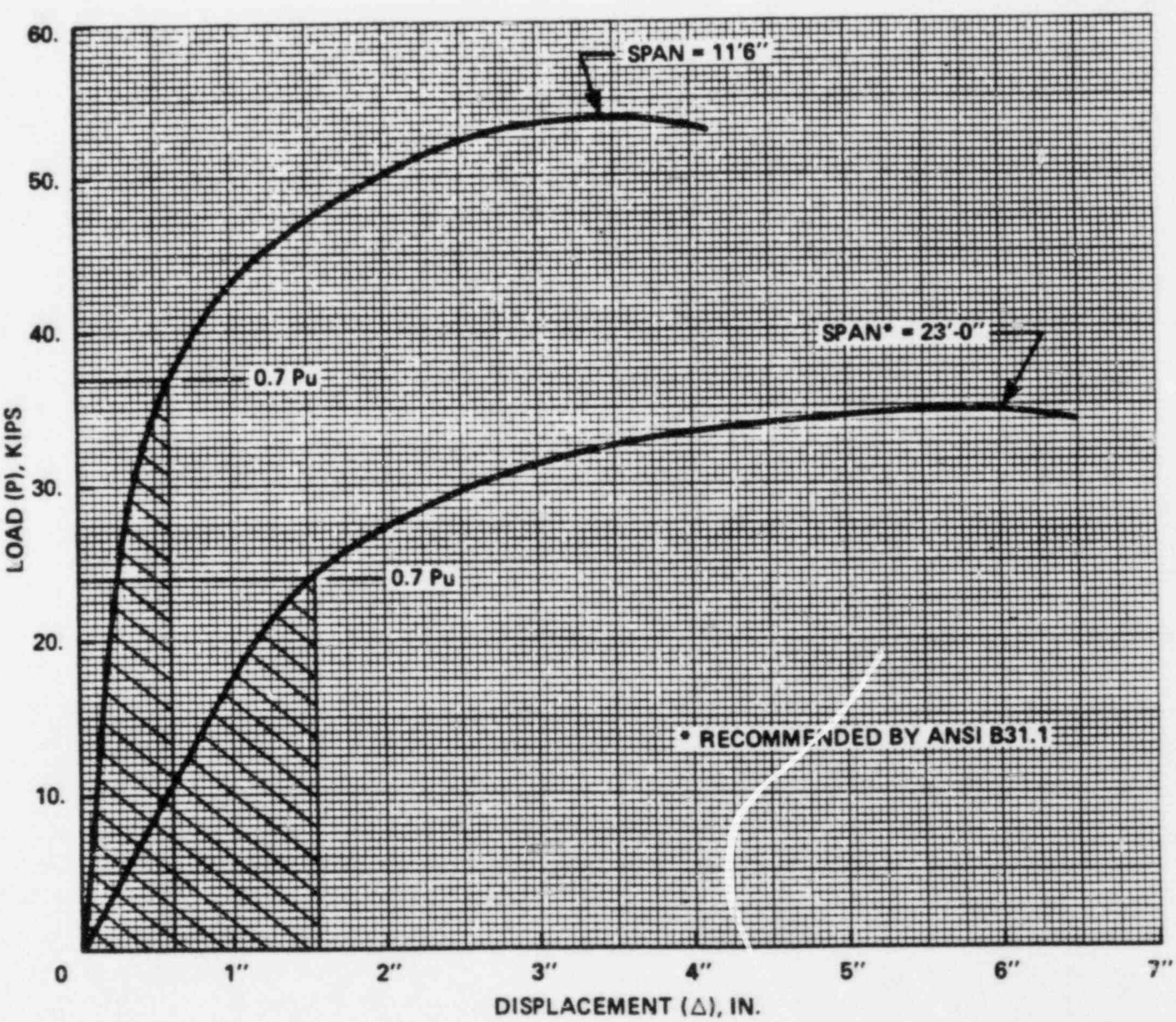


FIGURE 9
P-Δ DIAGRAM FOR 12" DIAMETER PIPE



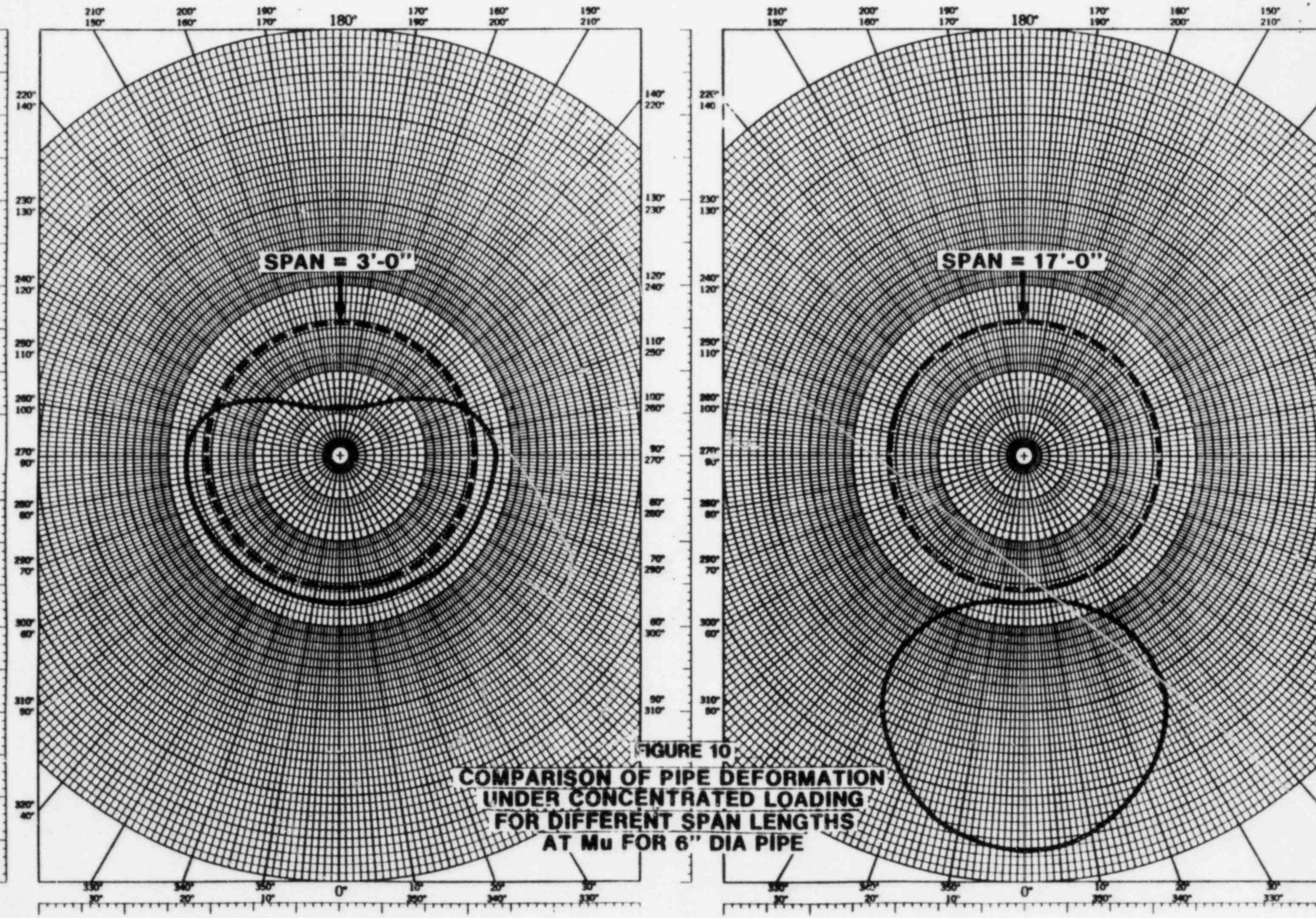


FIGURE 10
COMPARISON OF PIPE DEFORMATION
UNDER CONCENTRATED LOADING
FOR DIFFERENT SPAN LENGTHS
AT Mu FOR 6" DIA PIPE

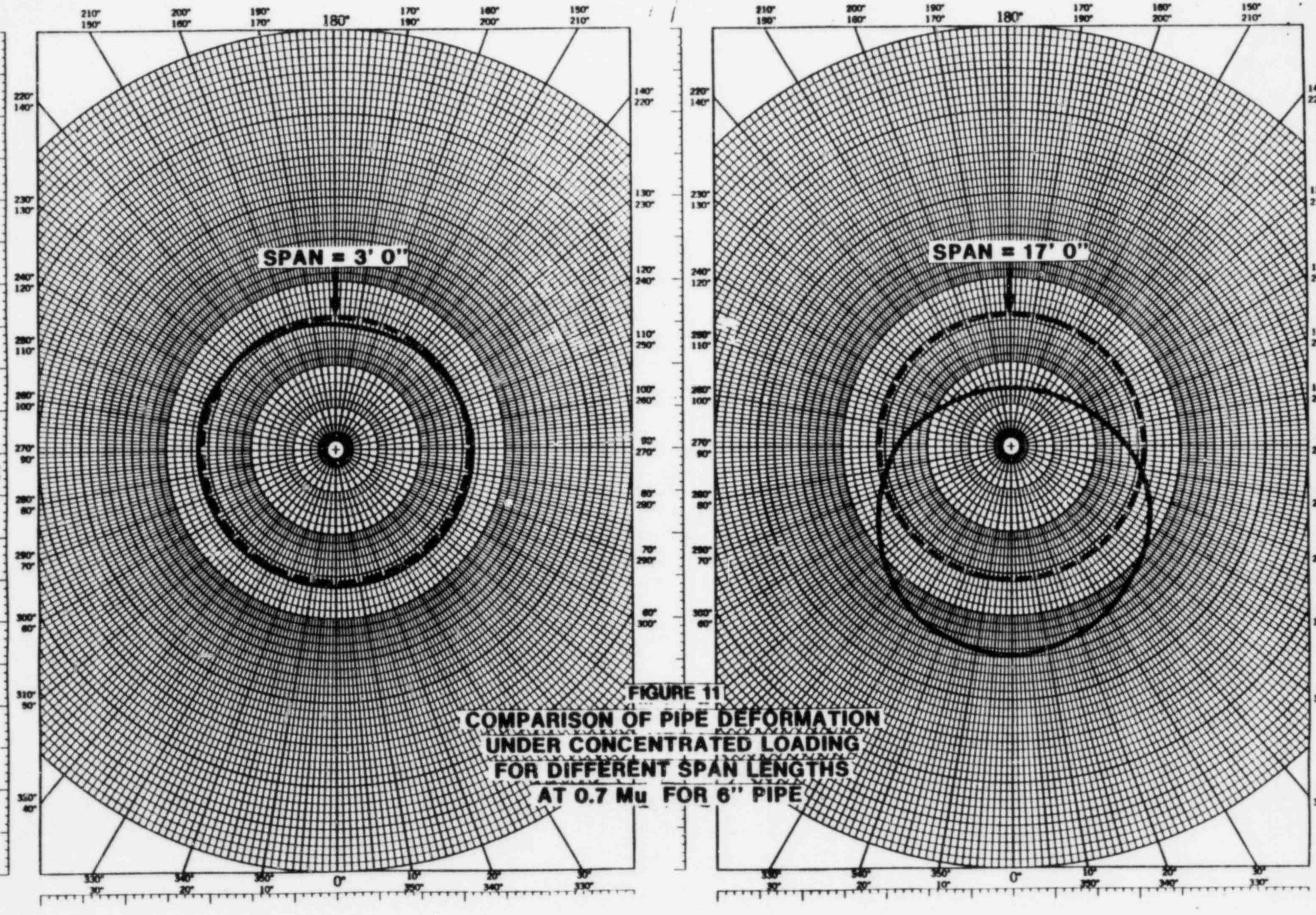


FIGURE 11
COMPARISON OF PIPE DEFORMATION
UNDER CONCENTRATED LOADING
FOR DIFFERENT SPAN LENGTHS
AT 0.7 Mu FOR 6" PIPE

FIGURE 12

COMPARISON OF MOMENT - ROTATION DIAGRAM FOR DIFFERENT SPANS

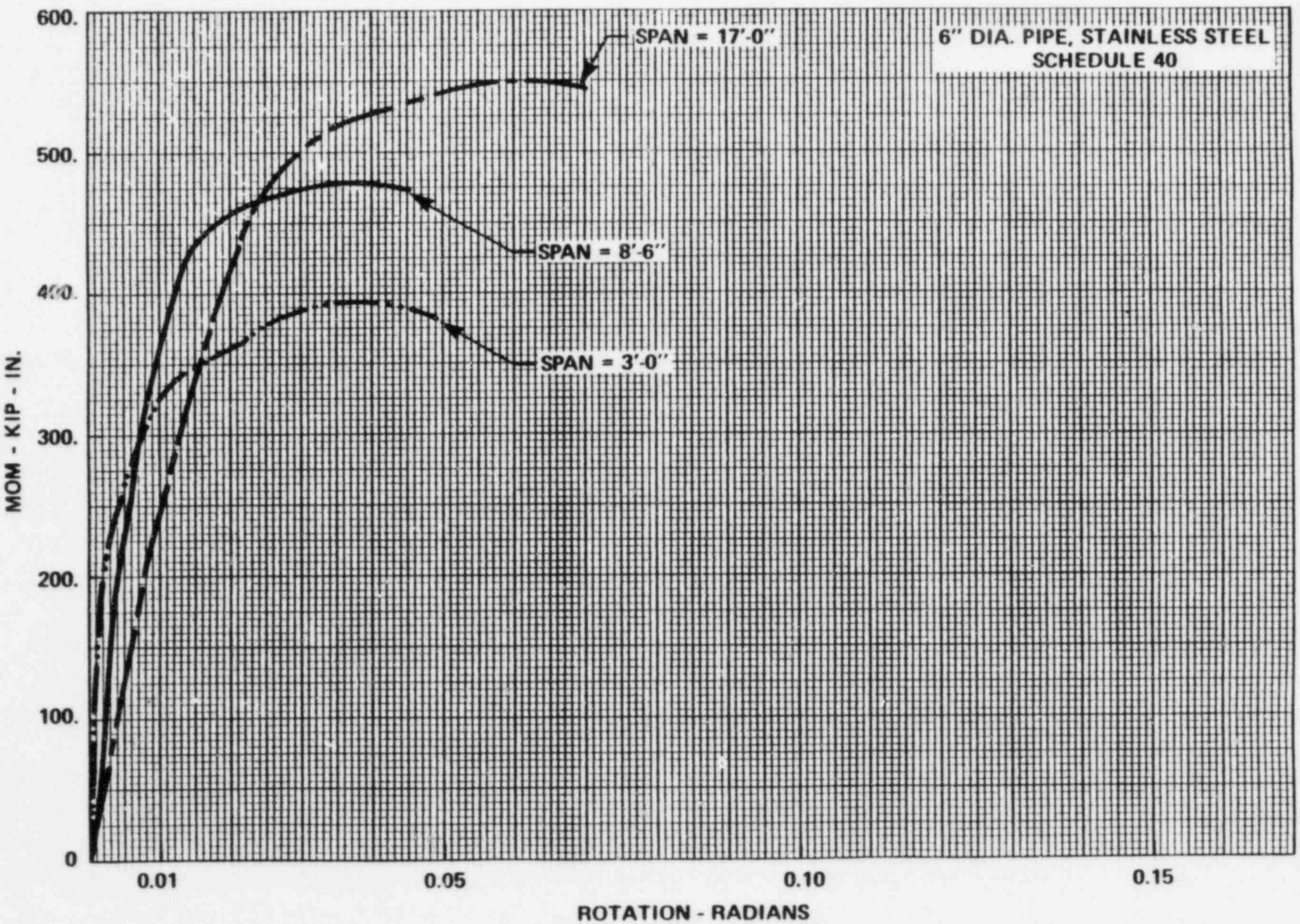


FIGURE 15

COMPARISON OF LOAD - DISPLACEMENT DIAGRAM FOR DIFFERENT SPANS

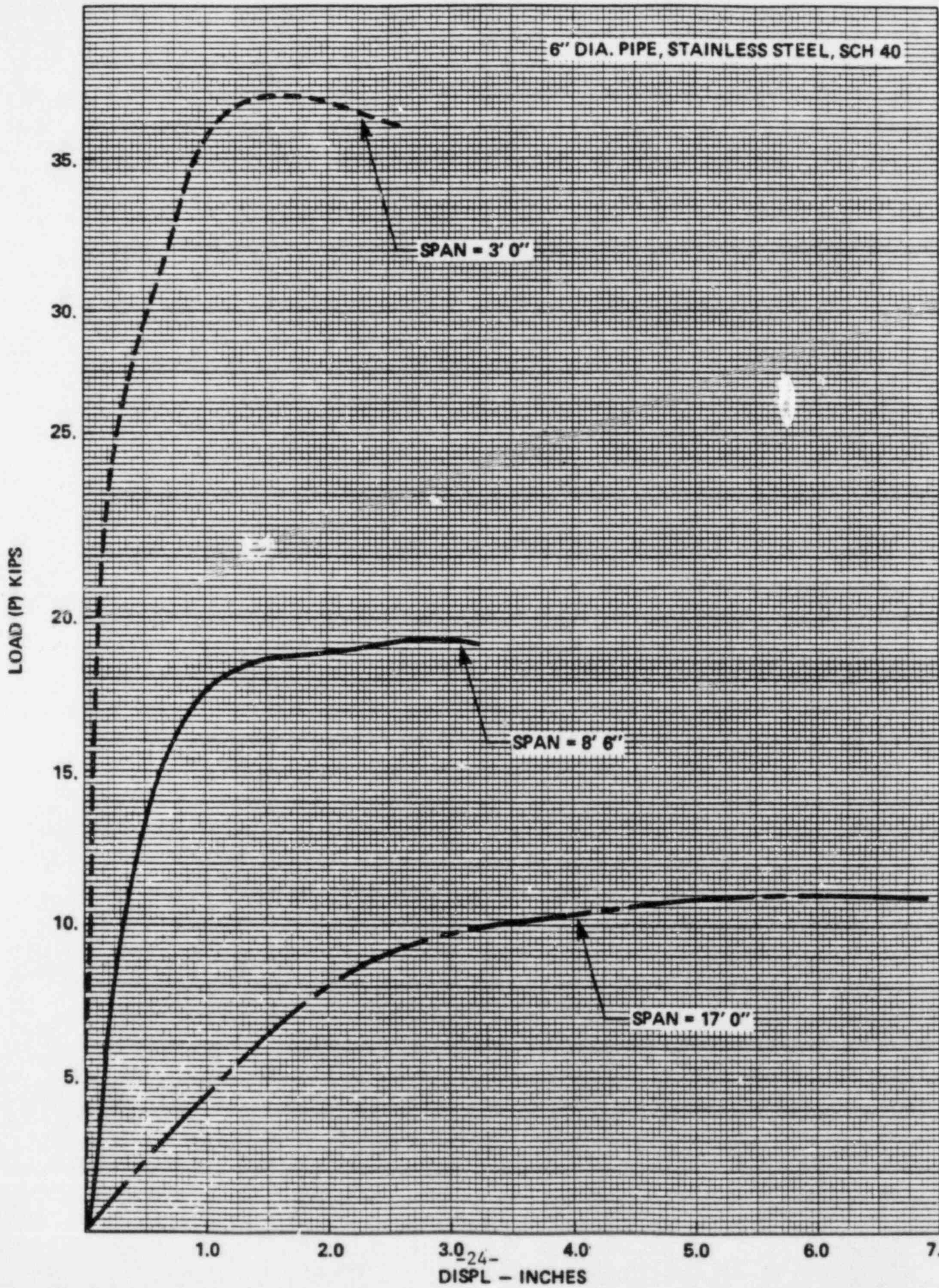
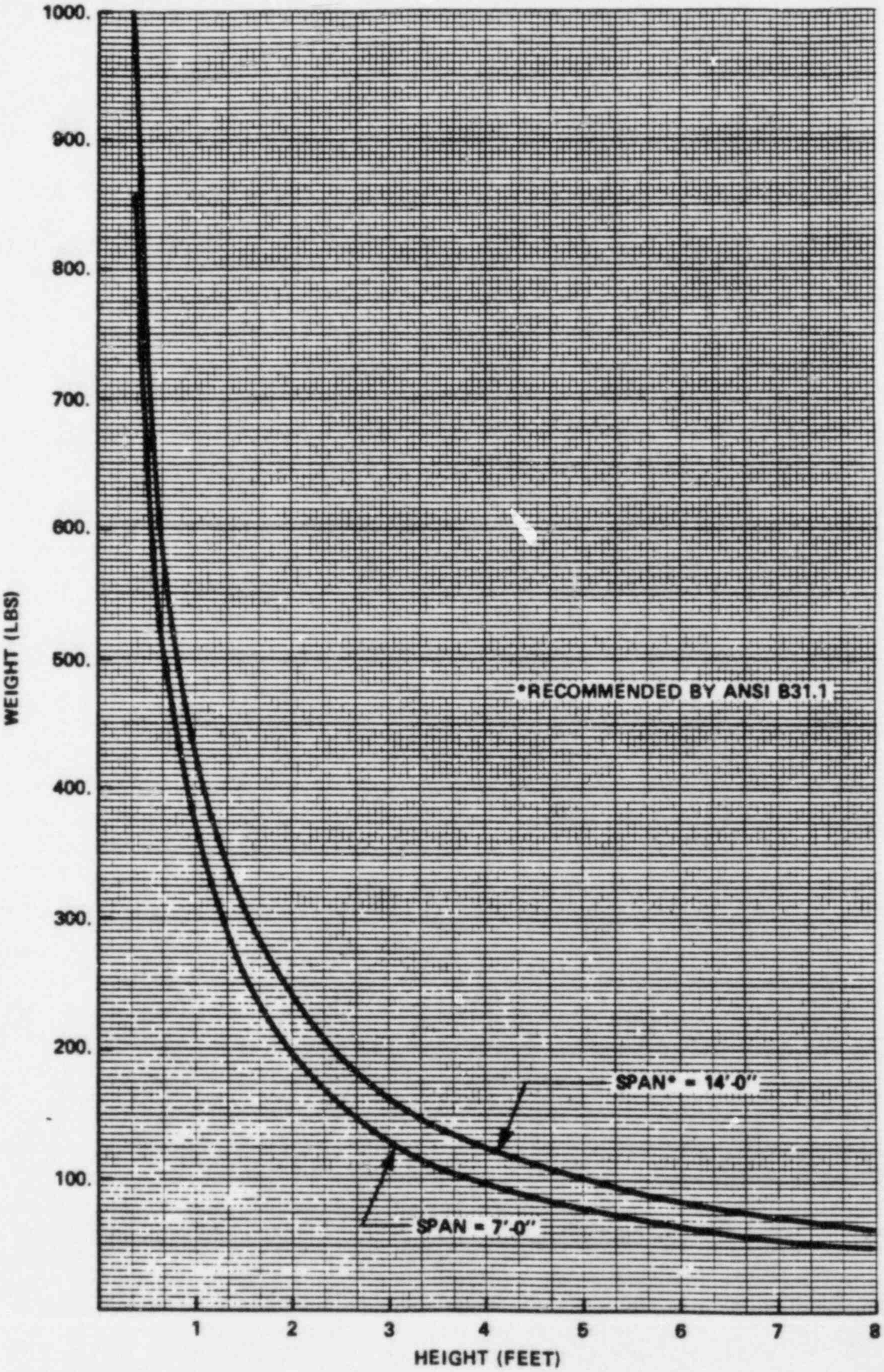
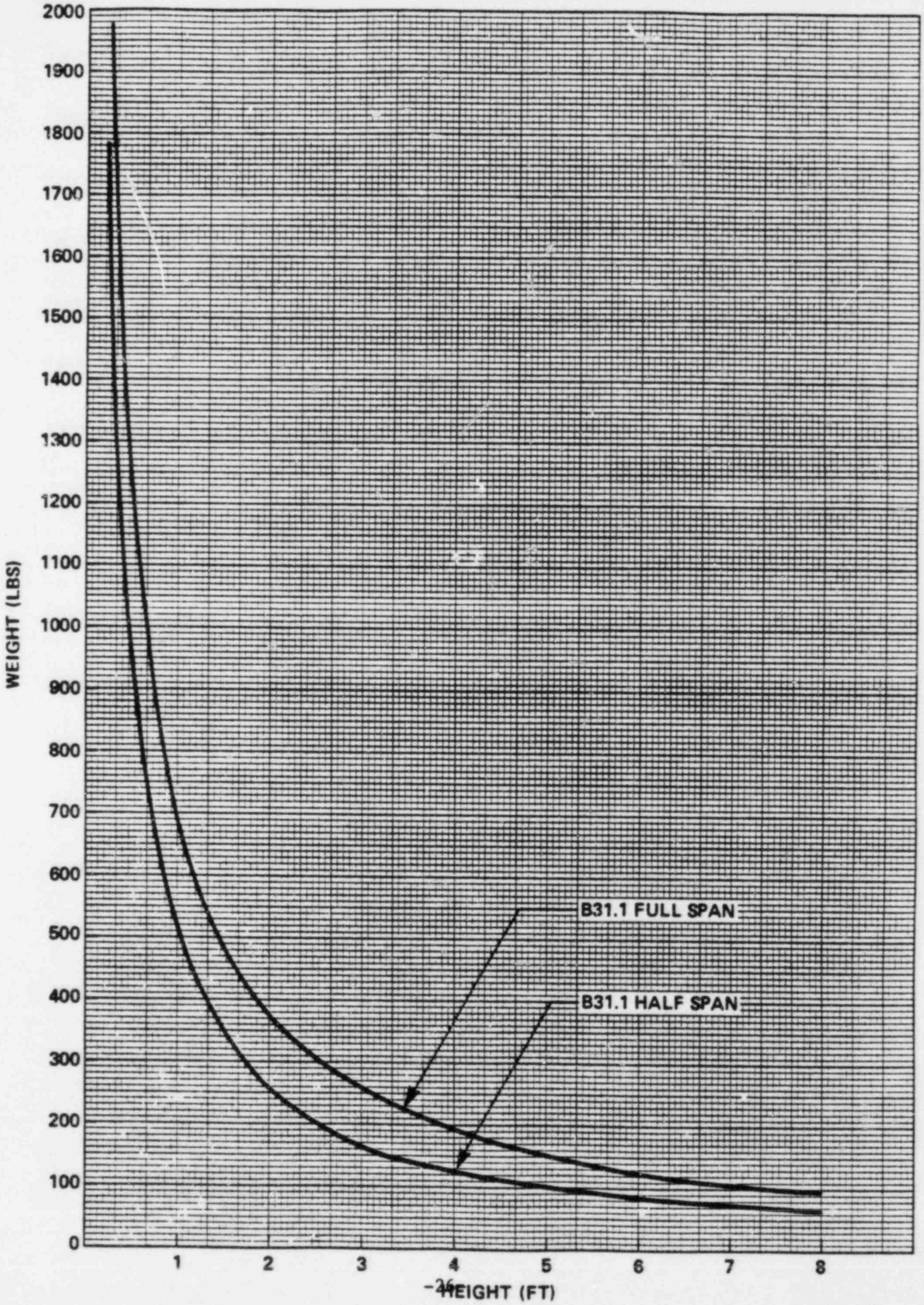


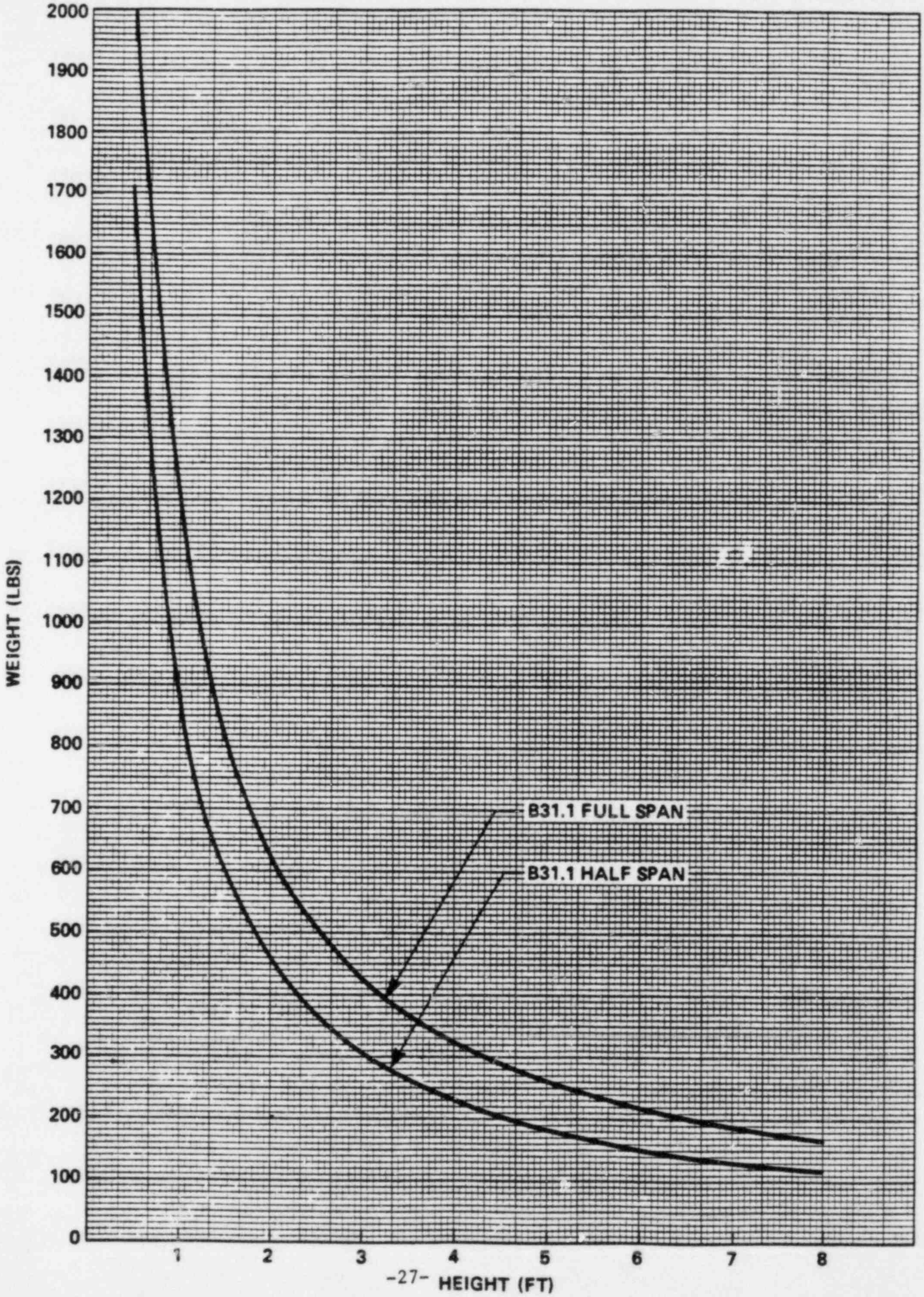
FIGURE 14
ALLOWABLE WEIGHT vs HEIGHT OF MISSILES SUSTAINED BY 4" PIPING SECTIONS



ALLOWABLE WEIGHT vs HEIGHT OF MISSILES SUSTAINED BY 6" PIPING SECTIONS



ALLOWABLE WEIGHT vs HEIGHT OF MISSILES SUSTAINED BY 8" PIPING SECTIONS



ALLOWABLE WEIGHT vs HEIGHT OF MISSILES SUSTAINED BY 10" PIPING SECTIONS

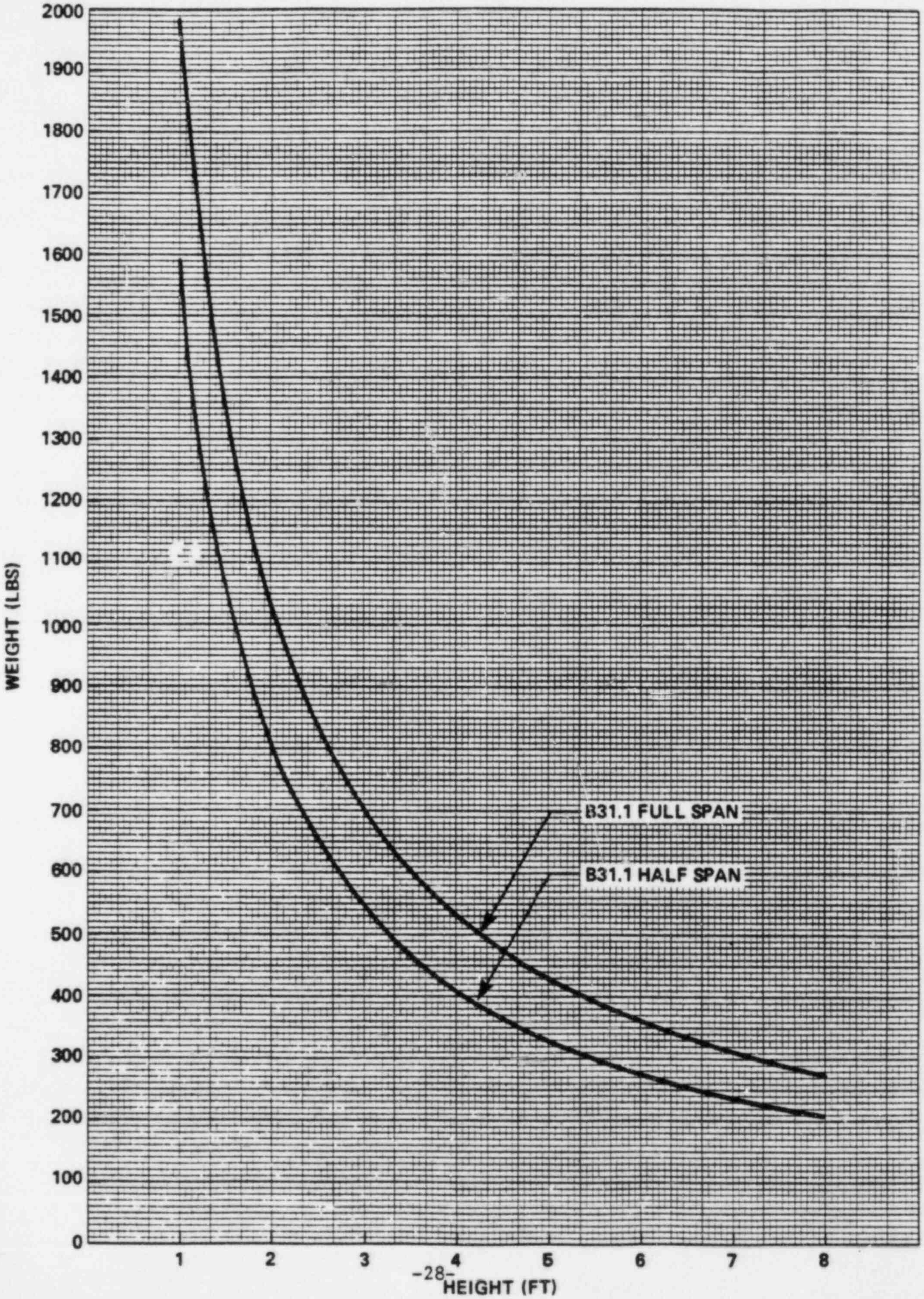
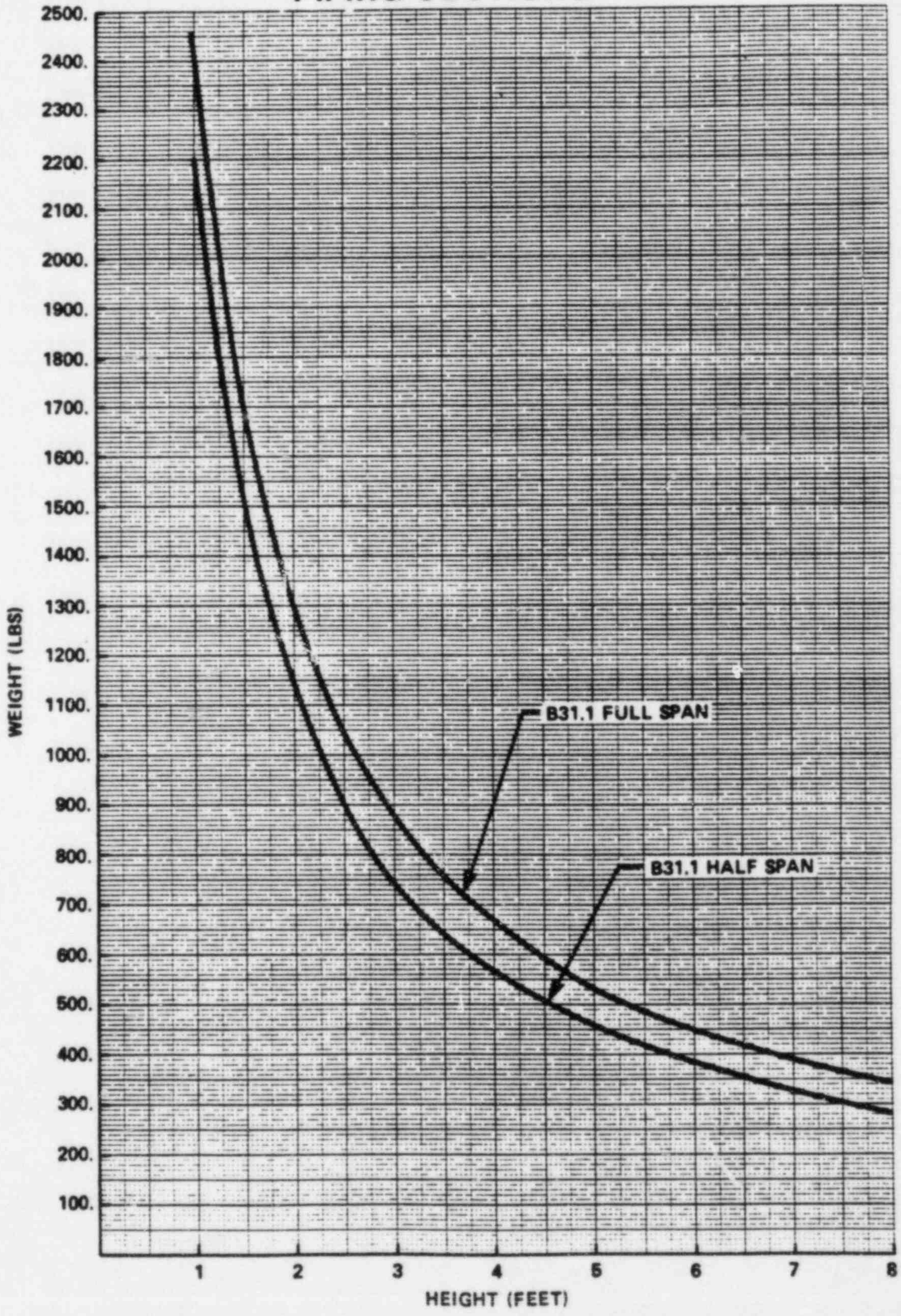


FIGURE 18
ALLOWABLE WEIGHT vs HEIGHT OF MISSILES SUSTAINED BY 12" PIPING SECTIONS



IMPACT CAPACITY OF HVAC DUCTS

I. INTRODUCTION

There is a large variety of ducts sizes and span length used in Comanche Peak. The analysis considered was done for a typical rectangular ductwork together with the stiffner, for various configurations of impact.

The techniques employed in this analysis utilized inelastic method for duct analysis and established deformation limits such that neither of the following is violated.

(1) The Maximum Strain
The maximum strain in the duct is less than 10% of the strain at ultimate or 10 times the strain at yield, whichever is smaller.

(2) The Functionality
The reduction in net flow cross sectional area of the duct is less than 10%.

It will be shown that the limitations of the flow area will govern.

Finite element analysis of the three dimensionally modelled duct (shell elements) with elasto plastic strain hardening material properties and large deformations consideration are conducted.

The typical rectangular duct considered for the analysis is a 36"x27", GA 16 (thickness = 0.0635") fabricated from Galvanized steel in conformance with ASTM A-526 and A-525.

The duct was considered built in accordance with HVAC-Ducts, Louvers and Accessories Specification 2323-MS-85 by Gibbs & Hill and in accordance with SMACNA High Velocity duct Construction Standards.

The span considered in the analysis was intentionally taken as one-half that recommended in the SMACNA standard. This is done to overstate the ill-effects of local failures, thus rendering the analysis conservative.

The duct is considered continuous at the supports.

The duct was provided with the typical stiffener angle at the support all around, made of one angle $1\frac{1}{4} \times 1\frac{1}{4} \times 1/8$.

II. METHODOLOGY

A. Loading and Modelling Approach

Load is applied thru vertical forces (normal to the longitudinal centerline of the duct) at various locations on the top surface to stimulate various conditions of impact.

There are four cases of impact considered.

1. Longitudinal distributed line load applied at the top of duct parallel to the longitudinal centerline of the duct, Fig. 19.

2. Transversal distributed line load applied at the top of duct perpendicular to the longitudinal centerline of duct at the center of the span, Fig. 20.

3. Concentrated load applied to the centerline of cross section of duct on the stiffner, Fig. 21.

4. Concentrated load applied at the top of duct at the center of the span, Fig. 22.

The duct is loaded with the forces which are gradually increased while the response of the duct is continuously checked against both limiting criteria, the maximum strain and functionality. As will be shown later, due to the significant postbuckling strength which the duct exhibits, the criteria used to establish the limits of the force was governed by the functionality requirement.

B. Model

A Three dimensional model of one span of the duct of 5'-0" was developed, considering the duct supported at the bottom side only, and assuring the continuity of the cross section of the duct over the supports.

C. Finite Element Code:

The ABAQUS, Version 4-5-147, using material and geometric nonlinearity was used.

D. Boundary Conditions

For all types of loading conditions the same model is used. The model has two planes of symmetry.

1. Vertical plane at the centerline of cross section containing the longitudinal centerline of duct.

2. Vertical plane containing the cross section of the duct, normal to the longitudinal centerline of duct located at the center of the span.

Therefore,both conditions of symmetry are utilized to reduce the model to 1/4 of its original size. Also, at the support the continuity of the duct is considered using a vertical plane of symmetry normal to the longitudinal centerline of duct.

The duct is supported at the bottom edge in the three translational directions, being able to rotate.

Description of the finite element Model

Six complex type shell elements are used to describe the half of the cross section of the duct. Along the length there are 5 subdivisions (5 rows of elements). Therefore, a total of 30 (6x5) shall elements connecting 143 nodal points are used to describe the 1/4 of the model.

The shell elements used are quadratic elements with corner points and midpoint along the edges there are 8 nodal points defining the shape function of the element (ABAQUS type S8R).

The stiffener is modelled with beam elements using a beam in space, with three nodal points and the cross section angle (L, ABAQUS type B32). There are 6 beam elements connecting 13 grid points.

The finite element model is shown in Fig. 23.

Material Properties

A stress strain curve of elasto-plastic strain hardening properties (Fig. 24) is usded to describe the shell and beams material properties.

The curve is digitized from the strain power law ($S=S_o \varepsilon'^n$) the plastic region, whereas in the elastic region the modulus of elasticity as per ASME code is used.

The ultimate stress and strain are extracted for the material properties.

Galvanized carbon steel sheet metal properties are in accordance with ASTM-A525, A526, A527; and AISI 1015-1017 grade, for Hot Rolled Carbon Steel:

$$Su = 50{,}100 \text{ psi}$$
$$\varepsilon u = 0.279$$
$$E = 27.8 \times 10^6 \text{ psi}$$
$$Sy = 27{,}500 \text{ psi}$$

III. RESULTS

The Force-Deformation (P-Δ) curves are obtained for the duct under each type of loading. A typical P-Δ curve is shown in Fig. 25.

This curve exhibits a well defined post-buckling behavior. As the load is increased from zero to Point B, the plate becomes geometrically unstable although it is still elastic. The instability may cause the plate to buckle by displacing to Point C while theoretically shedding load. However, when buckling commences the shape of the plate change from that of a thin plane plate to an inverted dome shape. This results in a strong catenary action which causes a post buckling stability at Point C. This post buckling stability brought about by the newly formed catenaries will continue in effect even as the load is increased beyond that at Point B. The penalty for this, however, is the excessive deformations and distortion to the duct as can be seen by examining Point D of Fig. 25. It is observed that the displacement has increased by an order of a magnitude. While the load hardly recovered its peak value of the beginning of instability.

Although the above scenario is possible in theory, in practice a modified version of that scenario takes place. This modification is caused by the fact that the plate is not only geometrically unstable at Point B, but also at any state between Point A and Point B. This is evident once one realizes that above Point A, the plate can undergo displacement at no load increase from any point on Segment A-B to the opposite point on Segment B-C, then continuing to Point C while shedding off load, then to post-buckling stability. This is known as snap-through buckling.

Snap-through buckling could be either stable or unstable, depending on Segment C-D of the P-Δ diagram. For stable snap-through buckling two conditions must be satisfied; first the Segment C-D should be continuously increasing, and second the Point D should sustain a load higher than that existed at Point B, prior to reaching the ultimate strain of the material. Both these conditions are met in every load case considered in this analysis. (In Fig.25, Point D was established at 10% area reduction).

Figures 26 through 29 show the P-Δ diagrams for the different impact scenario's of the duct. In each of these scenario's, a stable snap-through is achieved. Although the duct suffers large deformations in the post buckling region, strains are small. It was consistently found that the limiting criteria in duct impact analysis was the functionality requirement, established as a maximum of 10% reduction in the cross-sectional area of the duct.

The shape of the deformed cross-section of the duct at 10% area reduction is shown in Fig. 30. The energy expended by the duct in deforming up to the limiting criteria is computed by integrating the area under the P-Δ curve. This would be the energy available for impact, or conversely, the maximum energy contained in the missile if 100% energy transfer from the missile to the duct is assumed. However, due to the potential of a snap-through buckling, the energy contained under the unstable segment A-B-C is not considered when the area under the P-Δ curve is integrated to compute the energy available for impact.

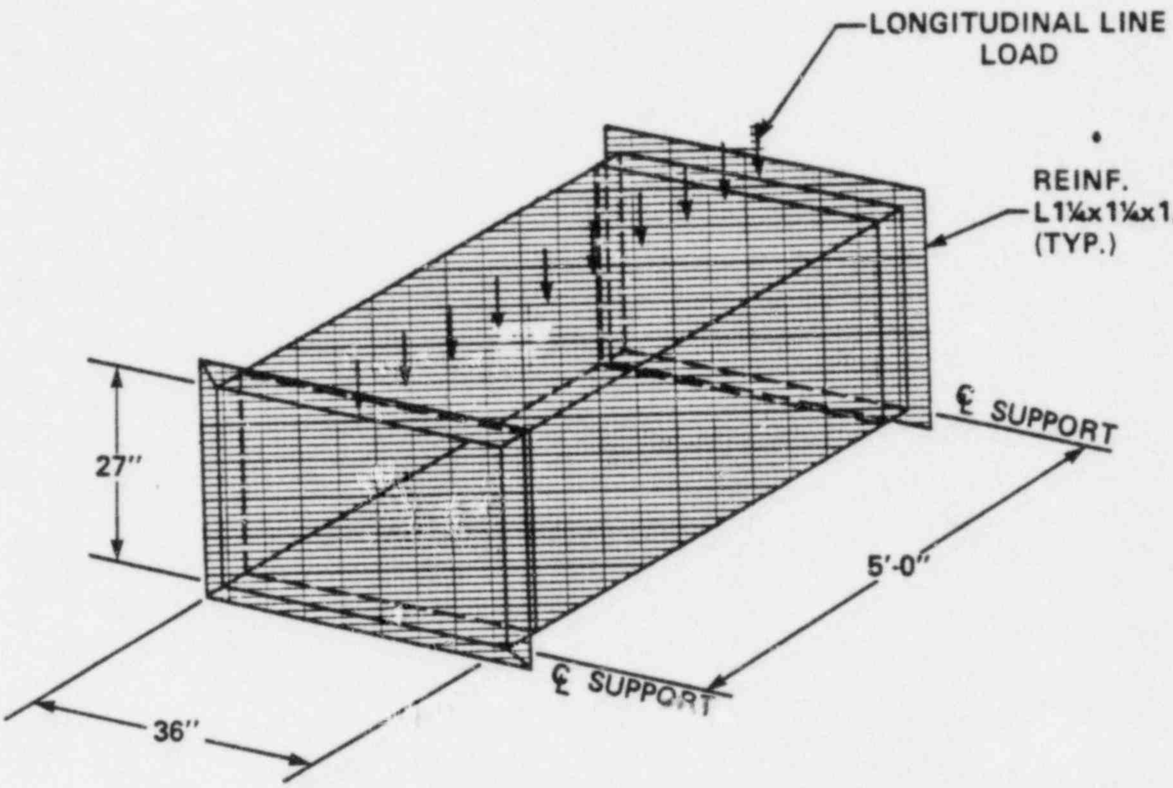
The allowable impact load (W) is obtained by equating external work done by the missile to the energy available for the impact.

$$W = \frac{E}{H}$$

where H is height of missile above the duct.

In Figures 31 through 34 are shown for each impact scenario, the allowable weight vs. height of missiles sustained by the duct when no more than 10% reduction in cross-sectional area is allowed.

FIGURE 19
TUG C COMANCHE PEAK



HVAC DUCT 3[illegible] 16 (t = 0.0635)
MAT ASTM A-525
SPAN = 5'-0"
REINF ANGLES @ SUPPORTS L1¼"x1¼"x1/8"

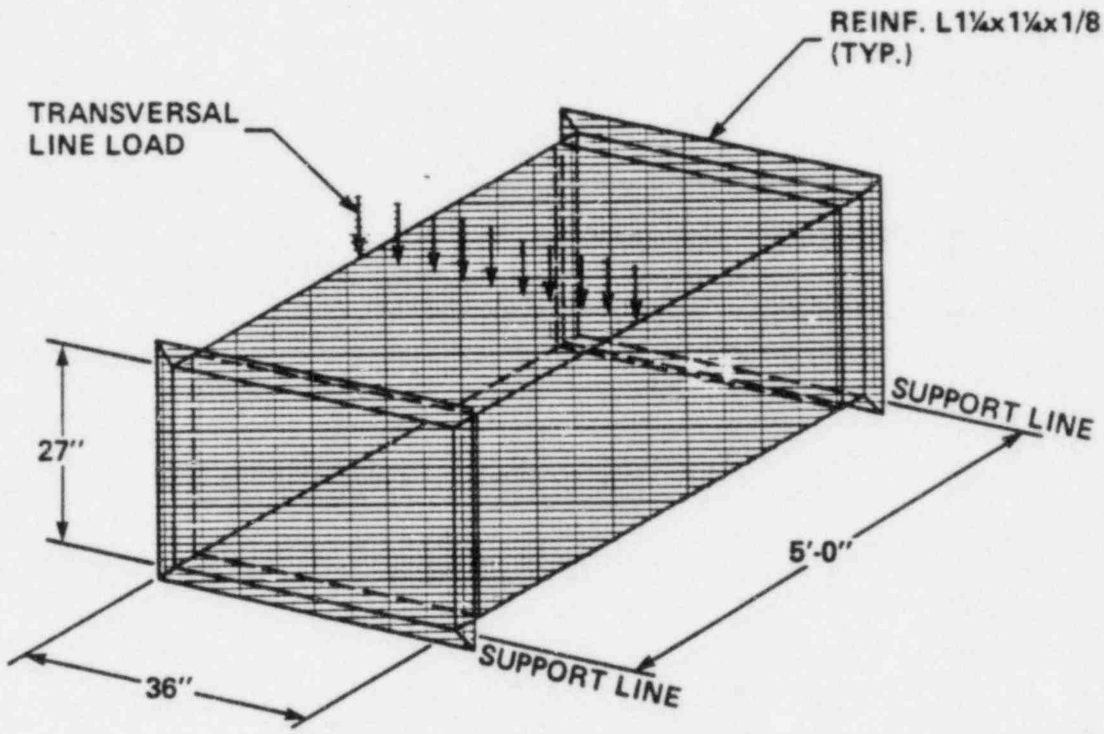
LONGITUDINAL LINE LOAD

FINITE ELEMENT MODEL – 1/4 OF MODEL

CODE – ABAQUS VERSION 4-5-147
NONLINEAR ANALYSIS

FIGURE 20

TUG C COMANCHE PEAK



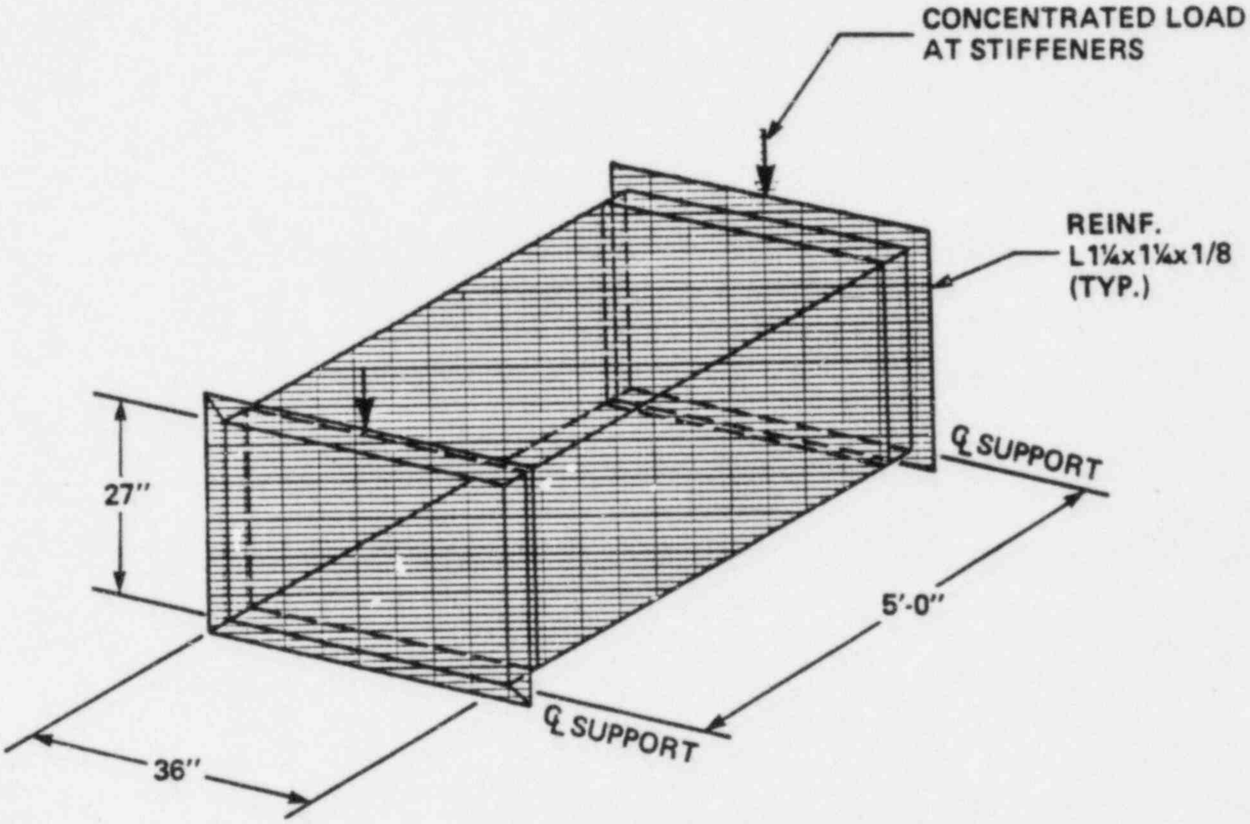
HVAC DUCT 36"x27" GA 16 (t = 0.0635")
SPAN = 5'-0"
REINF. ANGLES @ SUPPORTS L1¼x1¼x1/8

TRANSVERSAL LINE LOAD

FINITE ELEMENT MODEL – 1/4 OF MODEL

CODE – ABAQUS VERSION 4-5-147
NONLINEAR ANALYSIS

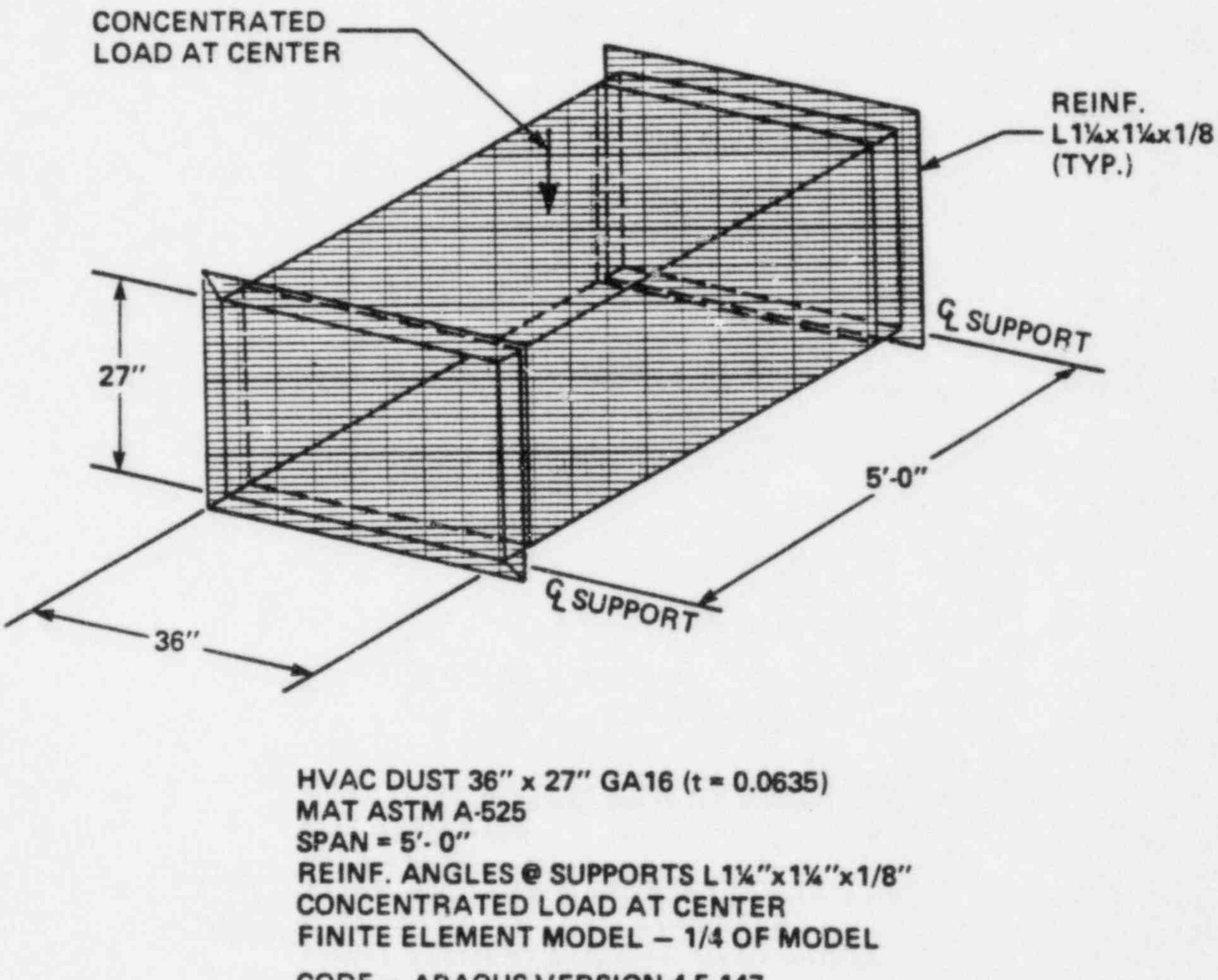
FIGURE 21
TUG C COMANCHE PEAK



HVAC DUST 36" x 27" GA16 (t = 0.0635)
MAT ASTM A-525
SPAN = 5'-0"
REINF. ANGLES @ SUPPORTS L1¼"x1¼"x1/8"
CONCENTRATED LOAD AT STIFFENERS
FINITE ELEMENT MODEL — 1/4 OF MODEL

CODE — ABAQUS VERSION 4-5-147
NONLINEAR ANALYSIS

FIGURE 22
TUG C COMANCHE PEAK



HVAC DUST 36" x 27" GA16 (t = 0.0635)
MAT ASTM A-525
SPAN = 5'- 0"
REINF. ANGLES @ SUPPORTS L1¼"x1¼"x1/8"
CONCENTRATED LOAD AT CENTER
FINITE ELEMENT MODEL – 1/4 OF MODEL

CODE – ABAQUS VERSION 4-5-147
NONLINEAR ANALYSIS

FIGURE 23
¼ MODEL HVAC 36" X 27" DUCT

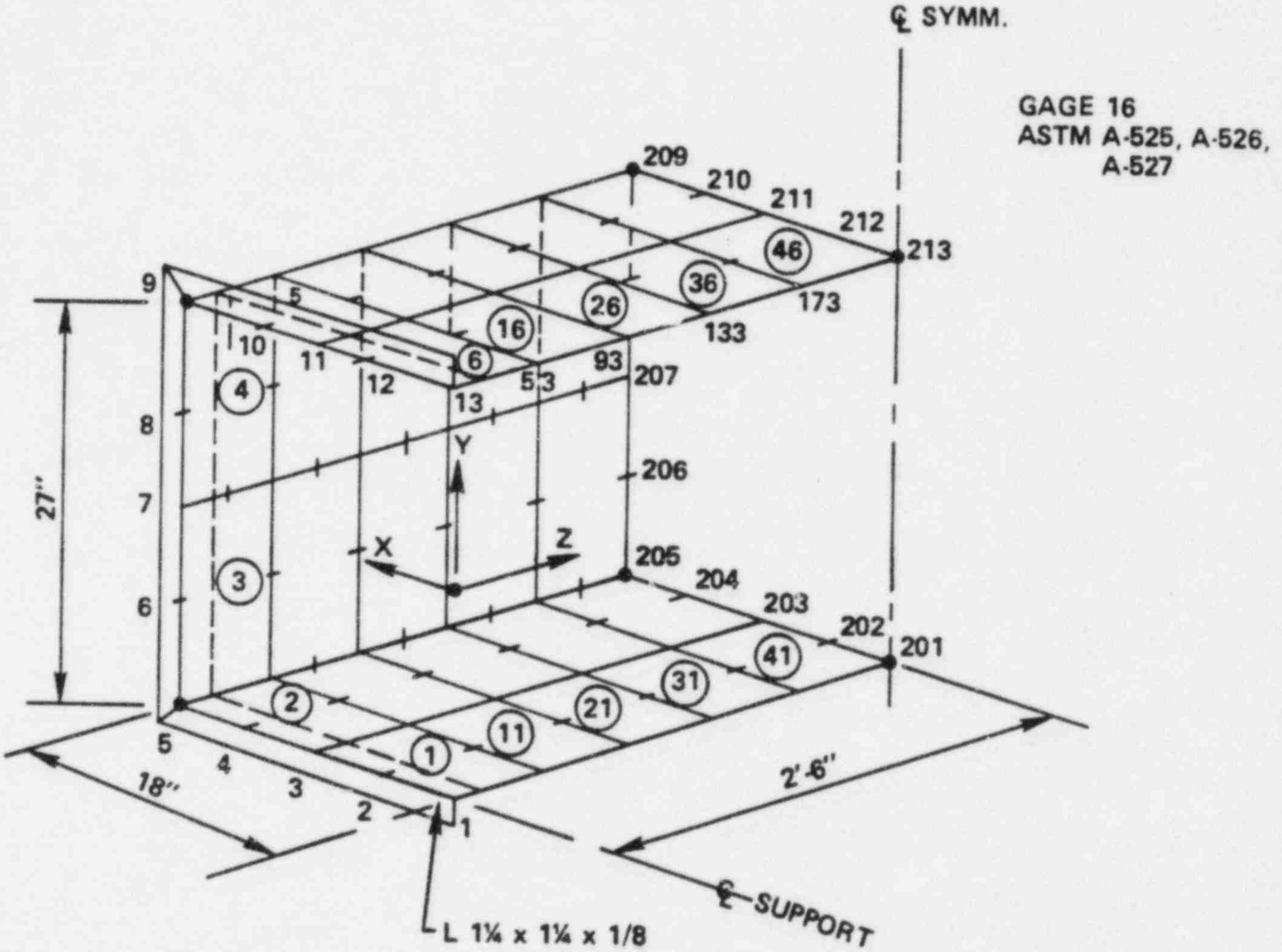


FIGURE 24
STRESS STRAIN CURVE FOR ASTM A-526, A-527, A-525

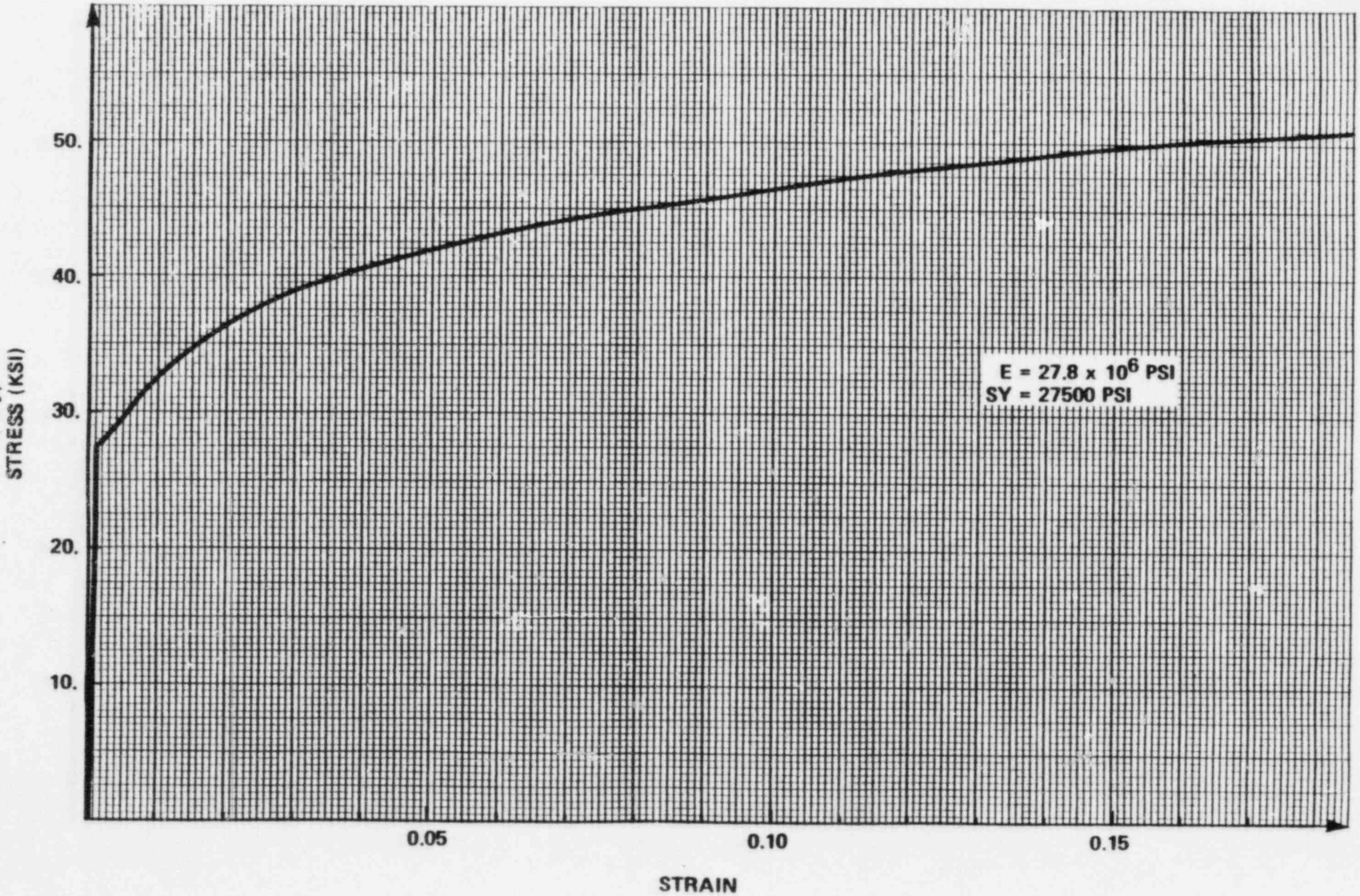


FIGURE 25
TYPICAL P-Δ DIAGRAM FOR DUCTS

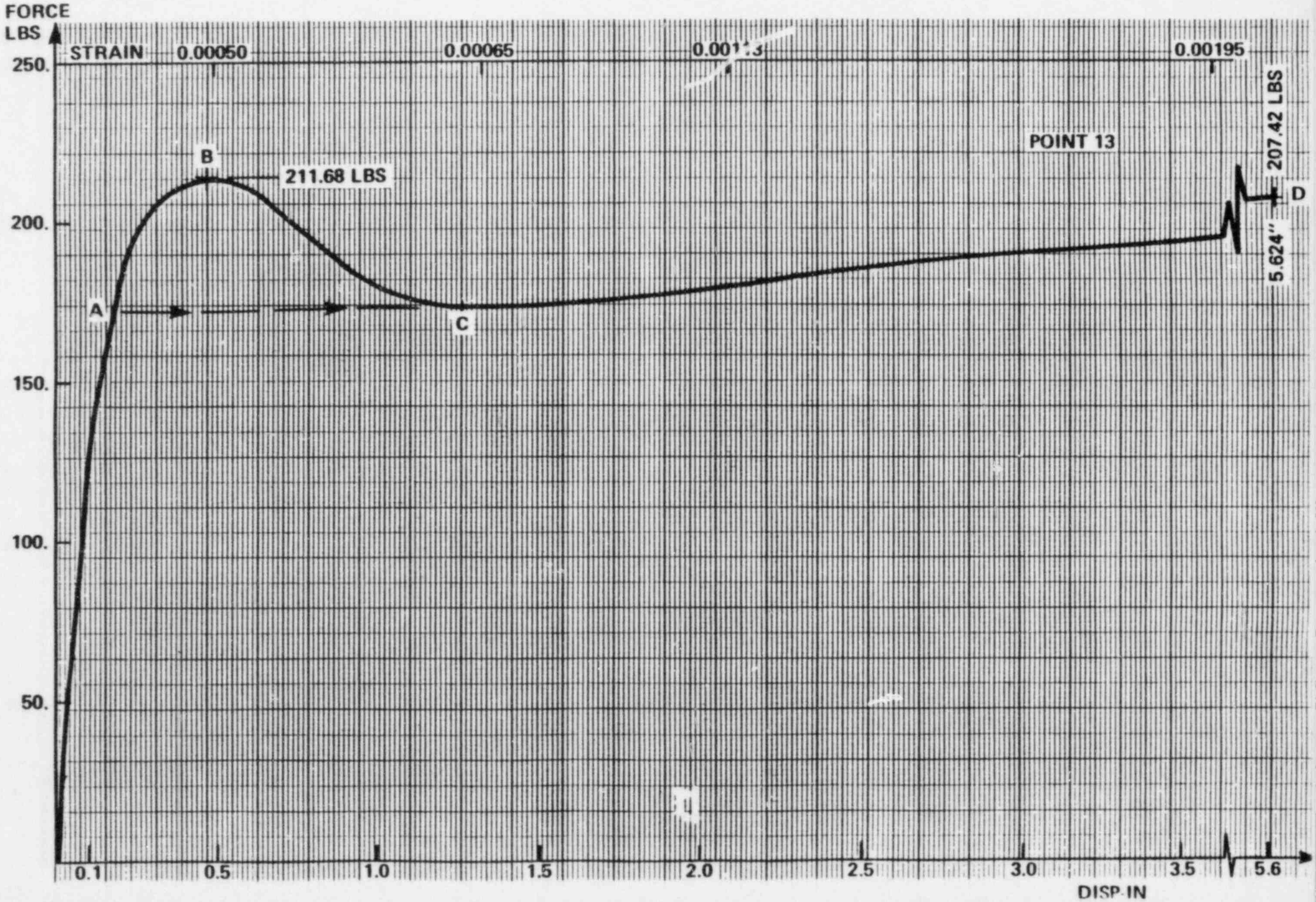


FIGURE 26

TUGC COMANCHE PEAK LOAD-DISPLACEMENT CURVE HVAC DUCT 36"x27" GA 16L = 5'-0, REINF L1¼x1¼x⅛ LONGITUDINAL LINE LOAD

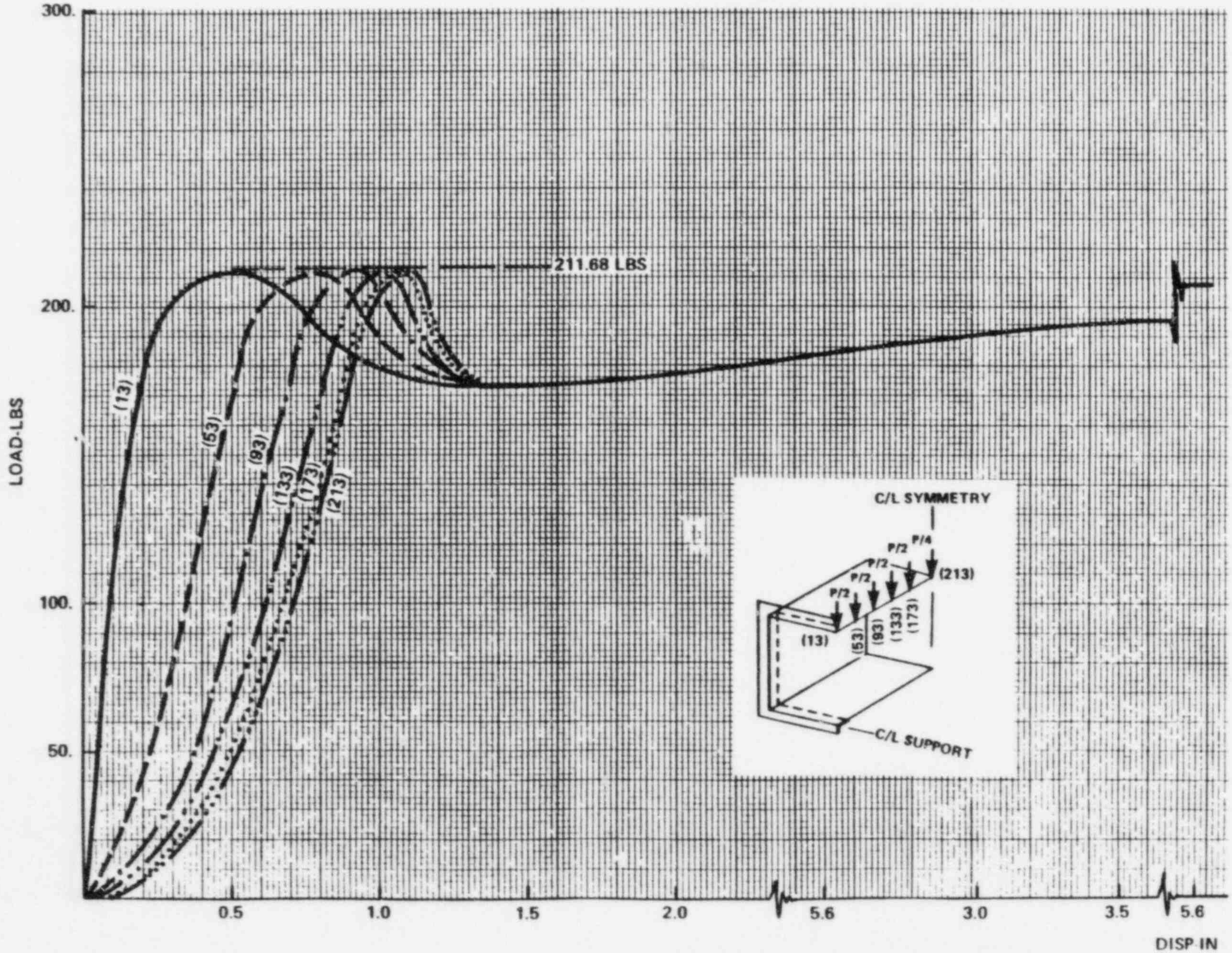
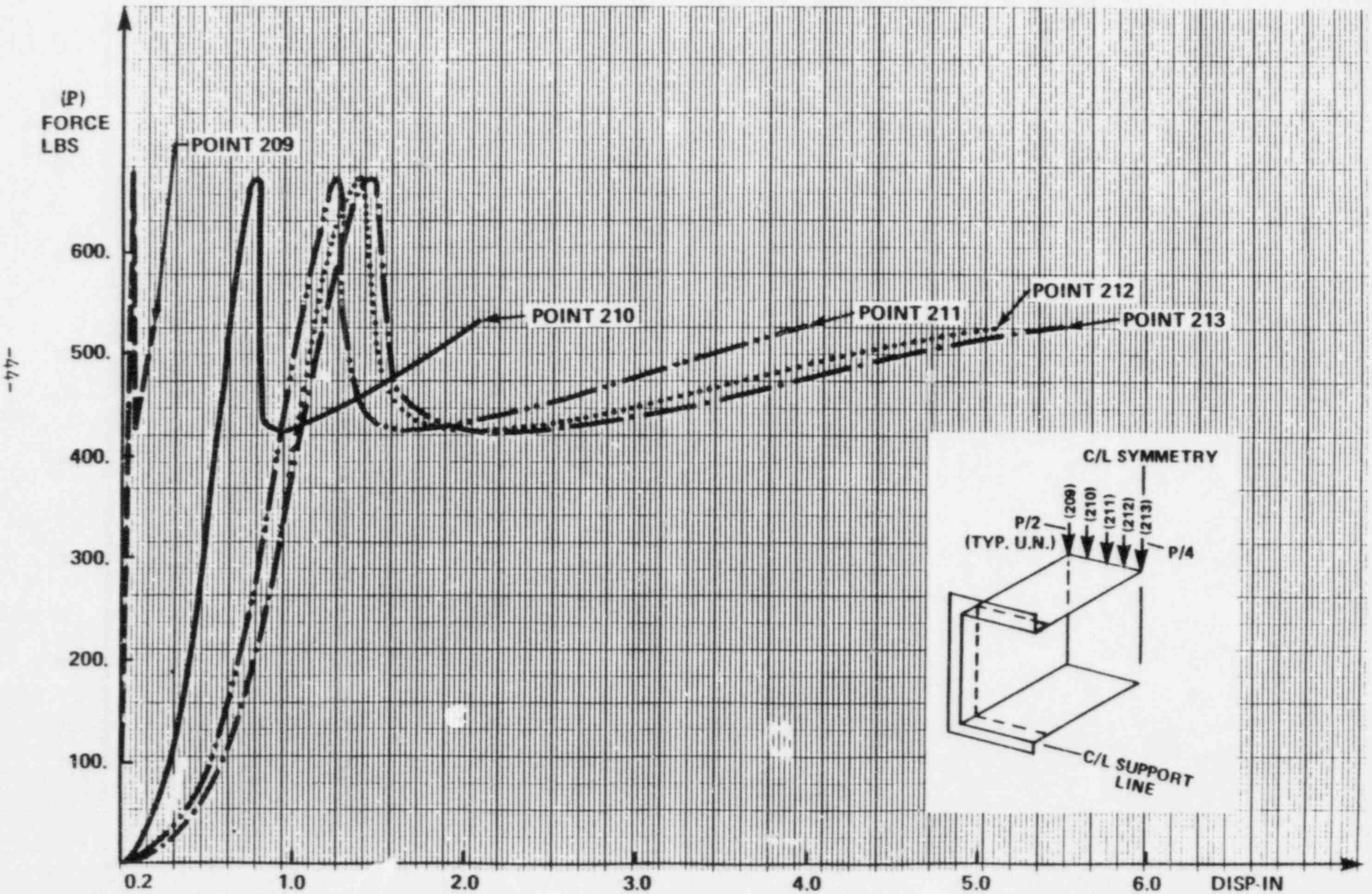


FIGURE 27
TUG C - COMANCHE PEAK
LOAD - DEFLECTION CURVE HVAC DUCT 36"x27", GA 16
L = 5'-0, REINF. L1¼x1¼x⅛ TRANSVERSAL LOAD



LOAD - DEFLECTION CURVE HVAC DUCT 36"x27", GA16 L = 5'-0, REINF. L1¼x1¼x⅛ CONCENTRATED LOAD AT STIFFENER

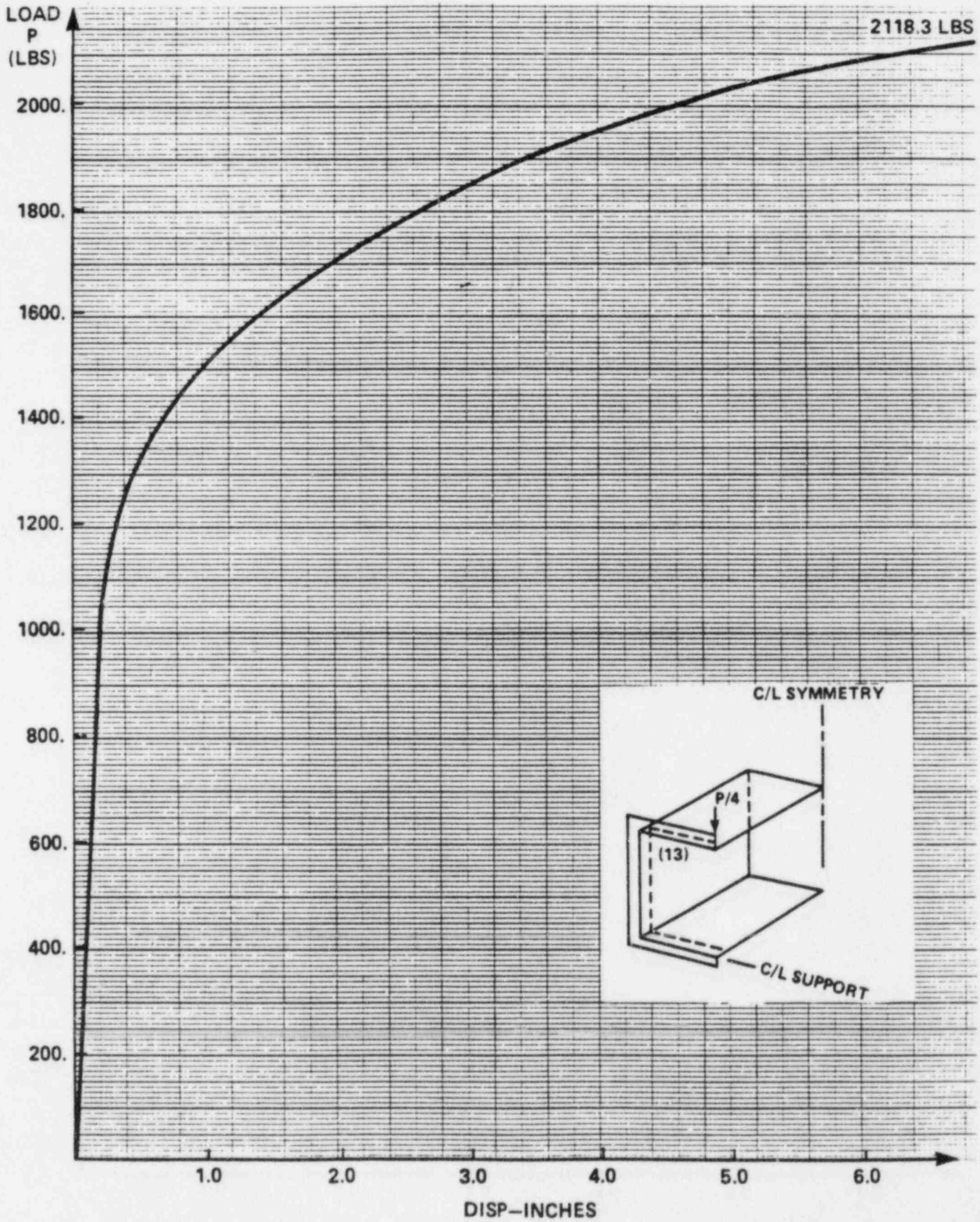


FIGURE 29

TUGC COMANCHE PEAK LOAD-DISPLACEMENT CURVE HVAC DUCT 36"x27" GA16, L = 5'-0, REINF L1¼x1¼x⅛ POINT LOAD AT CENTER

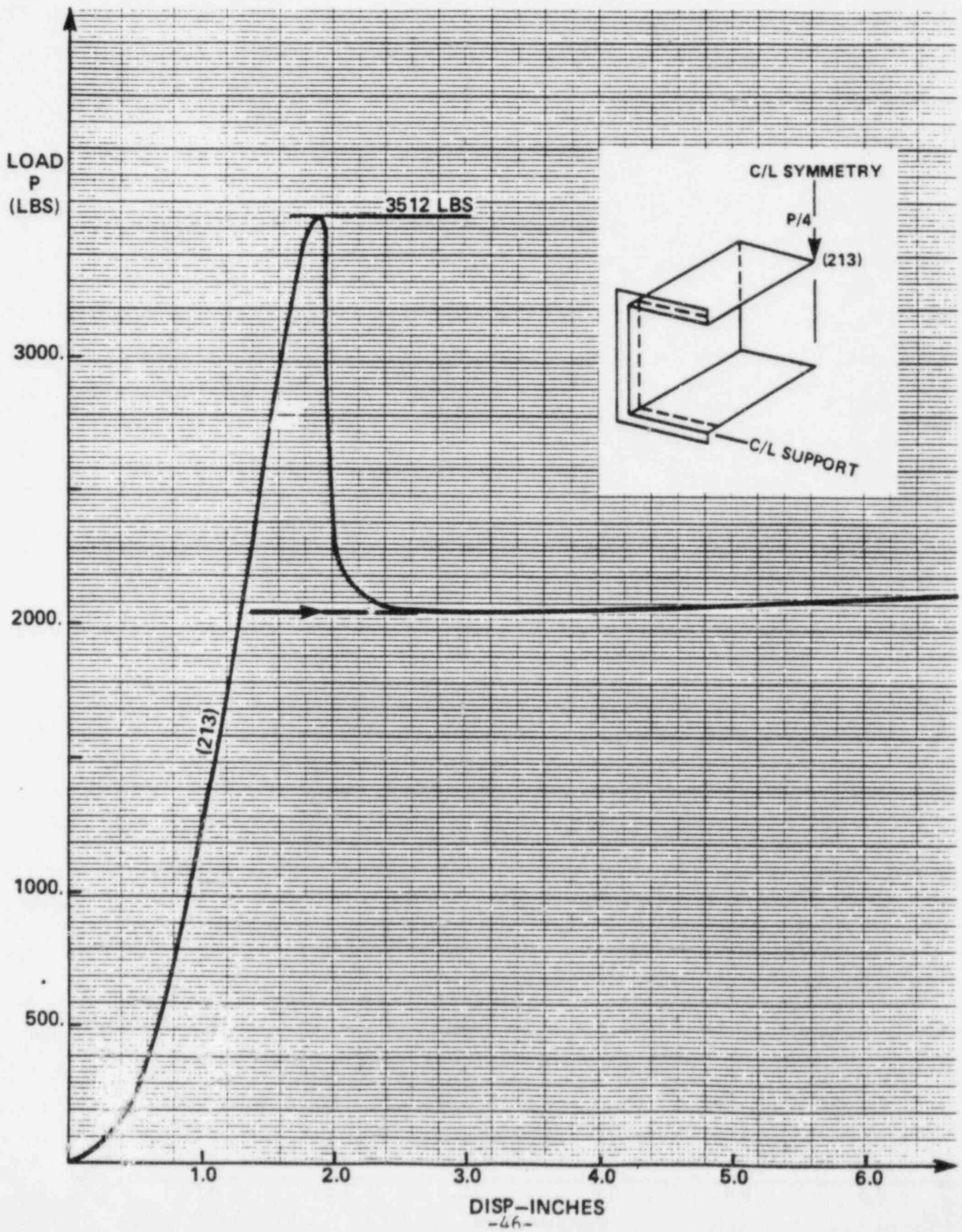
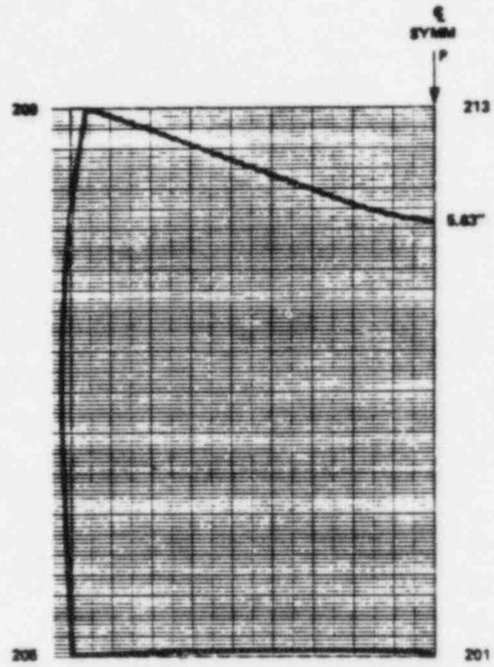
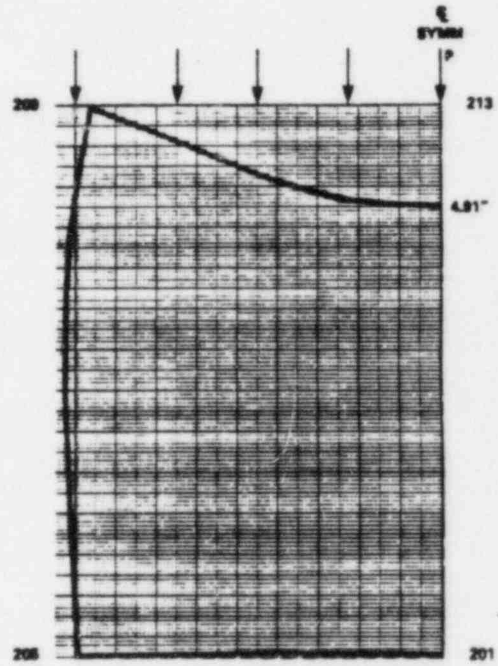


FIGURE 30

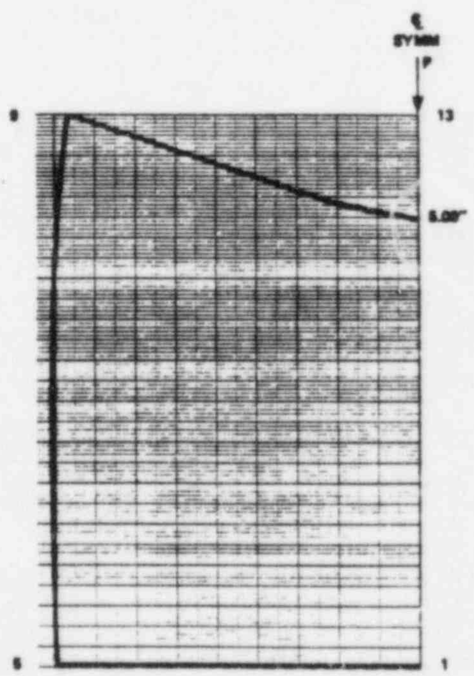
LONGITUDINAL LINE LOAD
DEFORMED SECTION AT 10% AREA REDUCTION



TRANSVERSAL LINE LOAD
DEFORMED SECTION AT 10% AREA REDUCTION



POINT LOAD AT STIFFENER
DEFORMED SECTION SECTION AT 10% AREA REDUCTION



POINT LOAD AT CENTER
DEFORMED SECTION AT 10% AREA REDUCTION

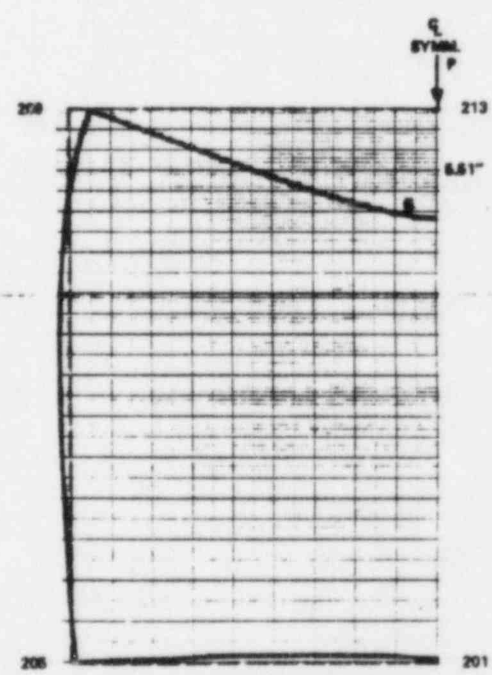


FIGURE 31

ALLOWABLE WEIGHT VS. HEIGHT OF MISSILES SUSTAINED BY A 36"x27" - GA16, DUCT IMPACTED ALONG LENGTH

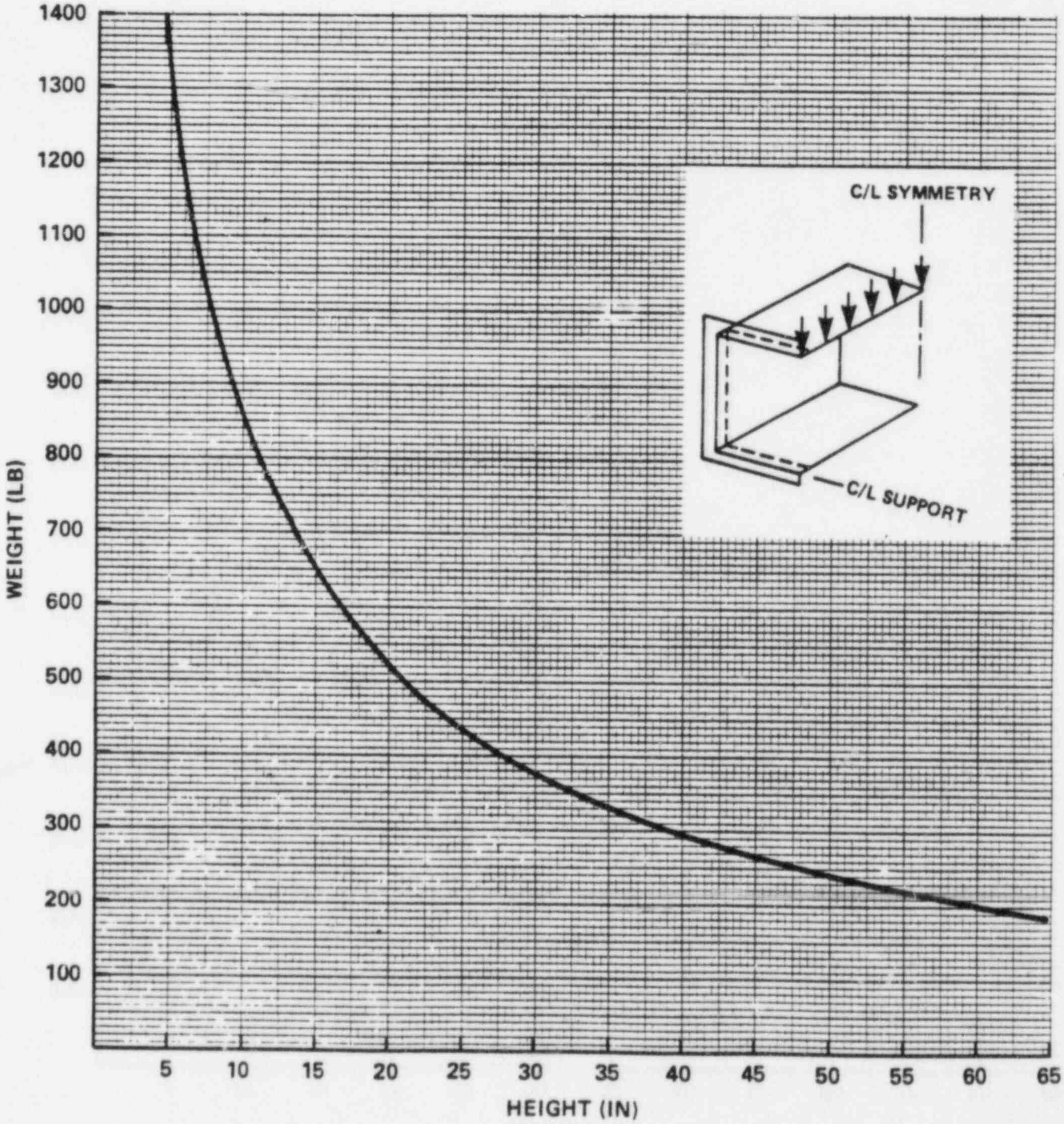


FIGURE 32
ALLOWABLE WEIGHT vs HEIGHT OF MISSILES SUSTAINED BY A 36"x27" - GA16, DUCT IMPACTED ACROSS LENGTH

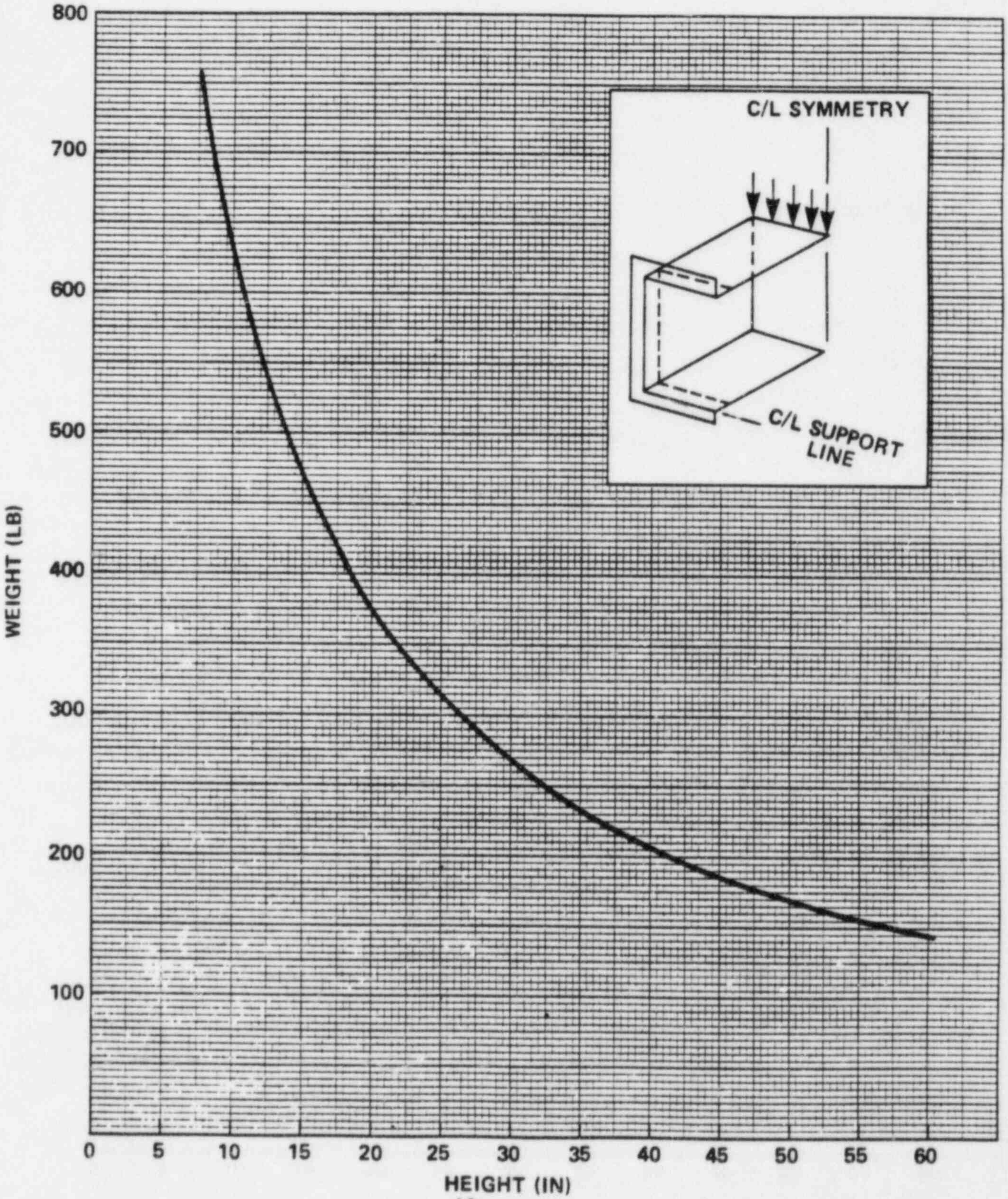


FIGURE 33

ALLOWABLE WEIGHT vs HEIGHT OF MISSILES SUSTAINED BY A 36"x27" - 16GA DUCT IMPACTED AT STIFFENER

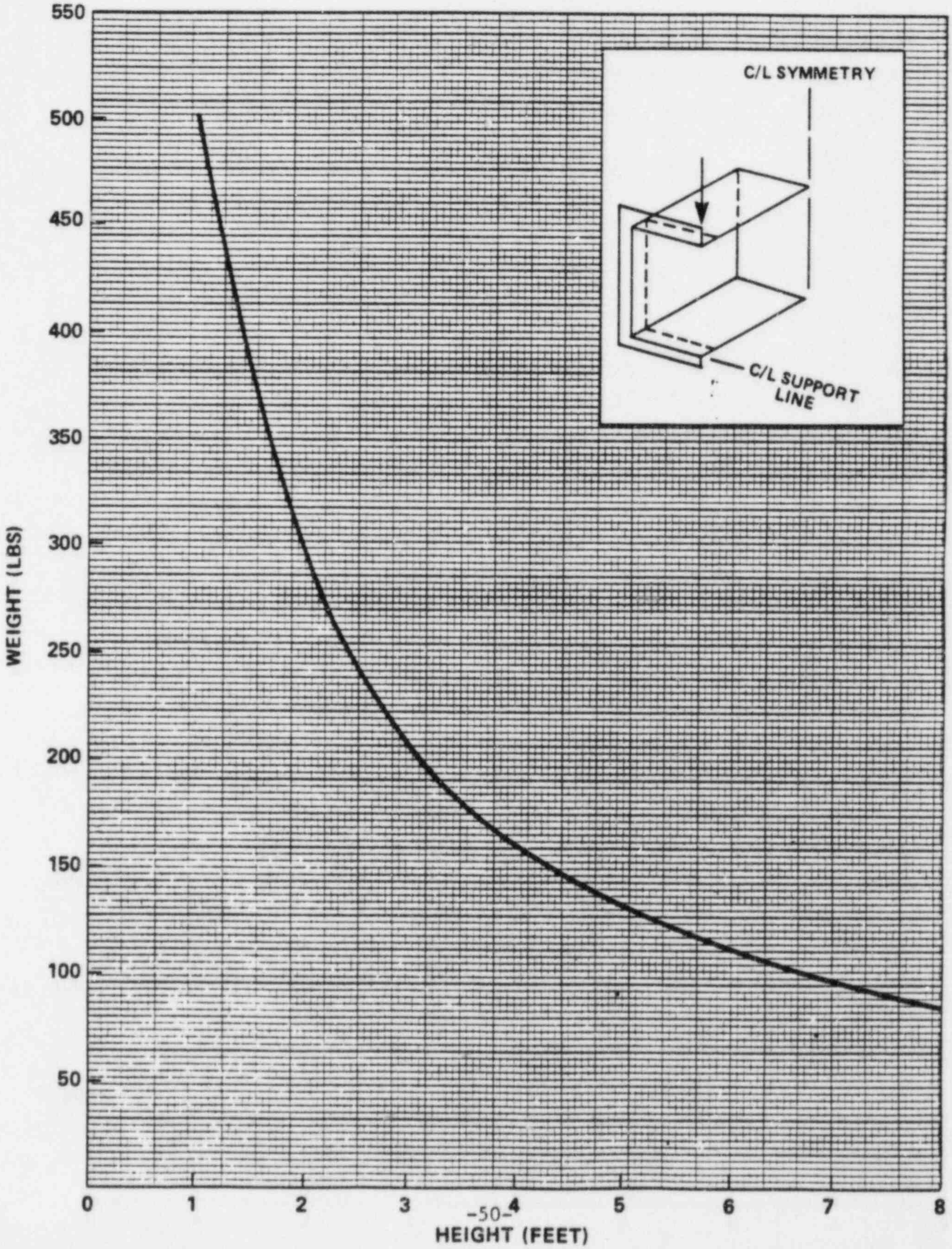
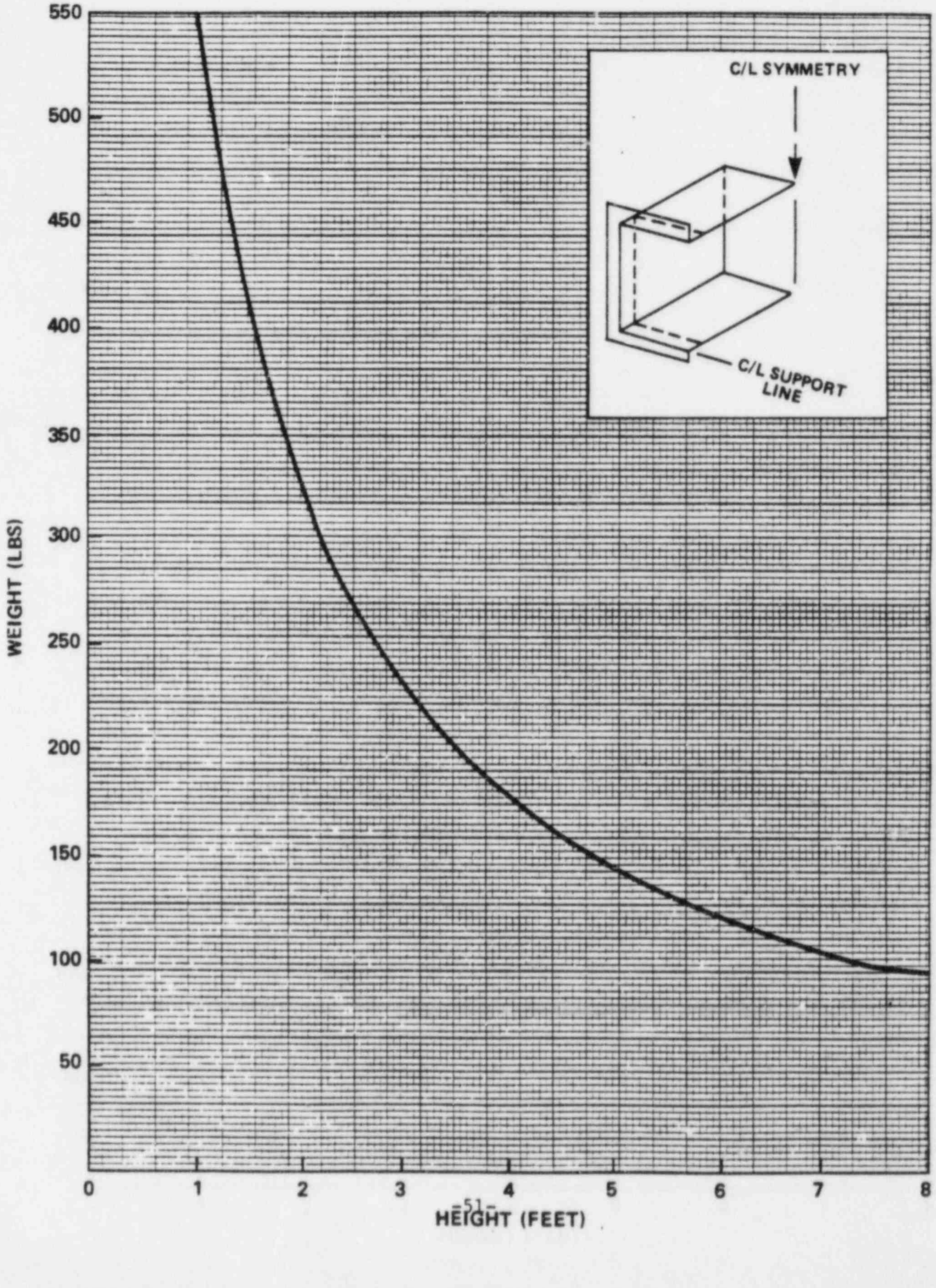


FIGURE 34
ALLOWABLE WEIGHT vs HEIGHT OF MISSILES SUSTAINED BY A 36"x27" - 16 GA DUCT IMPACTED AT MIDPOINT



IMPACT CAPACITY OF STRUCTURAL MEMBERS

Objective:

It is assumed that structural members can be allowed to deform during an impact such that the maximum strain anywhere in the member does not exceed 10 times the yield strain. The deformation may be axial or in bending. Thus the allowable impact energy may not exceed the work needed to reach 10 εy in a combined bending and axial deformation mode. The distribution of work done between axial and bending modes depend on geometry.

In this approach work done to reach 10 εy is plotted against ratio of M/P for various lengths for each structural section. Using these curves once M/P is known from the geometry, the allowable energy of impact can be found.

Procedure for Developing the Curves

To develop the curves one has to develop relations between M&θ and P&δ and also develop a procedure to find work done when both P&M act.

Finding M-θ Curve

$$M = \int \sigma y \, dA \; (-8\theta) \tag{1}$$

(1-8θ) is a correction factor such that at collapse , M-θ curve also reaches a maximum.

We assume $\frac{\theta}{\varepsilon_o} = \alpha =$ constant where ε_o is max strain in the section on the outside fibers. (2)

If ε is strain at distance y from nutral axis

$$\varepsilon = \frac{\varepsilon_o}{D} \cdot y \tag{3}$$

Where D = distance of maximum strain fibers from natral axis.

Also let $\sigma = \sigma_o \, \varepsilon^n$ (4)

$$M = \int \sigma_o \, \varepsilon^n \, y \, dA \quad (1-\beta\Theta)$$

$$= \sigma_o \left(\frac{\varepsilon_o}{D}\right)^n \int y^{n+1} \, dA \; (1-\beta\Theta)$$

$$= \frac{\sigma_o \varepsilon^n_o}{D^n} \int y^{n+1} dA \qquad (1-\beta\Theta)$$

$$= \frac{\sigma_o \Theta^n}{D^n \alpha^n} \int y^{n+1} dA \qquad (1-\beta\theta)$$

$\frac{dM}{d\theta}$ = 0 at $\theta = \theta^*$, the rotation at collapse

$$\therefore \beta = \frac{n}{(1+n)\theta^*}$$

$$\theta^* = \alpha \, \varepsilon^*$$

where ε^* is collapse strain

Take $\varepsilon^* = 100 \; \varepsilon y$

$$\therefore \quad M = \left[\sigma_o \left(\frac{\theta}{D\alpha}\right)^n \int y^{n+1} \, dA\right]\left[1- \frac{n\theta}{100\varepsilon y(1+n)\alpha}\right] \qquad (5)$$

However, to use this expression,

1) We must know α and ii) verify the validity of putting $\varepsilon^* = 100\ \varepsilon y$ Both of these are accomplished by doing a nonlinear analysis using ABAQUS. A value of α can be estimated from ABAQUS, which when applied in (5) gives moment values very close to ABAQUS results thus validating the procedure including putting $\varepsilon^* = 100\ \varepsilon y$.

Limitations of the Approach

1) For very short spans where shear deformation is predominant, α is no longer constant and the theory is not valid. The minimum span required was found to be about 4 times the beam depth.

2) For very thin sections where elastic buckling occurs in the section due to bending, this type or plastic yielding analysis is not applicable.

Finding P-δ Curve

$$P = \int \sigma \, dA$$

$$\delta = \varepsilon \, . \; L$$

From these P-δ curve is computed

Combined Bending & Axial Force

When P&M act simultaneously the area under the curves is different from the sum of either acting alone. This is accounted for in the calculation by applying P&M alternately in tiny steps, keeping track of the strain in each fibre, computing stress using the power law, performing the integrations, and keeping track of increment in area under M-θ or P-δ in step, till maximum allowable total strain is reached in any fibre.

Let ε_a max be final axial strain

ε_b max be final bending strain

$$\varepsilon_{a\,max} + \varepsilon_{b}\,max = 10\,\varepsilon_y \quad (1), \quad \varepsilon_y = \text{yield strain}$$

$$\text{Let } \frac{\varepsilon_{a\,max}}{\varepsilon_{b\,max}} = X \qquad (2)$$

For a given value of X we can find area under (P-δ) and (M-θ) curves. Also we can find P_{max} & M_{max} the highest axial force and bending moment values.

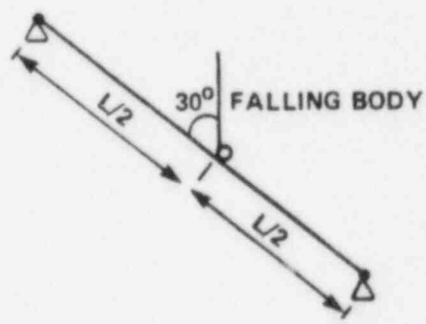
The sum of areas under (P-δ) and (M-θ) curves gives the total energy. Of this only energy from combined bending & axial strain of ε_y to 10 ε_y is available for impact.

Determination of M/P

(1) Determination M/P may not be required. Depending on their function, members may be axial members or bending members. In such cases the asymptotic value (M/P=0) for axial members and (M/P=) for bending members may be used.

A majority of members encountered fall under this category.

(2) However, in members where both bending & axial modes play a part in failure, M/P must be evaluated.



In the above problem, let F be the fictitious vertical force.

Its axial component = F Cos 30^{o}

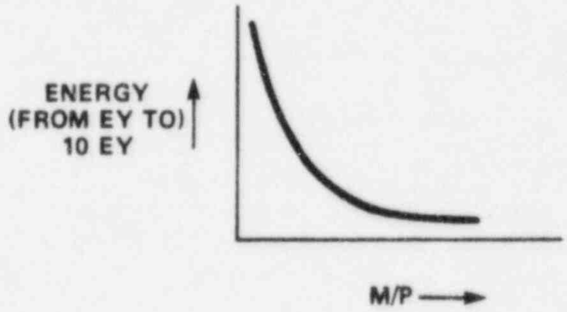
Its bending component = F Sin 30^{o}

Bending moment = $\frac{FL}{4}$ Sin 30^{o}

$$M/P = \frac{L}{4} \tan 30$$

$\frac{M_{max}}{P_{max}}$ gives a ratio

For each value of X, we can get this total energy and the M/P ratio. These can be plotted



Given a cross section and a length, we can thus form a energy versus M/P curve for that member.

Summary Procedure & Recommendations

When a mass falls on a member from a height of h, the work done by the mass

$$= W.h$$

W.h is equated to the sum of areas under (P-S) and (M-θ) curves from ϵy to 10 ϵy total strain.

Let this area be A

$$A = Wh$$

If we can find M/P based on some consideration we can find A from the curves earlier developed.

Thus if we know M/P, we know energy A and hence h is easily determined.

Thus, based on geometric configuration it is always possible to evaluate M/P.

Once M/P is known, corresponding total energy A due to combined bending and axial can be evaluated.

COMPARISON OF WORK DONE

ABAQUS VERSUS EBASCO METHOD

SECTION	LENGTH INCH	α using ABAQUS	*WORK DONE LB INCH ABAQUS	EBASCO
L 3x3x3/8	30	1.15	1.49E3	1.88E3
	60	2.25	2.93E3	3.59E3
	120	4.56	5.86E3	7.3E3
C4x7.25	30	1.3	4.4E3	4.24E3
	60	2.8	8.4E3	8.99E3
	120	4.8	1.69E4	1.54E4
C6x8.2	30	1.0	5.8E3	5.9E3
	60	1.7	1.05E4	1.01E4
	120	3.2	2.05E4	1.91E4
C8x11.5	30**	0.88	8.87E3	9.6E3
	60	1.3	1.49E4	1.42E4
	120	2.4	2.86E4	2.62E4
C10 x 15.3	30**	0.83	13.3E3	14.99E3
	60	1.1	20.2E3	19.86E3
	120	1.94	37.8E3	34.76E3

* ABAQUS values are interpolated

** α not constant for this length

FIGURE 35
ENERGY CAPACITY DIAGRAM
C 4 x 7.25

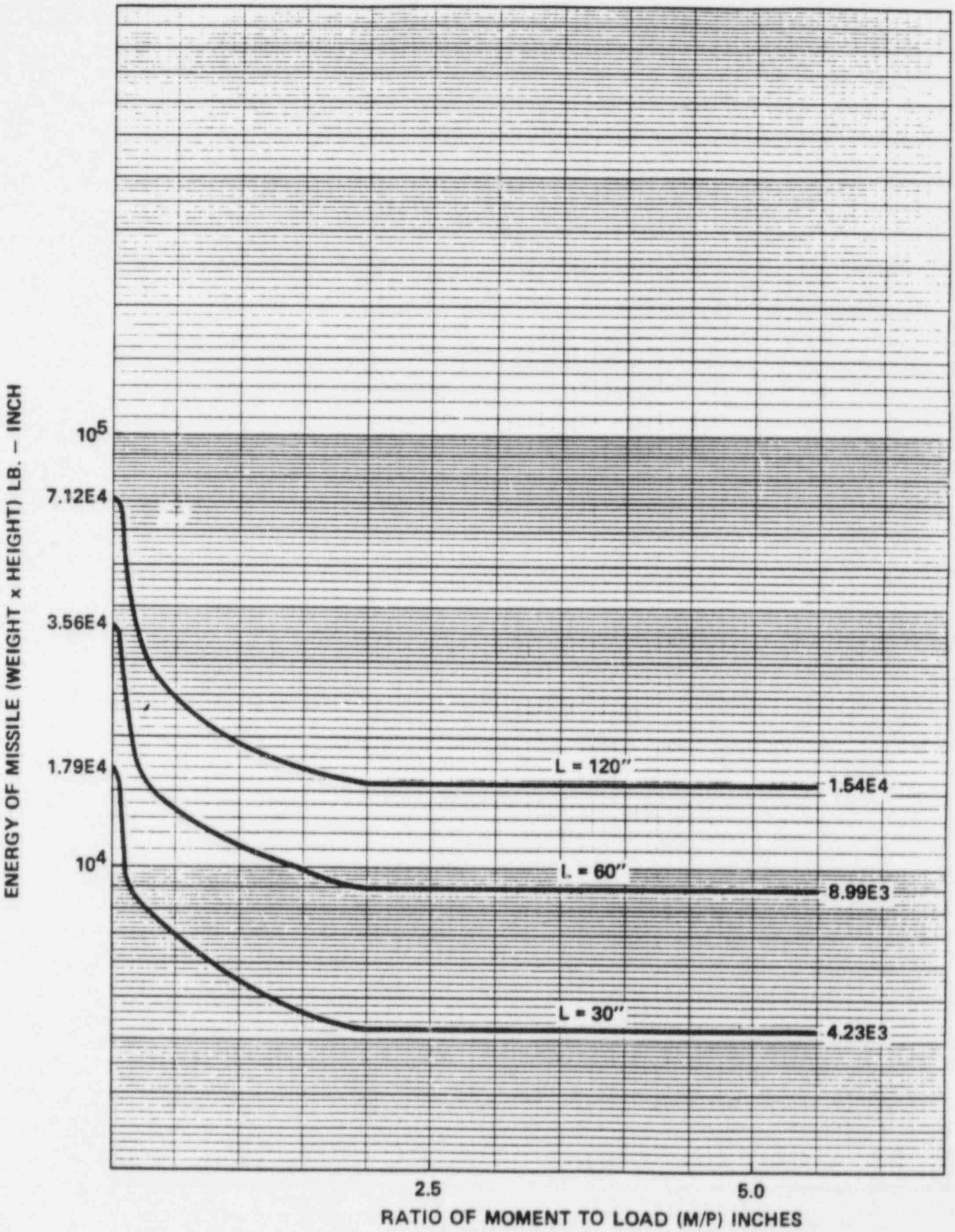


FIGURE 36
ENERGY CAPACITY DIAGRAM
C6X8.2

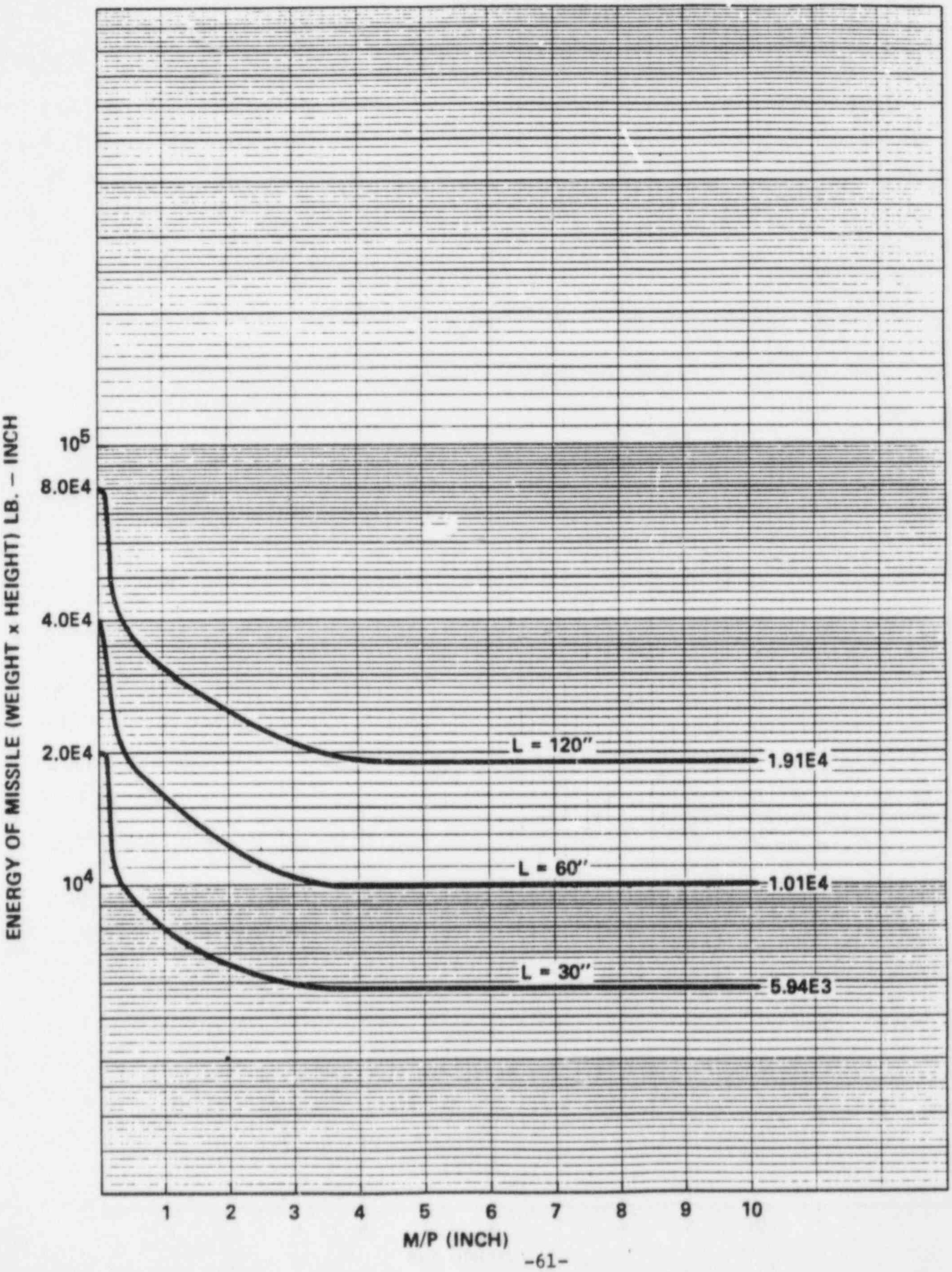


FIGURE 37
ENERGY CAPACITY DIAGRAM
C 8 x 11.5

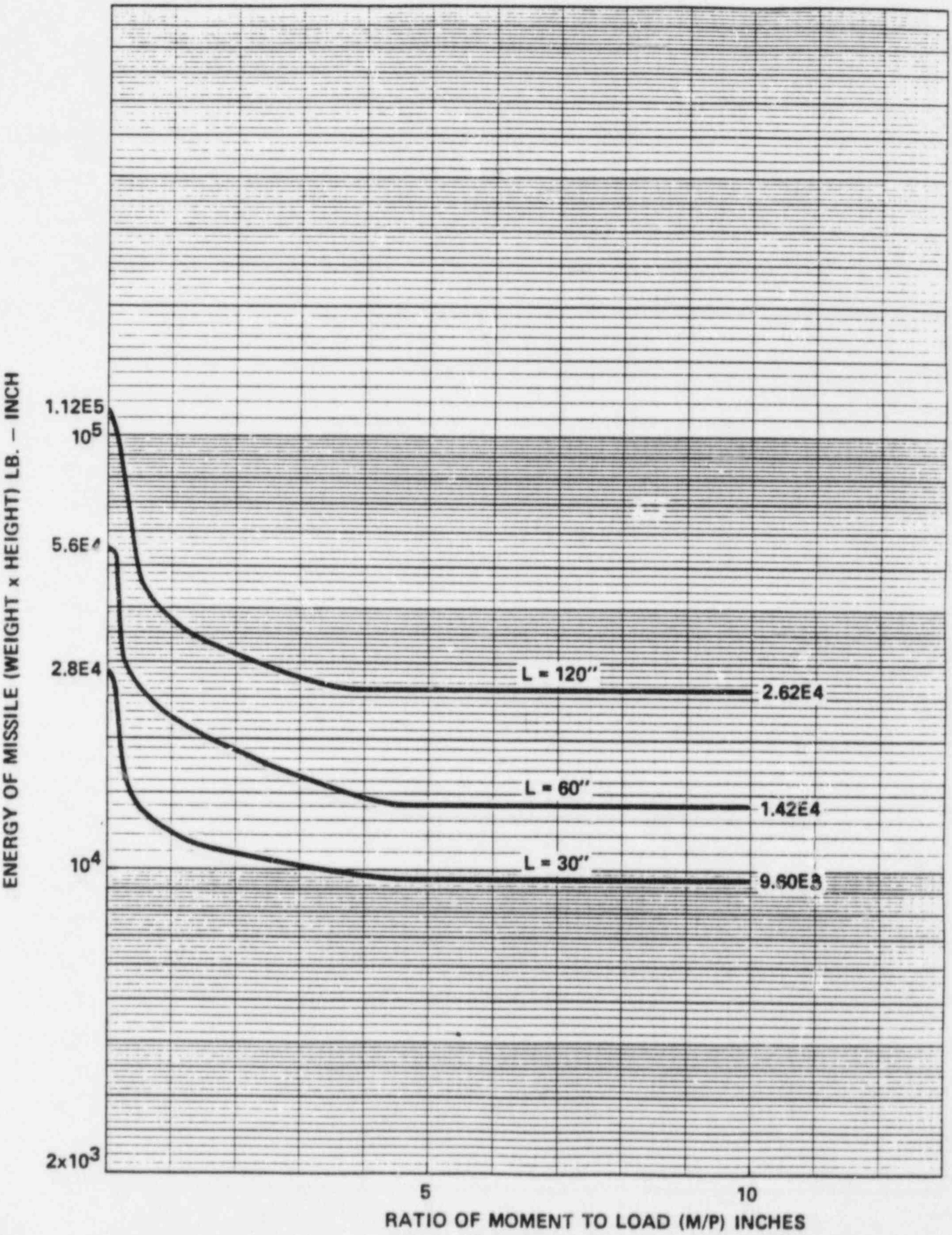


FIGURE 38

ENERGY CAPACITY DIAGRAM
C 10 x 15.3

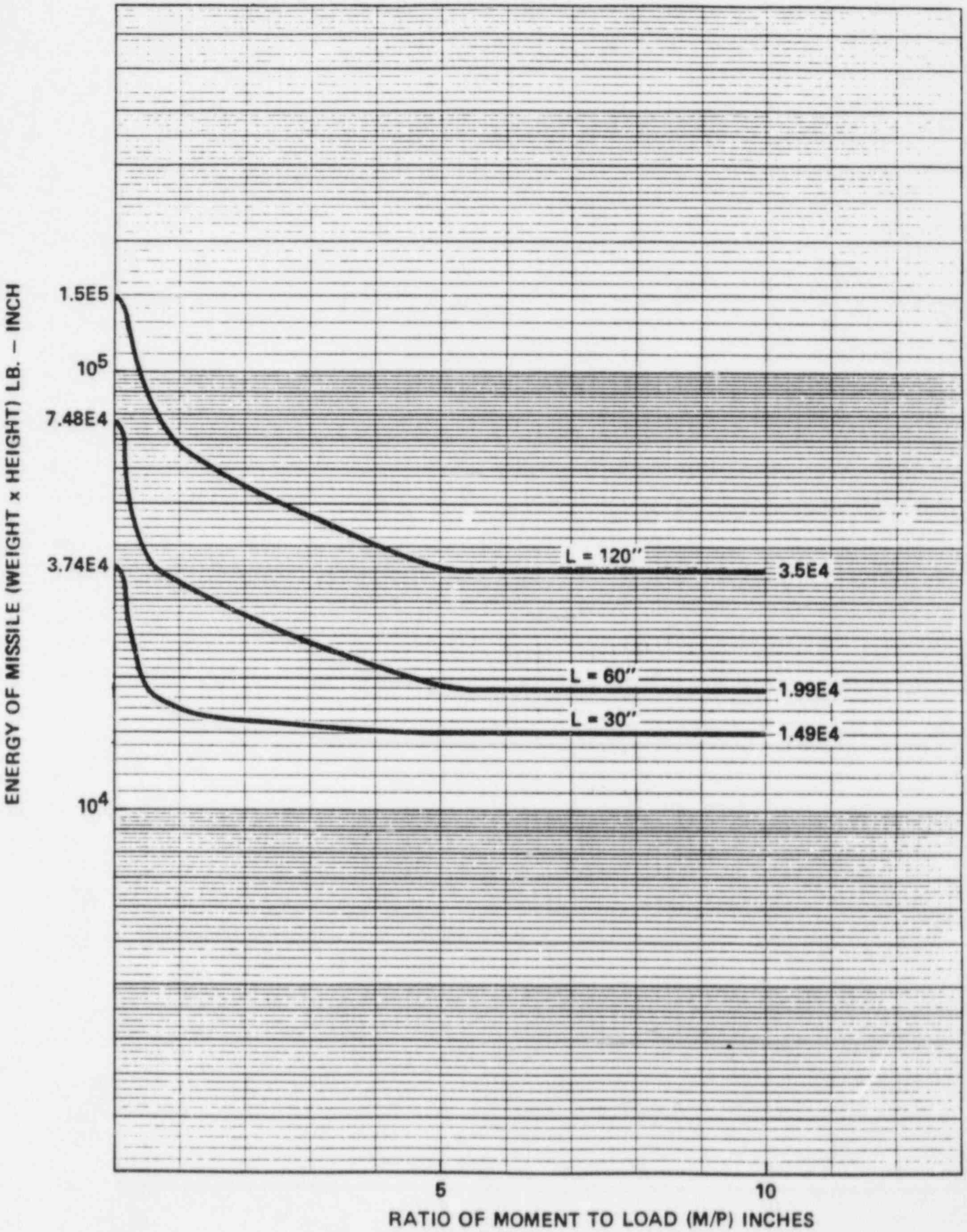
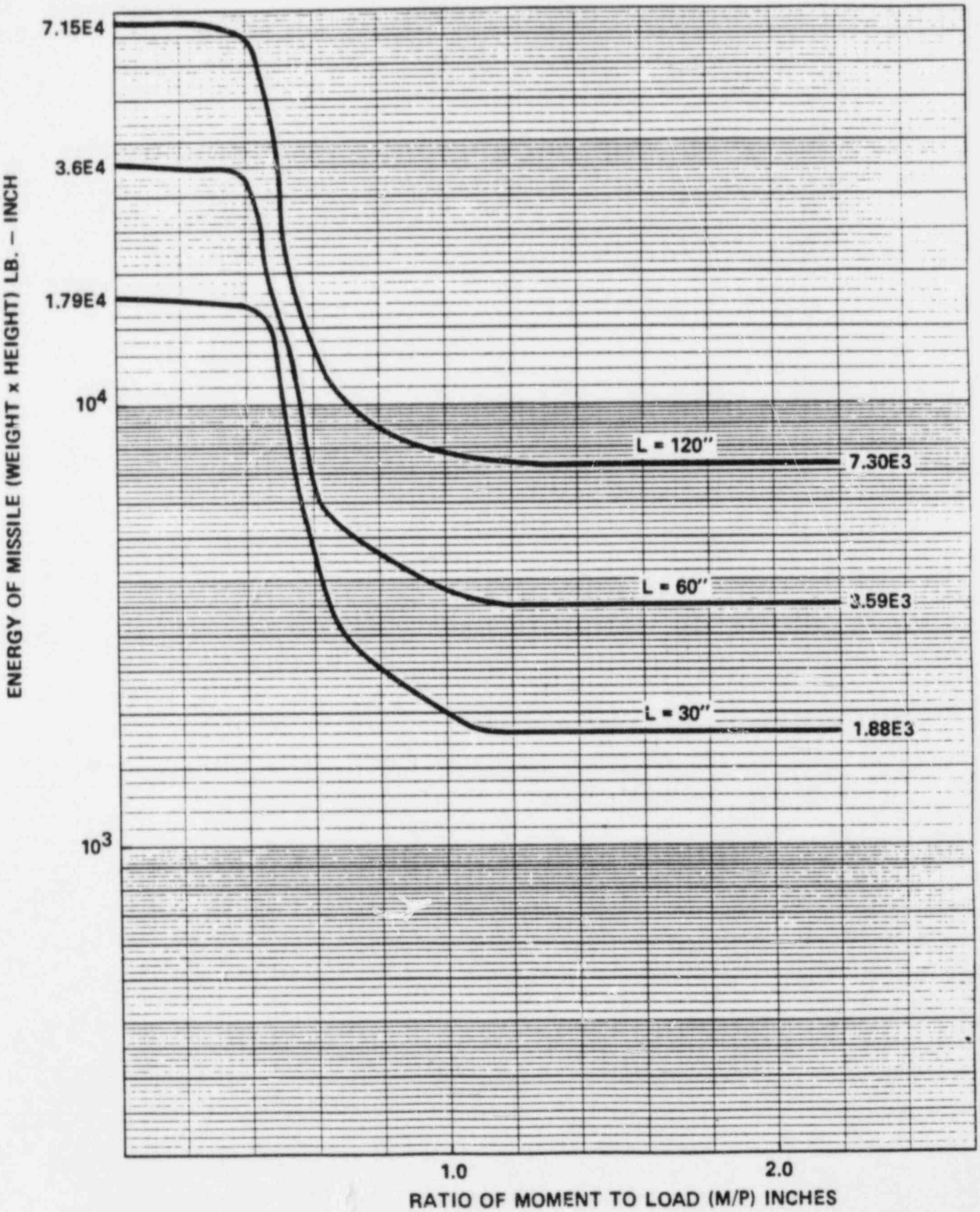


FIGURE 39

ENERGY CAPACITY DIAGRAM
L 3 x 3 x 3/8



Length of Missile Conduit Span which Participates in Impacts Upon Targets

A. Introduction:

The following scenario is to be investigated. A length of conduit falls from a certain height upon a target pipe. It is desired to determine the total energy transfer from the conduit to the pipe and hence the length of conduit which participates in the impact process. It is expected that the initial kinetic energy of the conduit at impact will be apportioned in the following way:

1. Energy is expended in overall beam bending of both the conduit and pipe spans.
2. Energy is expended in local shell deformations of both the missile and target.
3. Some of the original Kinetic energy of the conduit becomes shell vibration and beam vibration kinetic energy in both the missile and target.

Eventually the conduit rebounds and if succeeding impacts are assumed it can be expected that additional energy can be transferred from the conduit to the pipe. In this connection, however, it is important to note for the energy categories 1 and 2, cited above, that energy corresponding to permanent shall deformations or the energy involved in bending beyond the point where a plastic hinge is formed is non-recoverable. Non-recoverable energy in the conduit is not available to be transferred to the pipe in succeeding impacts. Non- recoverable energy in the pipe is not available to be given back to the conduit when it rebounds.

To investigate the energy transfer from the conduit to the pipe, the computer program PISCES 2D has been used. PISCES 2D is a general purpoase continuum mechanics code based on a finite difference formulation. It has capability to indicate the shell deformation of the target pipe, and the beam behavior of the missile conduit. It also keeps track of the relevant kinetic energies. However, since the two dimensional version of PISCES was used translational symmetry was employed.

In the PISCES model, described in the next section, the conduit was modeled as equivalent plate and as such it was not possible to determine the energy lost due to conduit shell permanent deformation. This is obviously conservative as regards to the energy transfer to the target pipe. If latter one wishes to remove this conservatism it may be done by investigating local shell deformation of both the conduit and pipe using a traditional finite element program.

B. Discussion of PISCES Model

For both the conduit and pipe translational symmetry was used well and only half the pipe and conduit was actually modelled. Appropriate boundary conditions were used to ensure the impact occurred along the line of symmetry namely:

1. At x = 0 the conduits has no x motion and no rotation.

2. At x = 0 the pipe shell has no x motion and no rotation. In addition the bottom of the pipe has no y motion.

The pipe was modelled as a shell corresponding to six inch schedule 40. The conduit was modelled as a plate with the following characteristics.

1. It is assumed that the conduit is two inch schedule 40 pipe, sixty percent full of cable, and continuously repeated into the transverse plane

2. The plate thickness was chosen to give the area moment of inertia of the conduit and hence the correct bending stiffness.

3. The plate mass density was chosen to give the correct conduit weight per foot and hence the correct inertia.

For this conduit-pipe combination eight cases were studied, Namely, the conduit was dropped from heights of two and eight feet and the conduit lengths used were eight, sixteen, twenty-four, and thirty-two feet. Some results of these analyses are included in the next section. For example, Table 1 lists the maximum percent energy transfer from the conduit to the pipe for a single impact. It is also the percent of the original conduit span which participates in the first impact. For the case of thirty-two feet of conduit falling from eight feet, figures one through fifteen show how the conduit bends and the pipe deforms as the impact progresses. Figures sixteen through twenty-two privide velocity vector plots for the conduit as the impact proceeds. In particular we note that after the conduit rebounds from the pipe the ends of the conduit still have a downward velocity and hence there is still the possibility of a plastic hinge forming which will result in some energy becoming unavailable for succeeding impacts.

Finally, as mentioned before, the energy transfer described in table one should also be further reduced by also considering the permanent shell deformation of the conduit. This would have to be done by a separate analysis.

Horizontal Impact by a Swinging Mass

To obtain the permissible distance between a swinging mass and a duct, pipe, conduit or structural member, two checks must be made:

i) Judge the trajectory of the swinging mass to see whether it impacts the target.

ii) If so, check whether the distance is permissible according to the formula dericed here.

The formula derived here represents the energy loss on impact by the swinging object (which gets transmitted to the target). If the energy capacity of the target is known, as given in other sections of this Appendix then the permissible distance to target for a given weight of missile can be determined.

Derivation

If a pendulum impacts a target member, let δ_2 be distance from pendulum to the member. Let δ_1 be distance of swing of pendulum if impacted.

I We calculate δ_1 the following way. Natural frequency of pendulum,

$$w_p = \sqrt{\frac{g}{\ell}}$$

From the response spectra, accln of response corresponding to w_p = A

$$A = w_p^2 \delta_1$$

$$\therefore \delta_1 = \frac{A}{w_p^2}$$

II No of impacts

$$= \frac{\text{Duration of Seismic event}}{\text{period of pendulum}} = 10 \text{ Sec}$$

$$= \frac{10}{2\pi}\sqrt{g/\ell} \quad = \quad \frac{5}{\pi}\sqrt{g/\ell}$$

III Energy of pendulum without impact

$$= \frac{mV_1^2}{2} \quad = \quad \frac{m}{2} \quad (\delta_1 wp)^2$$

$$= \quad \frac{m}{2} \quad \left(\frac{A}{w_p}\right)^2$$

IV Energy of pendulum if swing is restricted to δ_2 by impact (Assume no rebound-conservative)

$$= \frac{mv_2^2}{2} \quad = \quad \frac{m}{2} \quad (\delta_2 \ w_p)^2$$

V Energy change $= \frac{m}{2}\left[\left(\frac{A}{wp}\right)^2 - (w_p \ \delta_2)^2\right]$

$$\Delta E \quad = \frac{W}{2g}\left[\left(\frac{A}{wp}\right)^2 - (w_p \delta 2)^2\right]$$

VI Total energy lost in impacts

$= \Delta E$. No of impacts

$$= \frac{5}{\pi} \sec \sqrt{g/\ell} \quad \cdot \quad \frac{w}{2g}\left[\left(\frac{A}{wp}\right)^2 - (wp\delta 2)^2\right]$$

$$= \frac{5}{2\pi\sqrt{g\ell}}\left[\left(\frac{A}{wp}\right)^2 - (W_p\ \delta_2)^2\right]$$

VII Equate the above to energy capacity in pure bonding of the structural member.

$$E_{bending} = \frac{5}{2\pi}\ \frac{\sec W}{\sqrt{g\ell}}\left[\left(\frac{A}{w_p}\right)^2 - (w_p\ \delta_2)^2\right]$$

$$= \frac{5 \text{ Sec } W}{2\pi\sqrt{g\ell}}\left[A^2\frac{\ell}{g} - \delta_2^2\ \frac{g}{\ell}\right]$$

$$= \frac{5W}{2\pi}\sqrt{\frac{g}{\ell}}\left(\left(\frac{A}{g}\right)^2 \ell - \frac{\delta_2^2}{\ell}\right) = \frac{5W\sqrt{g\ell}}{2\pi}\left[\left(\frac{A}{g}\right)^2 - \left(\frac{\delta_2}{\ell}\right)^2\right]$$

VIII. From this given ℓ, A, W & Ebending find permissible value of Δ_2

EXAMPLE

$L = 60''$ $\qquad$ $E_b = 6990$ ℓb inch $A = 3G$

δ_2 inch	W lbs
5	6.41
10	6.43
15	6.45
20	6.49
30	6.59
40	6.74
50	6.95
60	7.21

At $\delta_2 = 60$,
error in small angle
approximation is about 11%
(sin 45 = .707, 45 = .78 radians)

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APPENDIX E

GOOD SUPPORTS
SCREEN LEVELS 2 AND 4

APPENDIX E

TRAIN "C" CONDUIT

2" DIAMETER AND UNDER

CRITERIA

FOR

SCREEN LEVEL 1

SCREEN LEVEL 2 AND SCREEN LEVEL 4

FOR THE

WALKDOWN TEAM

REVISION 0

MARCH 3, 1986

PREPARED FOR

TEXAS UTILITIES GENERATING COMPANY

P.O.BOX 1002

GLEN ROSE, TEXAS 76043

PREPARED BY

GIBBS & HILL INC.

11 PENN PLAZA

NEW YORK, N.Y. 10001

SCREEN LEVEL 1
WEIGHT CHECK

BASED ON THE ANALYSES AND EVALUATIONS PERFORMED ON THE 126 RANDOM SAMPLES AND 131 ENGINEERING SAMPLES ,IT IS FOUND THAT THE FOLLOWING TYPES OF SYSTEMS AND SUPPORTS ARE 'GOOD'.

SYSTEMS	SUPPORT TYPES	APPLICABLE SEISMIC ZONE
ONE CONDUIT SYSTEM	ALL	I to V
MULTI-CONDUIT SYSTEM-TOTAL CODUIT UNIT WEIGHT EQUAL TO OR LESS THAN 6 #/ft AND A MAXIMUM SUPPORT SPAN OF 10 ft	ALL	I to V

SCREEN LEVEL 2

BASES FOR SCREENING THE 'GOOD' SUPPORT TYPES

- 'GOOD' SUPPORTS FROM THE RANDOM AND ENGINEERING SAMPLE EVALUATIONS BASED ON THE ' OLD ' BUT CONSERVATIVE CRITERIA.

    - ELASTIC ANALYSIS
    - STRESS =0.9 F_y
      F_y=36 KSI FOR STRUCTURAL STEEL SHAPES
      F_y=33 KSI FOR UNISTRUT SHAPES
    - ANCHOR BOLTS FACTOR OF SAFETY ≥ 3.
    - 7% DAMPING
    - ENGINEERING SAMPLES = 131 SAMPLE RUNS
      WITH A TOTAL OF 1186 SUPPORTS
      NO. OF 'GOOD' SUPPORTS = 1091.
    - RANDOM SAMPLES = 126 SAMPLE RUNS
      WITH A TOTAL OF 1227 SUPPORTS
      NO.OF 'GOOD' SUPPORTS = 1089.
    - TOTAL NO. OF 'GOOD' SUPPORTS FROM BOTH SAMPLES = 2180
      (OUT OF A TOTAL OF 2413 SUPPORTS 90.3%)

- USE THE 'REFINED' CRITERIA INSTEAD OF THE 'OLD' CRITERIA TO MAXIMIZE 'GOOD' SUPPORT SCREENING.

    - TOTAL NUMBER OF 'GOOD' SUPPORTS INCREASE TO 2370
      (OUT OF A TOTAL OF 2413 SUPPORT 98.2 %).

"REFINED" CRITERIA

A. ELASTIC ANALYSIS

1. SEVEN PERCENT DAMPING

2. EQUIVALENT STATIC METHOD USING DYNAMIC AMPLIFICATION FACTOR (DAF) = 1.5/1.1 TIMES PEAK ACCELERATION OR RESPONSE SPECTRA METHOD.

3. FACTOR OF SAFETY FOR HILTI ≥ 3

4. STRESS LIMIT

   F_y FOR STRUCTURAL STEEL SHAPES

   F_{ya} FOR UNISTRUT SHAPES (STRENGTH INCREASE FROM COLD WORK OF FORMING INCLUDED)

5. FIVE (5) SEISMIC ZONES

B. FATIGUE ANALYSIS

1. STRUCTURAL MEMBER AND/OR COMPONENT

   FACTOR OF SAFETY ≥1.5 (FATIGUE REQUIREMENT)

   MAXIMUM ALLOWABLE LOAD (CAPACITY) = 2/3 ULTIMATE LOAD (CAPACITY)

   DISPLACEMENT DUCTILITY LIMIT = 3

SEISMIC ZONES

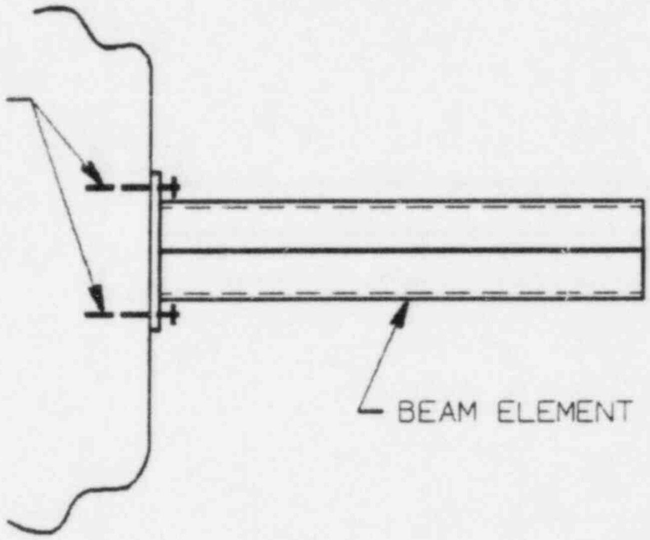
SEISMIC ZONE	ELEVATION	BUILDING	gv	gh
I	ABOVE EL. 860'-0"	ALL	2.219	2.717
II	ABOVE EL. 841'-0" BELOW EL.860'-0"	ALL	2.127	1.778
III	ABOVE EL. 825'-0" BELOW EL. 841'-0"	ALL	2.023	1.171
IV	ABOVE EL. 784'-0" BELOW EL. 825'-0"	ALL	1.877	1.10
V	BELOW EL. 784'-0"	ALL	1.685	0.649

gv = VERTICAL PEAK SEISMIC ACCELERATION
gh = HORIZONTAL PEAK SEISMIC ACCELERATION

GROUPING OF CONDUIT SUPPORT TYPES USED

GROUP NO.	SUPPORT TYPE
1a	CANTILEVER WELDED FOOT HANGER.
1b	CANTILEVER WELDED FOOT MEMBER WITH UNISTRUT HEADER.
1c	CANTILEVER MEMBER WITH UNISTRUT HEADER AND P1026 OR SIMILAR FITTINGS.
2a	DOUBLE CANTILEVER SUPPORTS USING WELDED FOOT MEMBER.
2b	DOUBLE CANTILEVER SUPPORTS WITH UNISTRUT HEADER USING WELDED FOOT MEMBERS.
3a	TRIPLE CANTILEVER SUPPORTS WITH UNISTRUT HEADER USING WELDED FOOT MEMBERS.
3b	TRIPLE CANTILEVER SUPPORTS WITH UNISTRUT HEADER USING UNISTRUT FITTINGS.
4a	ROD HANGER TRAPEZE SUPPORTS ATTACHED TO CEILING OR UNDERSIDE OF BEAM.
4b	ROD HANGER TRAPEZE SUPPORTS ATTACHED TO SIDE OF BEAM USING P1026 CONNECTION.
5	ONE HOLE PIPE STRAP.
6	TWO HOLE CLAMP.
7	SPECIAL SUPPORTS
8	MULTIPLE-TIERED GANG SUPPORTS WITH ROD HANGERS.

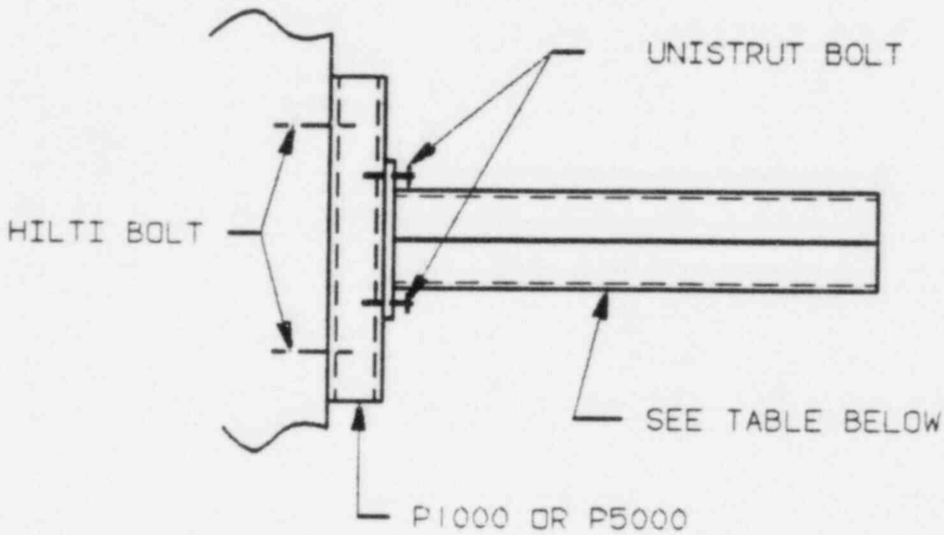
CANTILEVER WELDED FOOT HANGER - SUPPORT TYPE 1a



THE SUPPORTS MEETING THE FOLLOWING CONDITIONS ARE "GOOD" SUPPORTS.

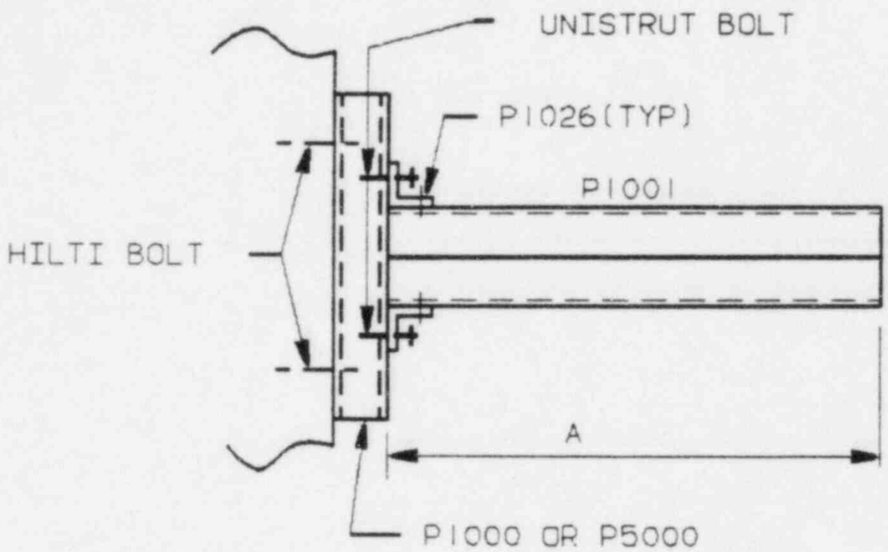
SUPPORT BEAM ELEMENT	ANCHORAGE		APPLICABLE SEISMIC ZONE	REMARKS
	BOLT SIZE	EMB.		
P2542	3/8"	≥ 2"	I to V	
P2543	3/8"	≥ 2"	III to V	
P2544	3/8"	≥ 2"	IV & V	
P2545 THRU P2546	3/8"	≥ 4"	IV & V	
P2542 THRU P2544	1/2"	≥ 3 1/2"	I to V	
P2545 THRU P2546	1/2"	≥ 5 1/2"	III to V	

CANTILEVER WELDED FOOT MEMBER WITH UNISTRUT HEADER - SUPPORT TYPE 1b



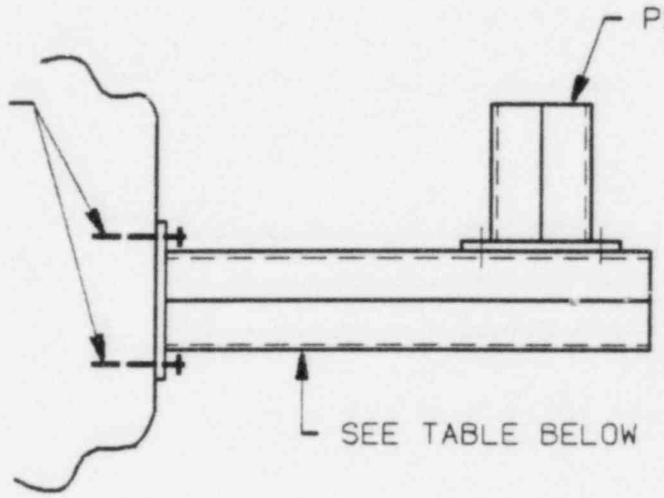
ANY CANTILEVER WELDED FOOT MEMBER WITH UNISTUT HEADER IS A GOOD SUPPORT.

CANTILEVER MEMBER WITH UNISTRUT HEADER AND P1026 OR SIMILAR FITTINGS - SUPPORT TYPE (1C)



ANY CANTILEVER MEMBER WITH UNISTRUT HEADER AND P1026 OR SIMILAR FITTINGS IS A GOOD SUPPORT.

DOUBLE CANTILEVER SUPPORTS USING WELDED FOOT MEMBERS - SUPPORT TYPE 2a



THE SUPPORTS MEETING THE FOLLOWING CONDITIONS ARE "GOOD" SUPPORTS.

SUPPORT BEAM ELEMENT	ANCHORAGE		APPLICABLE SEISMIC ZONE	REMARKS
	BOLT SIZE	EMB.		
P2542	3/8"	≥ 2"	I to V	
P2543	3/8"	≥ 2"	III to V	
P2544	3/8"	≥ 2"	IV & V	
P2545 THRU P2546	3/8"	≥ 4"	IV & V	
P2542 THRU P2546	1/2"	≥ 3 1/2"	I to V	

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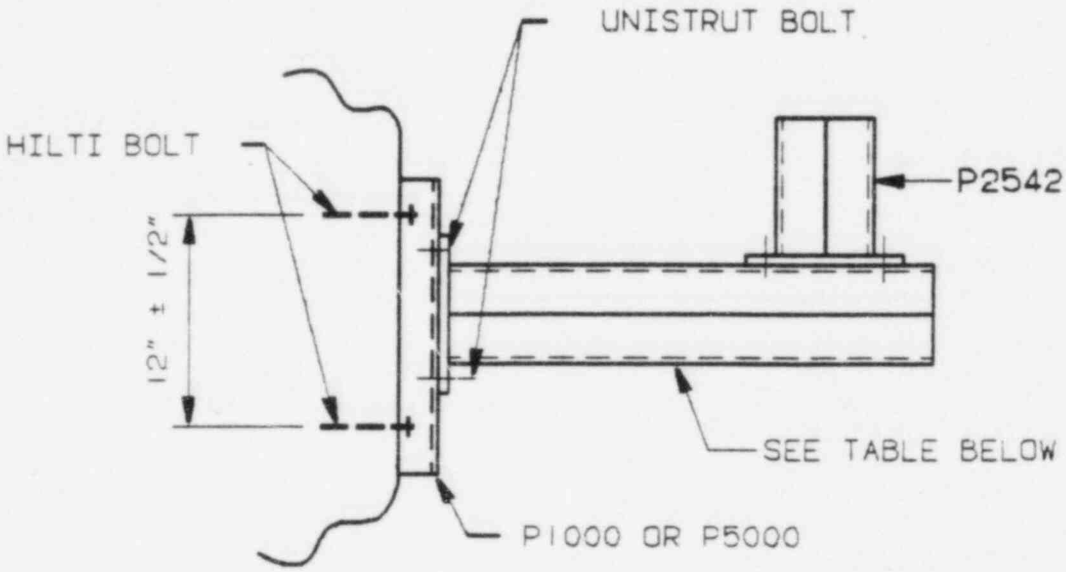
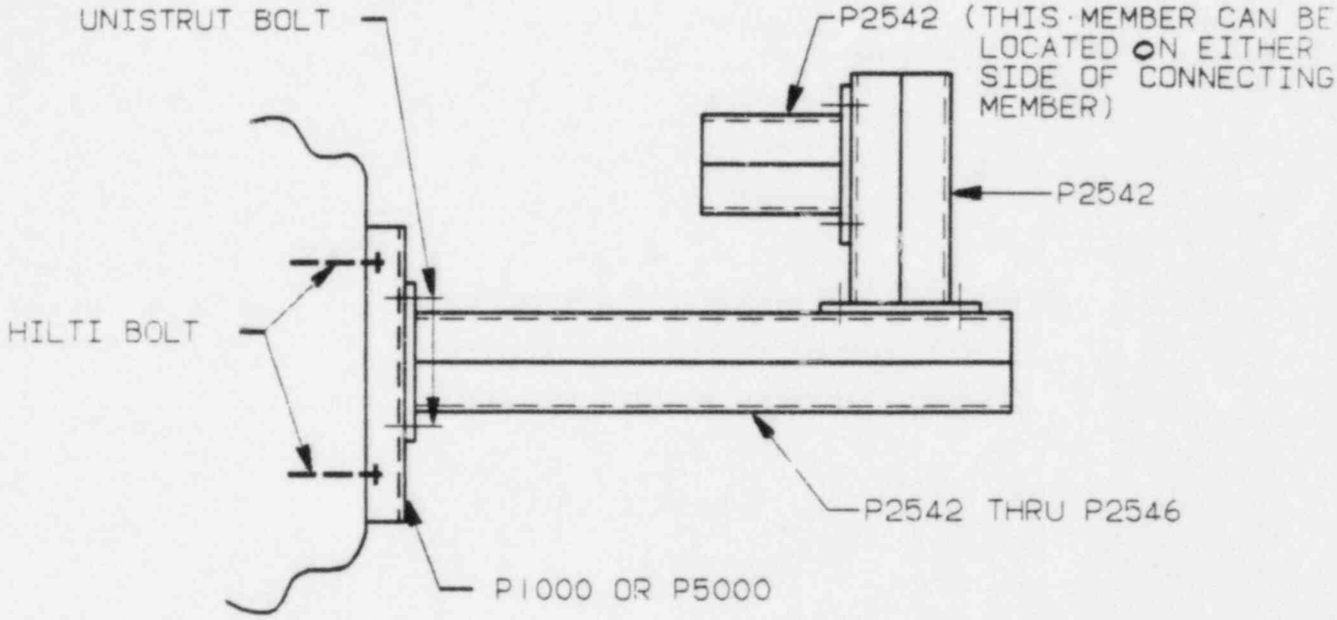
DOUBLE CANTILEVER SUPPORTS WITH UNISTRUT HEADER USING WELDED FOOT MEMBERS - SUPPORT TYPE (2b)



ANY DOUBLE CANTILEVER SUPPORT WITH UNISTRUT HEADER USING WELDED FOOT MEMBERS IS A GOOD SUPPORT.

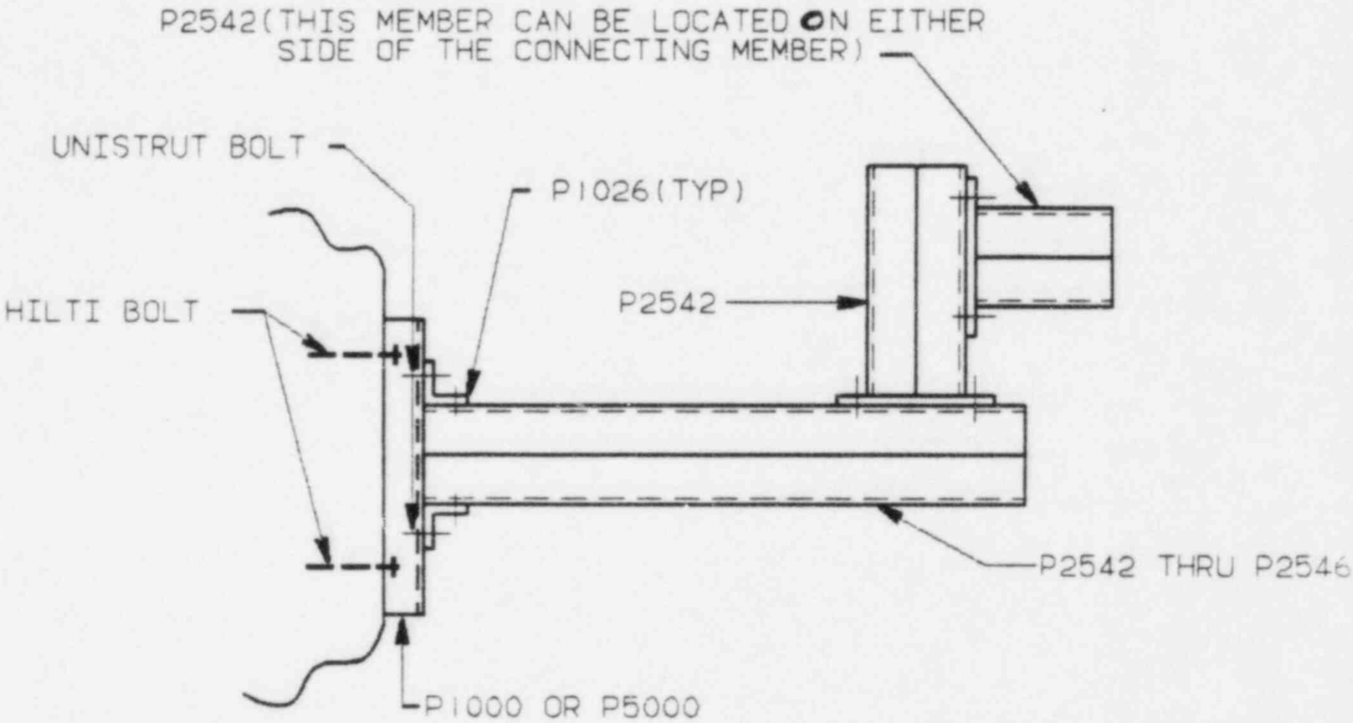
TRIPLE CANTILEVER SUPPORTS WITH UNISTRUT HEADER USING WELDED FOOT MEMBERS - SUPPORT TYPE 3a



SUPPORT CAPACITIES ARE GIVEN IN SCREEN LEVEL 4.

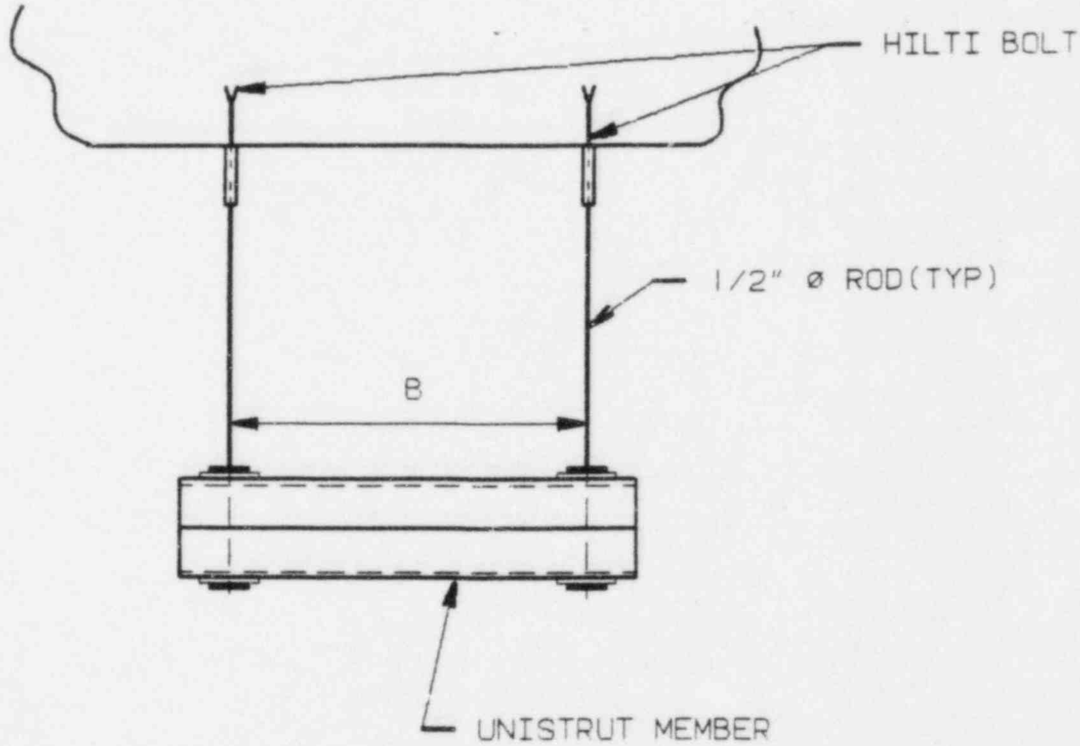
TRIPLE CANTILEVER SUPPORTS WITH UNISTRUT HEADER USING

UNISTRUT FITTINGS - SUPPORT TYPE 3b



SUPPORT CAPACITIES ARE GIVEN IN SCREEN LEVEL 4.

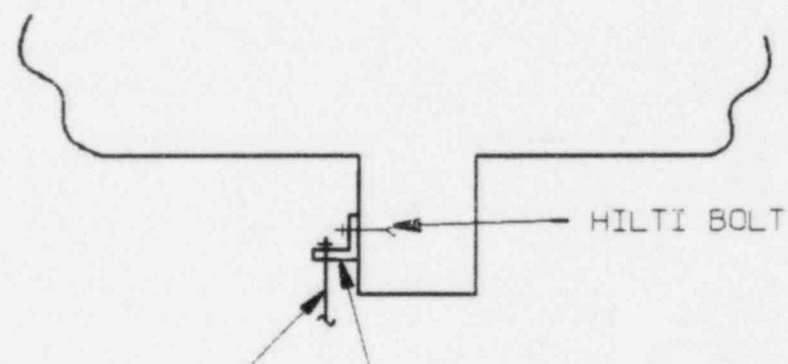
ROD HANGER TRAPEZE SUPPORTS ATTACHED TO CFILING OR UNDERSIDE OF BEAM - SUPPORT TYPE 4a



ANY ROD HANGER TRAPEZE SUPPORT ATTACHED TO CEILING OR UNDERSIDE OF THE BEAM IS A GOOD SUPPORT.

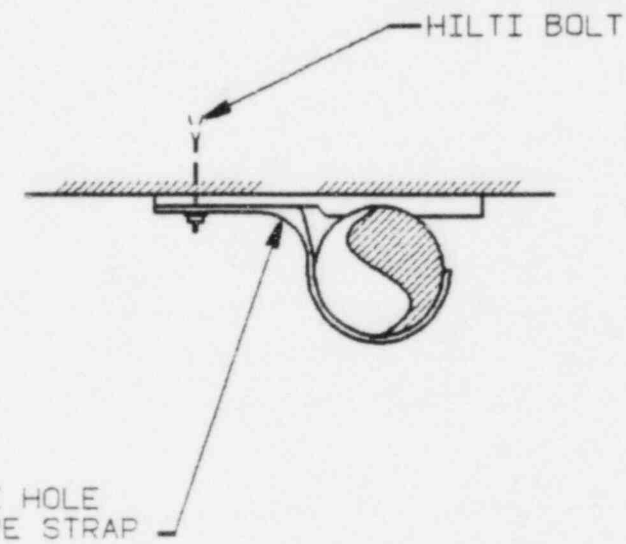
ROD HANGER TRAPEZE SUPPORTS ATTACHED TO SIDE OF BEAM

USING P1026 CONNECTION - SUPPORT TYPE 4b



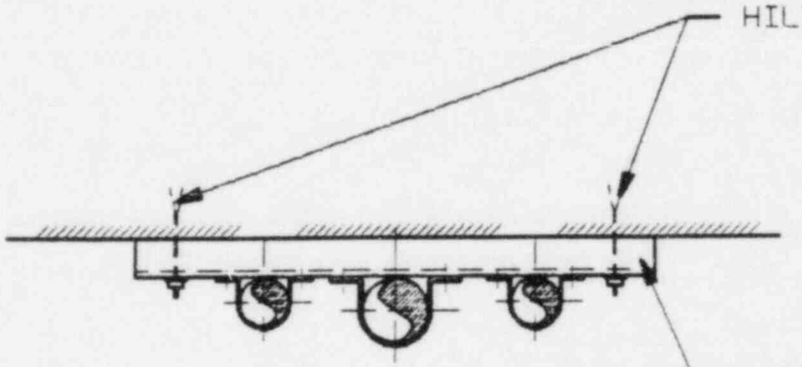
ANY ROD HANGER TRAPEZE SUPPORTS ATTACHED TO SIDE OF BEAM USING P1026 CONNECTION IS A GOOD SUPPORT.

ONE HOLE PIPE STRAP - SUPPORT TYPE 5



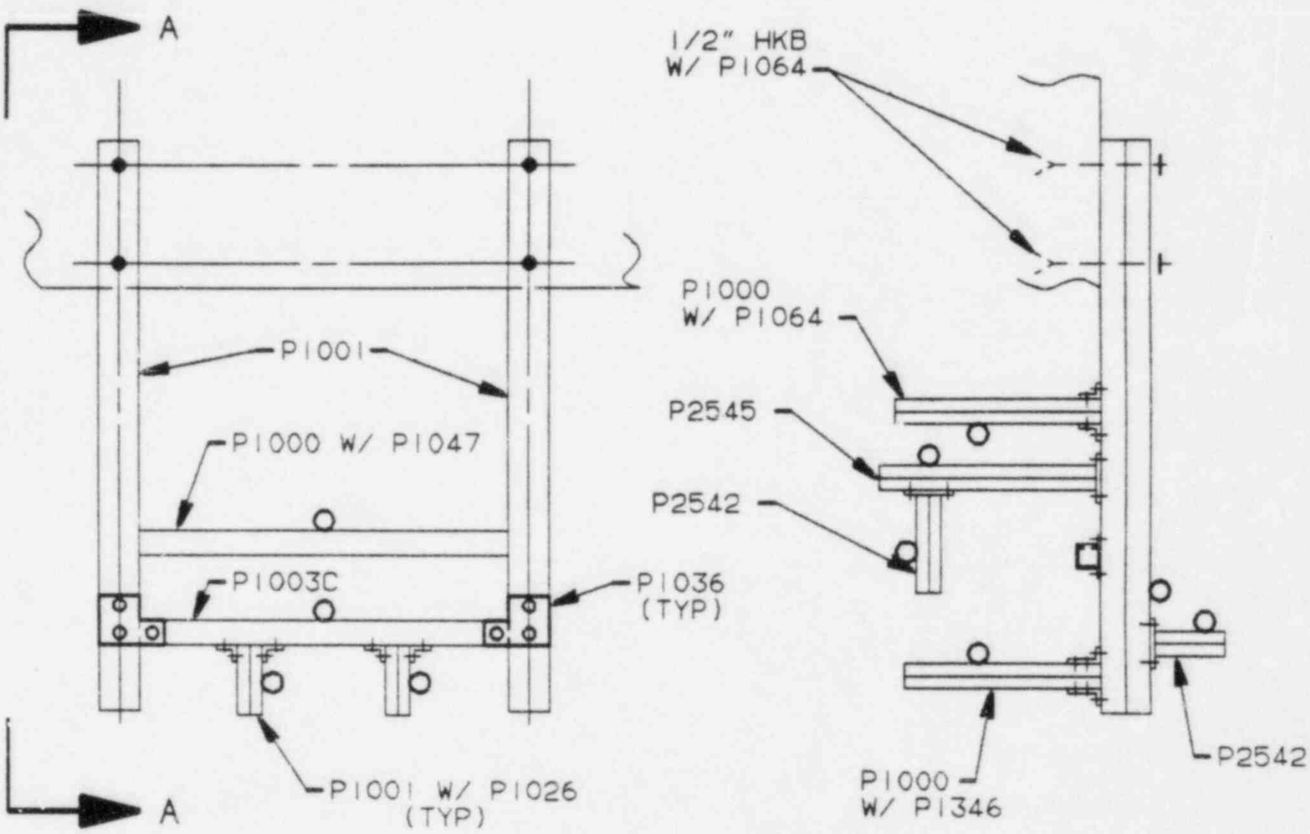
ANY ONE HOLE PIPE STRAP SUPPORT IS A 'GOOD' SUPPORT.

TWO HOLE CLAMP - SUPPORT TYPE (6)



ANY TWO HOLE CLAMP SUPPORT IS A 'GOOD' SUPPORT.

SPECIAL SUPPORTS - SUPPORT TYPE 7

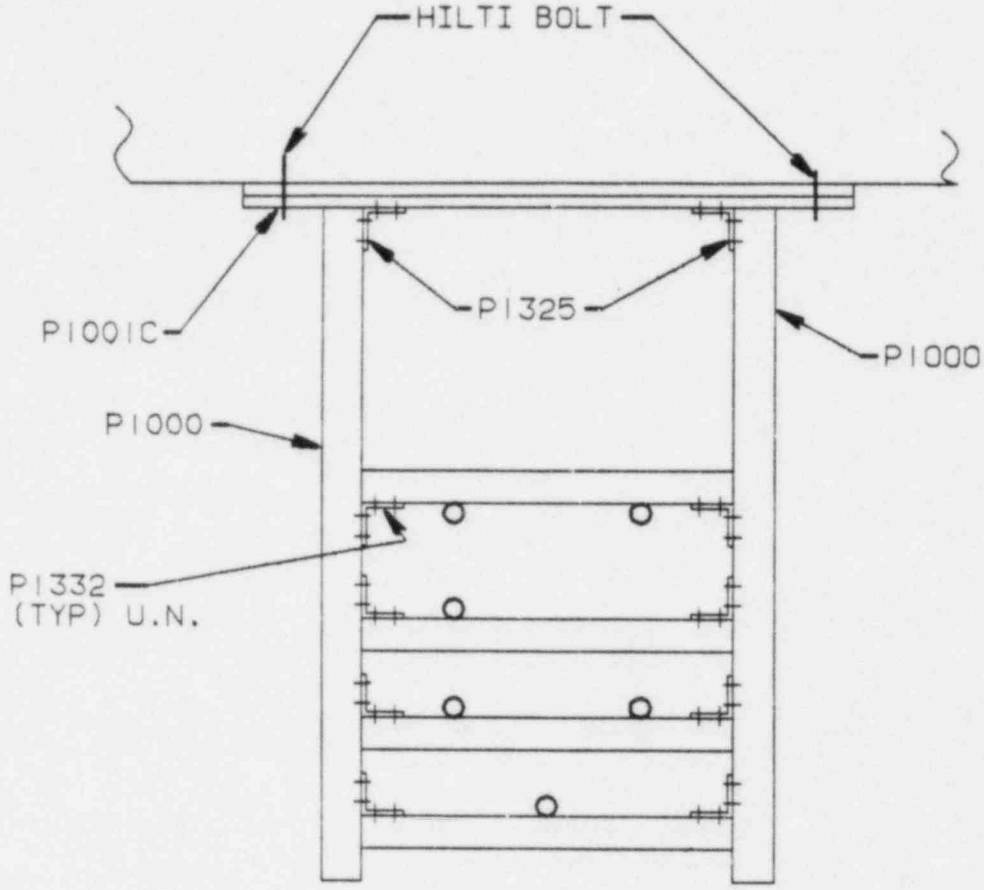


ELEVATION LOOKING NORTH

SECTION A - A
LOOKING EAST

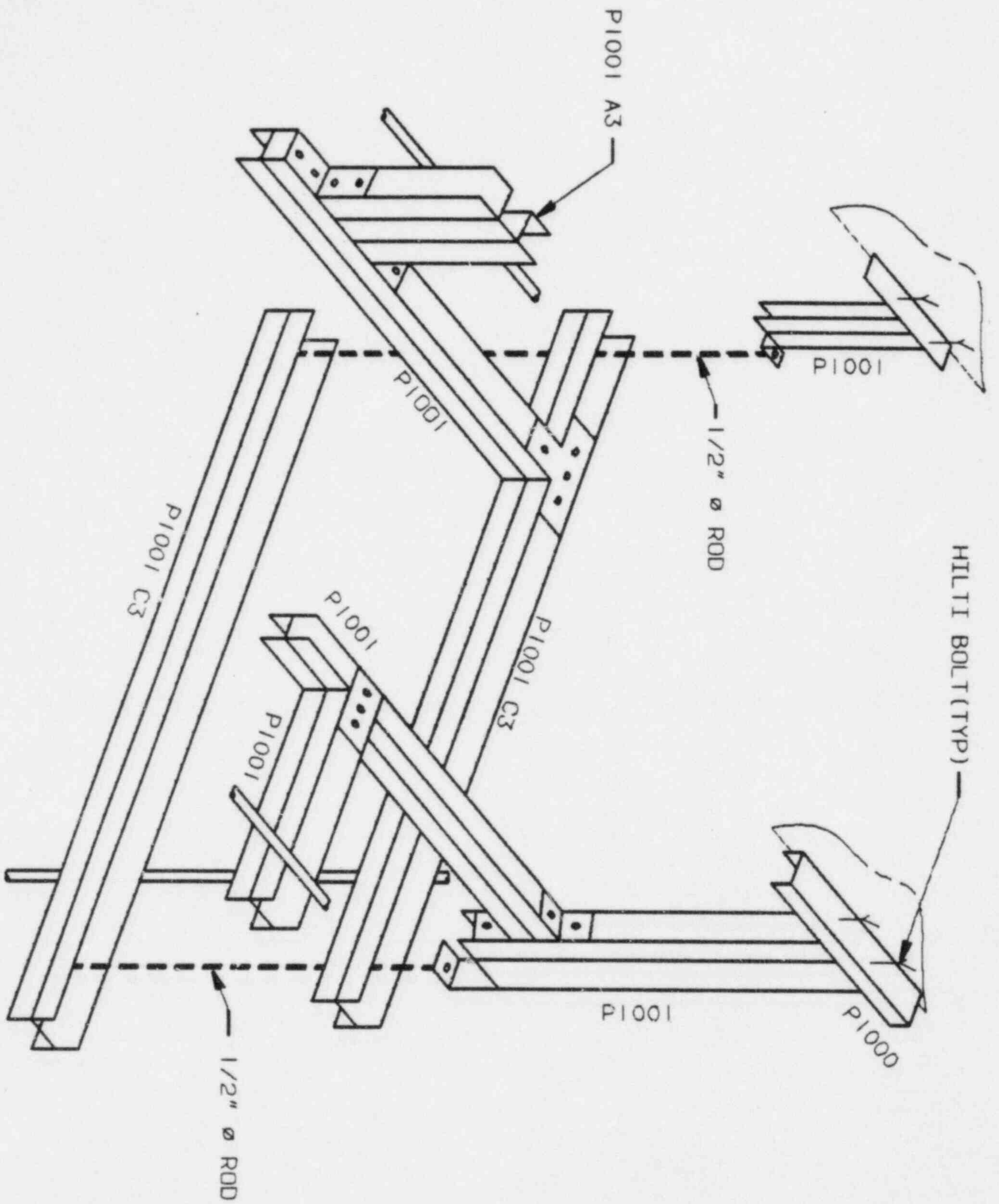
EVALUATE SUPPORT ON A CASE BY CASE BASIS.

SPECIAL SUPPORTS - SUPPORT TYPE 7



EVALUATE SUPPORT ON A CASE BY CASE BASIS.

MULTIPLE-TIERED GANG SUPPORTS WITH ROD HANGERS - SUPPORT TYPE (8)



ANY MULTIPLE-TIERED GANG SUPPORT WITH ROD HANGERS
IS 'GOOD' SUPPORT.

R323.P323
SUF...1

SCREEN LEVEL 4

SUPPORT CAPACITY CHECK

- DETERMINE CONDUIT SUPPORT TYPES

- DETERMINE SUPPORT CAPACITY BASED
  ON 'REFINED' CRITERIA OF SCREEN LEVEL 2.

- DETERMINE SUPPORT LOADS

- COMPARE SUPPORT CAPACITY VS SUPPORT LOAD
  IF SUPPORT CAPACITY > SUPPORT LOAD - SUPPORT ADEQUATE
  IF SUPPORT CAPACITY < SUPPORT LOAD - SUPPORT OVERSTRESSED -
  APPLY OTHER APPLICABLE SCREEN LEVELS OR REWORK

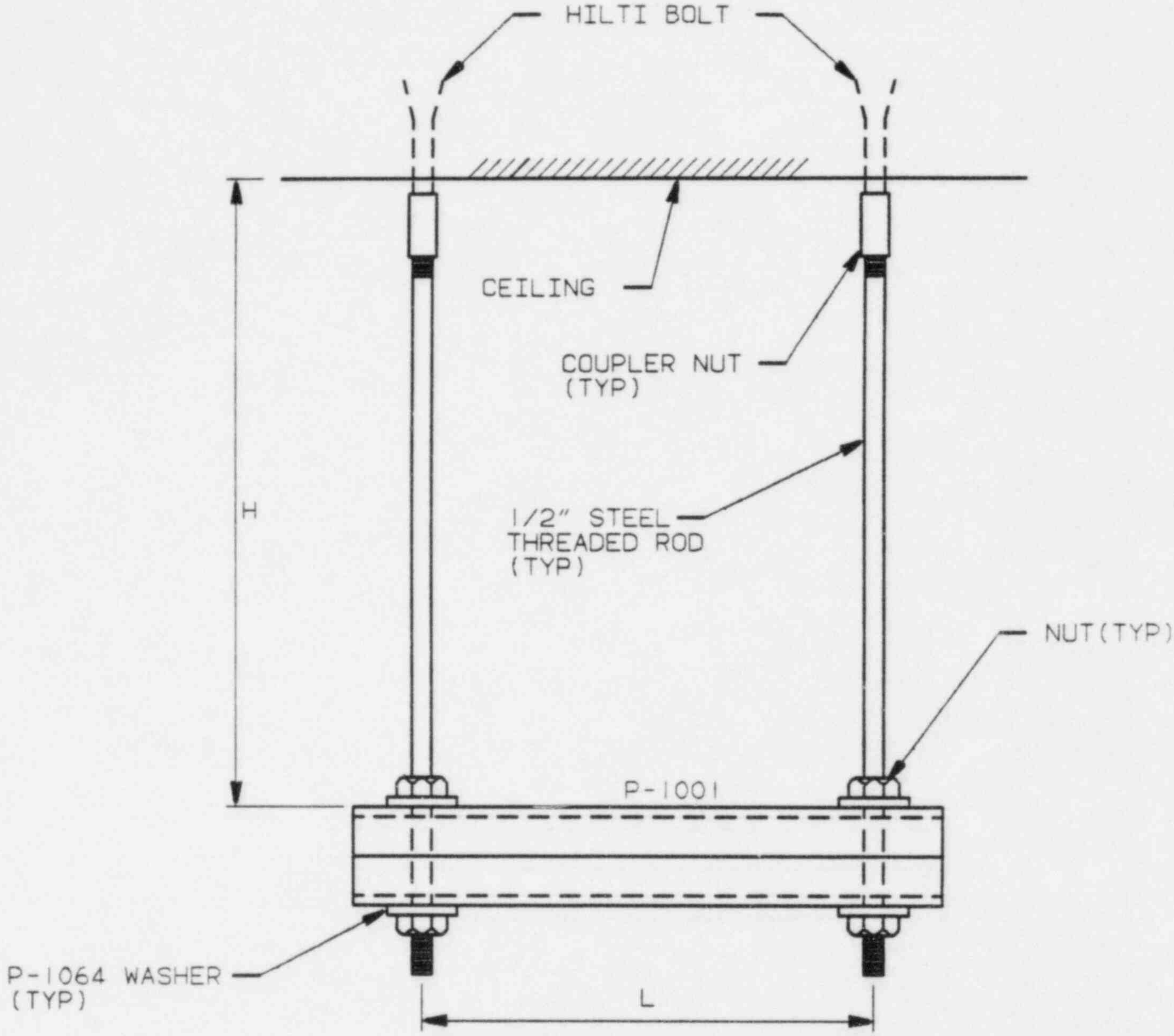
CONDUIT SUPPORT TYPES

THE SUPPORTS INSTALLED IN THE PLANT CAN BE DIVIDED INTO THE FOLLOWING THREE BASIC SUPPORT TYPES:

A. VERTICAL SUPPORTS - HAVE SUPPORTING CAPABILITY IN THE VERTICAL DIRECTION ONLY.

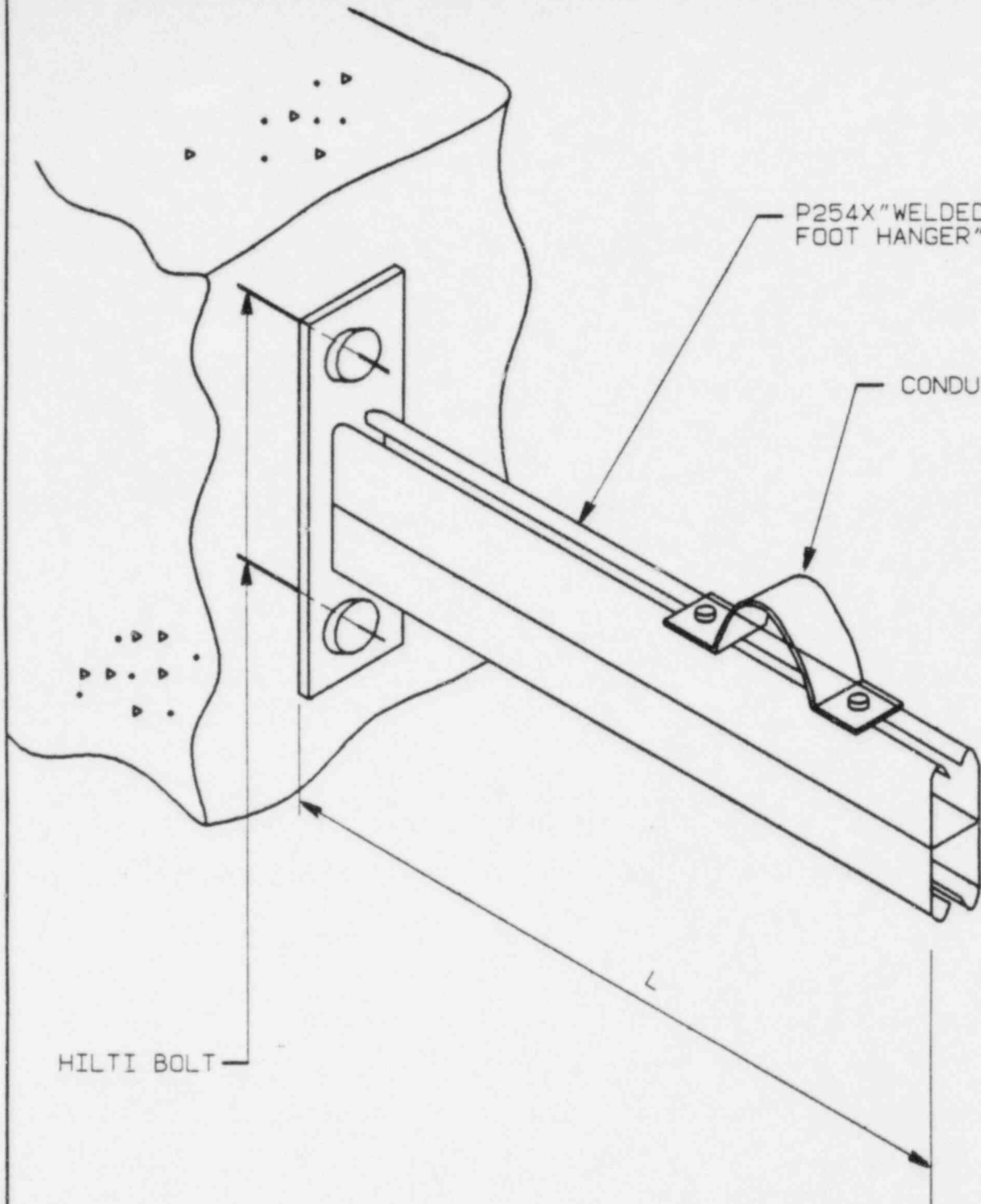
B. VERTICAL / TRANVERSE SUPPORTS - HAVE SUPPORTING CAPABILITY IN THE VERTICAL AND TRANSVERSE DIRECTIONS.

C. MULTI-DIRECTIONAL SUPPORTS - HAVE SUPPORTING CAPABILITY IN ALL THREE ORTHOGONAL DIRECTIONS.

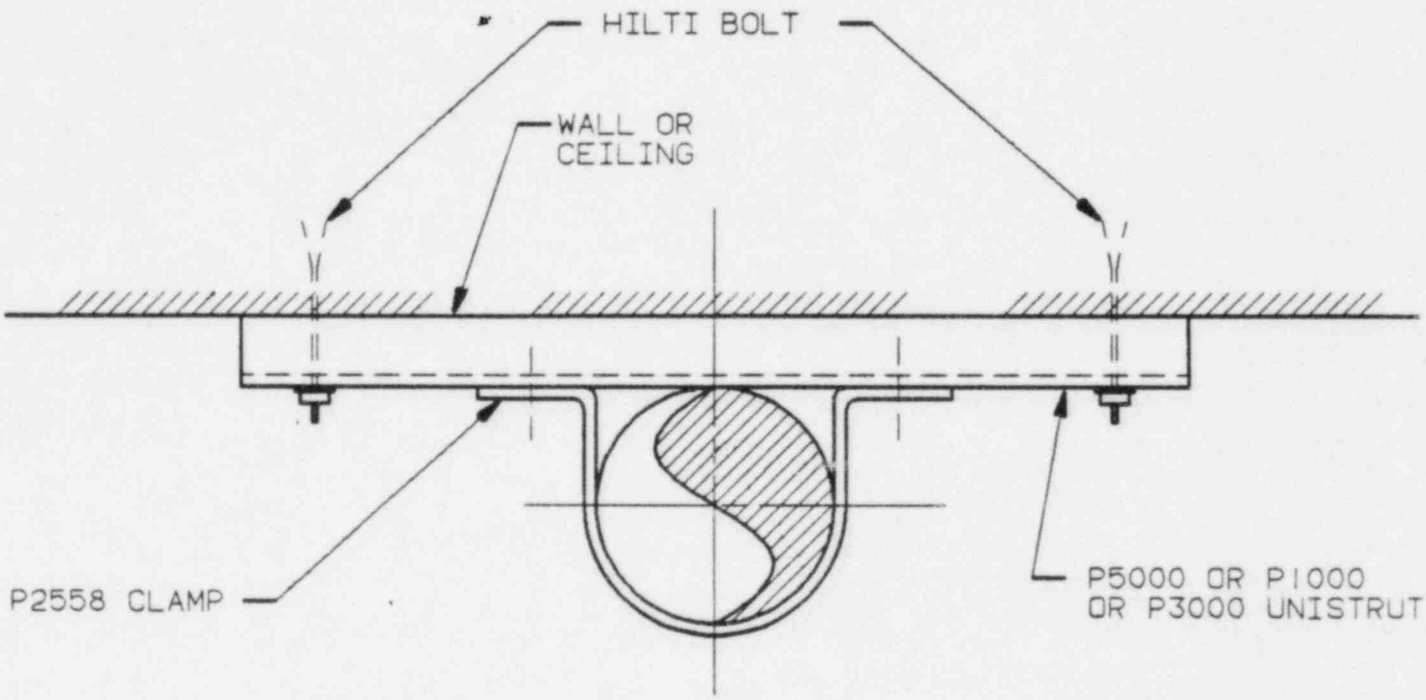


VERTICAL SUPPORT

B323.2323
C.SUPP.8



VERTICAL / TRANVERSE SUPPORT



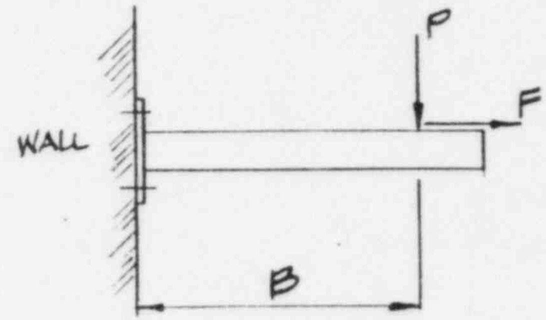
MULTI-DIRECTIONAL SUPPORT

B323.2323
C.SUPP.9

SUPPORT CAPACITY FOR
SUPPORT TYPE 1-A ZONE I

WALL MOUNTED SINGLE CANT. WELD. FT. HGR.
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B (INCH)	P (LB)	F (LB)
12	20	170
12	30	160
12	40	140
12	50	120
12	60	95
12	70	45
15	20	160
15	30	140
15	40	110
15	50	70
18	20	140
18	30	110
18	40	55
21	20	115
21	30	65
24	20	80



ELEVATION

SUPPORT CAPACITY FOR
SUPPORT TYPE 1-A · ZONE II

WALL MOUNTED SINGLE CANT. WELD. FT. HGR.
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B (INCH)	P (LB)	F (LB)
12	20	270
12	30	250
12	40	230
12	50	200
12	60	160
12	70	105
15	20	255
15	30	230
15	40	190
15	50	140
15	60	15
18	20	235
18	30	195
18	40	135
21	20	210
21	30	150
24	20	180
24	30	70
27	20	130
30	20	25

S U P P O R T   C A P A C I T Y   F O R
SUPPORT TYPE 1-A ZONE III

WALL MOUNTED SINGLE CANT. WELD. FT. HGR.
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B (INCH)	P (LB)	F (LB)
12	20	415
12	30	390
12	40	360
12	50	320
12	60	270
12	70	200
12	80	70
15	20	400
15	30	360
15	40	310
15	50	240
15	60	125
18	20	375
18	30	320
18	40	240
18	50	85
21	20	345
21	30	265
21	40	120
24	20	310
24	30	190
27	20	265
30	20	195
33	20	70

SUPPORT CAPACITY FOR
SUPPORT TYPE 1-A ZONE IV

WALL MOUNTED SINGLE CANT. WELD. FT. HGR.
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B (INCH)	P (LB)	F (LB)
12	20	445
12	30	420
12	40	390
12	50	350
12	60	305
12	70	245
12	80	150
15	20	430
15	30	390
15	40	340
15	50	280
15	60	185
18	20	405
18	30	350
18	40	280
18	50	160
21	20	380
21	30	305
21	40	180
24	20	345
24	30	235
27	20	300
27	30	125
30	20	240
33	20	155

S U P P O R T   C A P A C I T Y   F O R
SUPPORT TYPE 1-A ZONE V

WALL MOUNTED SINGLE CANT. WELD. FT. HGR.
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

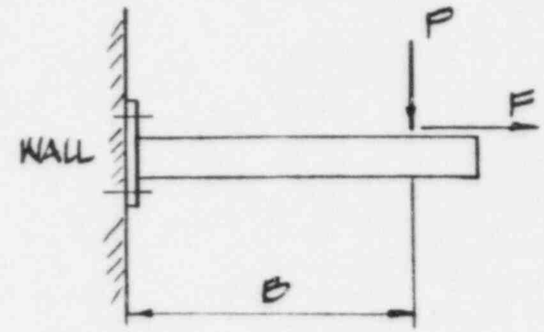
B (INCH)	P (LB)	F (LB)
12	20	770
12	30	730
12	40	680
12	50	625
12	60	560
12	70	480
12	80	370
12	90	200
15	20	745
15	30	685
15	40	615
15	50	530
15	60	410
15	70	220
18	20	710
18	30	630
18	40	530
18	50	385
18	60	65
21	20	675
21	30	565
21	40	415
21	50	70
24	20	630
24	30	485
24	40	225
27	20	575
27	30	375
30	20	510
30	30	185

33	20	425
36	20	310

SUPPORT CAPACITY FOR
SUPPORT TYPE 1-A ZONE I

WALL MOUNTED SINGLE CANT. WELD. FT. HGR.
3/8"HKB-WITH/2" EMBEDMENT
VERTICAL CONDUIT RUN

B (INCH)	P (LB)	F (LB)
12	20	175
12	30	165
12	40	150
12	50	130
12	60	105
12	70	60
15	20	160
15	30	145
15	40	120
15	50	80
18	20	145
18	30	115
18	40	60
21	20	120
21	30	65
24	20	80



PLAN

SUPPORT CAPACITY FOR
SUPPORT TYPE 1-A ZONE II

WALL MOUNTED SINGLE CANT. WELD. FT. HGR.
3/8"HKB-WITH/2" EMBEDMENT
VERTICAL CONDUIT RUN

B (INCH)	P (LB)	F (LB)
12	20	280
12	30	270
12	40	265
12	50	255
12	60	240
12	70	225
12	80	205
12	90	175
12	100	145
15	20	265
15	30	255
15	40	240
15	50	225
15	60	200
15	70	165
15	80	120
15	90	10
18	20	245
18	30	230
18	40	210
18	50	180
18	60	130
18	70	30
21	20	225
21	30	200
21	40	165
21	50	100
24	20	190
24	30	155
24	40	80
27	20	145
27	30	65

S U P P O R T   C A P A C I T Y   F O R
SUPPORT TYPE 1-A   ZONE III

WALL MOUNTED SINGLE CANT. WELD. FT. HGR.
3/8"HKB-WITH/2" EMBEDMENT
VERTICAL CONDUIT RUN

B (INCH)	P (LB)	F (LB)
12	20	430
12	30	425
12	40	420
12	50	415
12	60	405
12	70	395
12	80	385
12	90	375
12	100	360
12	120	320
12	140	270
12	160	200
12	180	50
15	20	415
15	30	410
15	40	400
15	50	390
15	60	375
15	70	360
15	80	340
15	90	315
15	100	290
15	120	210
18	20	395
18	30	385
18	40	370
18	50	355
18	60	335
18	70	305
18	80	270
18	90	230
18	100	165
21	20	370
21	30	355
21	40	335
21	50	305
21	60	270
21	70	225
21	80	155

24	20	335
24	30	310
24	40	280
24	50	240
24	60	175
24	70	35
27	20	285
27	30	255
27	40	205
27	50	115
30	20	215
30	30	160
33	20	80

SUPPORT CAPACITY FOR
SUPPORT TYPE 1-A ZONE IV

WALL MOUNTED SINGLE CANT. WELD. FT. HGR.
3/8"HKB-WITH/2" EMBEDMENT
VERTICAL CONDUIT RUN

B (INCH)	P (LB)	F (LB)
12	20	460
12	30	455
12	40	450
12	50	445
12	60	440
12	70	430
12	80	420
12	90	405
12	100	395
12	120	360
12	140	315
12	160	260
12	180	170
15	20	445
15	30	440
15	40	430
15	50	420
15	60	410
15	70	395
15	80	375
15	90	355
15	100	330
15	120	265
15	140	155
18	20	425
18	30	415
18	40	405
18	50	390
18	60	370
18	70	345
18	80	315
18	90	275
18	100	230
21	20	400
21	30	385
21	40	370
21	50	345
21	60	315
21	70	275

21	80	220
21	90	135
24	20	365
24	30	345
24	40	320
24	50	280
24	60	230
24	70	150
27	20	320
27	30	295
27	40	250
27	50	185
27	60	50
30	20	260
30	30	215
30	40	130
33	20	160
33	30	10

S U P P O R T   C A P A C I T Y   F O R
SUPPORT TYPE 1-A ZONE V

WALL MOUNTED SINGLE CANT. WELD. FT. HGR.
3/8"HKB-WITH/2" EMBEDMENT
VERTICAL CONDUIT RUN

B (INCH)	P (LB)	F (LB)
12	20	795
12	30	790
12	40	790
12	50	785
12	60	780
12	70	775
12	80	770
12	90	765
12	100	755
12	120	740
12	140	720
12	160	695
12	180	665
12	200	635
12	240	550
12	280	430
12	320	230
15	20	770
15	30	770
15	40	765
15	50	760
15	60	750
15	70	745
15	80	735
15	90	725
15	100	710
15	120	680
15	140	650
15	160	605
15	180	555
15	200	490
15	240	295
18	20	745
18	30	740
18	40	730
18	50	725
18	60	715
18	70	700
18	80	685
18	90	670
18	100	650
18	120	605

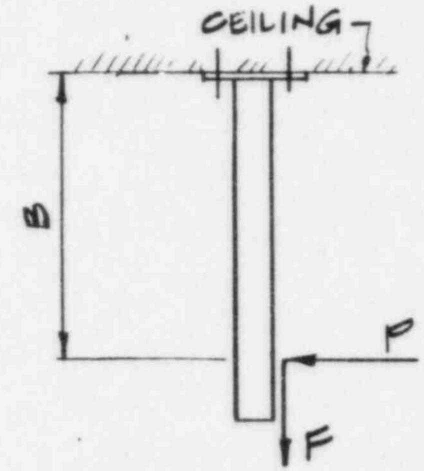
18	140	550
18	160	475
18	180	370
18	200	210
21	20	710
21	30	700
21	40	690
21	50	680
21	60	665
21	70	645
21	80	625
21	90	600
21	100	570
21	120	500
21	140	395
21	160	235
24	20	665
24	30	655
24	40	640
24	50	625
24	60	600
24	70	575
24	80	540
24	90	505
24	100	460
24	120	330
27	20	610
27	30	595
27	40	575
27	50	550
27	60	515
27	70	475
27	80	425
27	90	365
27	100	275
30	20	535
30	30	515
30	40	485
30	50	445
30	60	395
30	70	330
30	80	230
33	20	435
33	30	400
33	40	355
33	50	290

33	60	180
36	20	270
36	30	205
36	40	50

SUPPORT CAPACITY FOR
SUPPORT TYPE 1-A ZONE I

CEILING MNTD. SNGL. CANT. WELD. FT. HGR.
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B (INCH)	P (LB)	F (LB)
12	20	150
12	30	125
12	40	90
12	50	25
15	20	135
15	30	100
15	40	35
18	20	115
18	30	60
21	20	85
24	20	45



ELEVATION

S U P P O R T   C A P A C I T Y   F O R
SUPPORT TYPE 1-A   ZONE II

CEILING MNTD. SNGL. CANT. WELD. FT. HGR.
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B (INCH)	P (LB)	F (LB)
12	20	170
12	30	160
12	40	150
12	50	130
12	60	105
12	70	70
12	80	20
15	20	165
15	30	150
15	40	130
15	50	105
15	60	65
15	70	0
18	20	155
18	30	140
18	40	110
18	50	65
21	20	145
21	30	120
21	40	80
21	50	5
24	20	135
24	30	95
24	40	35
27	20	115
27	30	65
30	20	90
30	30	15

33	20	55
36	20	0

S U P P O R T   C A P A C I T Y   F O R
SUPPORT TYPE 1-A ZONE III

CEILING MNTD. SNGL. CANT. WELD. FT. HGR.
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B (INCH)	P (LB)	F (LB)
12	20	185
12	30	180
12	40	175
12	50	170
12	60	160
12	70	145
12	80	130
12	90	115
12	100	95
12	120	40
15	20	180
15	30	175
15	40	170
15	50	155
15	60	145
15	70	125
15	80	105
15	90	80
15	100	45
18	20	180
18	30	170
18	40	160
18	50	145
18	60	125
18	70	100
18	80	70
18	90	25
21	20	175
21	30	165
21	40	150
21	50	130
21	60	100
21	70	65
21	80	10
24	20	170
24	30	155

24	40	135
24	50	110
24	60	70
24	70	15
27	20	160
27	30	145
27	40	120
27	50	80
27	60	25
30	20	150
30	30	130
30	40	95
30	50	45
33	20	140
33	30	110
33	40	70
36	20	125
36	30	90
36	40	30

S U P P O R T   C A P A C I T Y   F O R
SUPPORT TYPE 1-A   ZONE IV

CEILING MNTD. SNGL. CANT. WELD. FT. HGR.
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B (INCH)	P (LB)	F (LB)
12	20	195
12	30	195
12	40	185
12	50	180
12	60	170
12	70	160
12	80	145
12	90	130
12	100	115
12	120	65
15	20	195
15	30	190
15	40	180
15	50	170
15	60	160
15	70	140
15	80	125
15	90	100
15	100	70
18	20	190
18	30	185
18	40	170
18	50	160
18	60	140
18	70	120
18	80	90
18	90	55
18	100	0
21	20	185
21	30	175
21	40	160
21	50	145
21	60	120
21	70	90
21	80	45
24	20	180

24	30	165
24	40	150
24	50	125
24	60	95
24	70	50
27	20	175
27	30	155
27	40	135
27	50	100
27	60	55
30	20	165
30	30	145
30	40	115
30	50	75
30	60	5
33	20	155
33	30	130
33	40	90
33	50	35
36	20	140
36	30	110
36	40	[illegible]

SUPPORT CAPACITY FOR
SUPPORT TYPE 1-A ZONE V

CEILING MNTD. SNGL. CANT. WELD. FT. HGR.
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B (INCH)	P (LB)	F (LB)
12	20	215
12	30	215
12	40	215
12	50	210
12	60	205
12	70	205
12	80	200
12	90	195
12	100	190
12	120	175
12	140	160
12	160	140
12	180	115
12	200	85
15	20	215
15	30	215
15	40	210
15	50	205
15	60	205
15	70	195
15	80	190
15	90	185
15	100	175
15	120	155
15	140	130
15	160	100
15	180	60
15	200	5
18	20	215
18	30	210
18	40	205
18	50	200
18	60	195
18	70	190
18	80	180
18	90	170
18	100	160
18	120	135
18	140	100
18	160	50

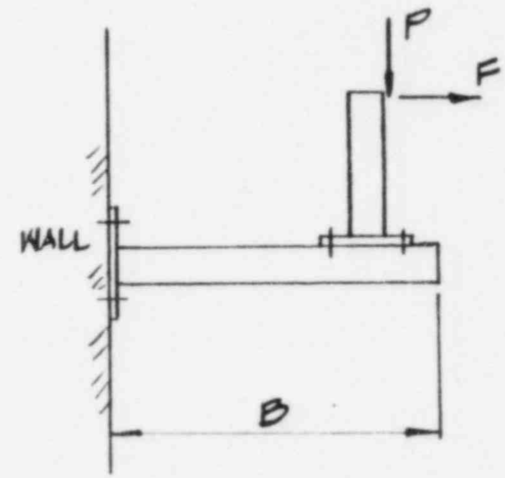
21	20	210
21	30	210
21	40	205
21	50	195
21	60	190
21	70	180
21	80	170
21	90	155
21	100	140
21	120	105
21	140	55
24	20	210
24	30	205
24	40	200
24	50	190
24	60	180
24	70	170
24	80	155
24	90	140
24	100	120
24	120	70
27	20	205
27	30	200
27	40	195
27	50	185
27	60	170
27	70	155
27	80	140
27	90	115
27	100	90
27	120	15
30	20	205
30	30	195
30	40	185
30	50	175
30	60	160
30	70	140
30	80	120
30	90	90
30	100	55
33	20	200
33	30	190
33	40	180
33	50	165
33	60	145
33	70	125
33	80	95
33	90	55

33	100	5
36	20	195
36	30	185
36	40	170
36	50	155
36	60	130
36	70	100
36	80	65
36	90	15

SUPPORT CAPACITY FOR
SUPPORT TYPE 2 A ZONE I

WALL MOUNTED DOUBLE CANTILEVER SUPPORT
USING WELDED FOOT MEMBERS
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B	P	F
12	10	65
12	20	60
12	30	55
12	40	50
12	50	40
12	60	30
12	70	10
18	10	55
18	20	45
18	30	30
24	10	25



ELEVATION

S U P P O R T   C A P A C I T Y   F O R
SUPPORT TYPE 2 - A   ZONE II

WALL MOUNTED DOUBLE CANTILEVER SUPPORT
USING WELDED FOOT MEMBERS
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B	P	F
12	10	105
12	20	100
12	30	95
12	40	85
12	50	75
12	60	60
12	70	35
18	10	95
18	20	85
18	30	65
18	40	40
24	10	75
24	20	50
30	10	35

SUPPORT CAPACITY FOR
SUPPORT TYPE 2 - A ZONE III

WALL MOUNTED DOUBLE CANTILEVER SUPPORT
USING WELDED FOOT MEMBERS
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B	P	F
12	10	165
12	20	160
12	30	150
12	40	135
12	50	120
12	60	100
12	70	75
12	80	20
18	10	155
18	20	140
18	30	115
18	40	80
24	10	135
24	20	105
24	30	45
30	10	105
30	20	25
36	10	40

SUPPORT CAPACITY FOR
SUPPORT TYPE 2 - A ZONE IV

WALL MOUNTED DOUBLE CANTILEVER SUPPORT
USING WELDED FOOT MEMBERS
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B	P	F
12	10	180
12	20	170
12	30	160
12	40	150
12	50	135
12	60	115
12	70	90
12	80	55
18	10	165
18	20	150
18	30	130
18	40	100
18	50	45
24	10	150
24	20	120
24	30	70
30	10	120
30	20	60
36	10	65

S U P P O R T   C A P A C I T Y   F O R
SUPPORT TYPE 2 A ZONE V

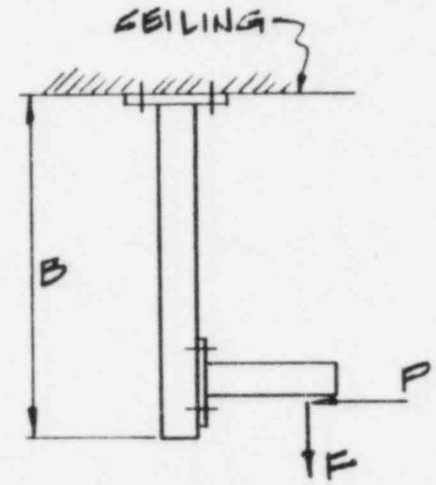
WALL MOUNTED DOUBLE CANTILEVER SUPPORT
USING WELDED FOOT MEMBERS
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B	P	F
12	10	310
12	20	300
12	30	285
12	40	265
12	50	240
12	60	215
12	70	185
12	80	140
12	90	75
18	10	295
18	20	270
18	30	240
18	40	195
18	50	135
24	10	275
24	20	230
24	30	170
24	40	30
30	10	245
30	20	170
36	10	200
36	20	25

SUPPORT CAPACITY FOR
SUPPORT TYPE 2 A ZONE I

CEILING MOUNTED DOUBLE CANTILEV. SUPPORT
USING WELDED FOOT MEMBER
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B	P	F
12	10	65
12	20	60
12	30	55
12	40	50
12	50	40
12	60	30
12	70	15
18	10	50
18	20	45
18	30	30
18	40	10
24	10	25
24	20	0



ELEVATION

SUPPORT CAPACITY FOR
SUPPORT TYPE 2 - A ZONE II

CEILING MOUNTED DOUBLE CANTILEV. SUPPORT
USING WELDED FOOT MEMBER
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B	P	F
12	10	70
12	20	65
12	30	65
12	40	65
12	50	60
12	60	55
12	70	50
12	80	45
12	90	40
12	100	30
12	110	20
12	120	5
18	10	65
18	20	60
18	30	55
18	40	50
18	50	40
18	60	30
18	70	10
24	10	55
24	20	45
24	30	35
24	40	20
30	10	35
30	20	20
36	10	0

S U P P O R T   C A P A C I T Y   F O R
SUPPORT TYPE 2 - A   ZONE III

CEILING MOUNTED DOUBLE CANTILEV. SUPPORT
USING WELDED FOOT MEMBER
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B	P	F
----	-----	-----
12	10	75
12	20	70
12	30	70
12	40	70
12	50	70
12	60	65
12	70	65
12	80	65
12	90	60
12	100	55
12	110	55
12	120	50
12	130	45
12	140	40
12	150	35
18	10	70
18	20	70
18	30	65
18	40	65
18	50	60
18	60	55
18	70	50
18	80	45
18	90	40
18	100	30
18	110	15
18	120	0
24	10	65
24	20	65
24	30	60
24	40	55
24	50	45
24	60	40
24	70	25
24	80	10
30	10	60
30	20	55

30	30	45
30	40	35
30	50	25
30	60	0
36	10	50
36	20	40
36	30	25
36	40	5

S U P P O R T C A P A C I T Y F O R
SUPPORT TYPE 2 - A   ZONE IV

CEILING MOUNTED DOUBLE CANTILEV. SUPPORT
USING WELDED FOOT MEMBER
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B	P	F
12	10	80
12	20	75
12	30	75
12	40	75
12	50	75
12	60	70
12	70	70
12	80	70
12	90	65
12	100	65
12	110	60
12	120	55
12	130	50
12	140	50
12	150	45
18	10	75
18	20	75
18	30	70
18	40	70
18	50	65
18	60	60
18	70	60
18	80	50
18	90	45
18	100	35
18	110	30
18	120	15
18	130	0
24	10	70
24	20	70
24	30	65
24	40	60
24	50	55
24	60	45
24	70	35
24	80	20
24	90	5

30	10	65
30	20	60
30	30	55
30	40	45
30	50	30
30	60	15
36	10	55
36	20	45
36	30	35
36	40	20

SUPPORT CAPACITY FOR
SUPPORT TYPE 2 - A ZONE V

CEILING MOUNTED DOUBLE CANTILEV. SUPPORT
USING WELDED FOOT MEMBER
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

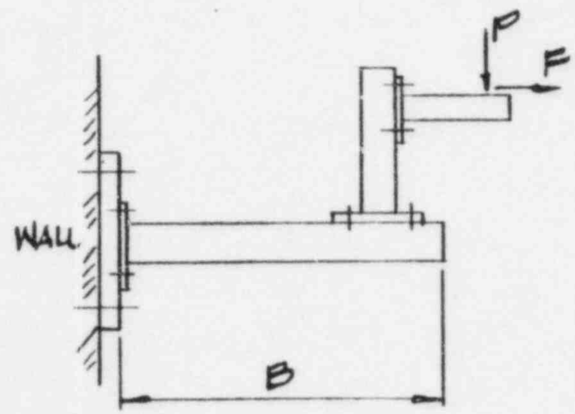
B	P	F
12	10	85
12	20	85
12	30	85
12	40	85
12	50	85
12	60	85
12	70	85
12	80	80
12	90	80
12	100	80
12	110	80
12	120	75
12	130	75
12	140	75
12	150	75
18	10	85
18	20	85
18	30	85
18	40	80
18	50	80
18	60	80
18	70	80
18	80	75
18	90	75
18	100	70
18	110	70
18	120	65
18	130	60
18	140	60
18	150	55
24	10	80
24	20	80
24	30	80
24	40	80
24	50	75
24	60	75
24	70	70
24	80	65
24	90	65
24	100	60
24	110	50

24	120	45
24	130	40
24	140	30
24	150	20
30	10	80
30	20	80
30	30	75
30	40	75
30	50	70
30	60	65
30	70	60
30	80	55
30	90	45
30	100	35
30	110	25
30	120	15
36	10	75
36	20	75
36	30	70
36	40	65
36	50	60
36	60	55
36	70	45
36	80	35
36	90	20
36	100	5

S U P P O R T   C A P A C I T Y   F O R
SUPPORT TYPE 3 - A   ZONE I

WALL MOUNTED TRIPLE CANTILEVER SUPPORT
W/UNIST. HEADER USING WELDED FOOT MEMBER
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B	P	F



ELEVATION

S U P P O R T C A P A C I T Y F O R
SUPPORT TYPE 3 - A   ZONE II

WALL MOUNTED TRIPLE CANTILEVER SUPPORT
W/UNIST. HEADER USING WELDED FOOT MEMBER
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B	P	F

SUPPORT CAPACITY FOR
SUPPORT TYPE 3 - A ZONE III

WALL MOUNTED TRIPLE CANTILEVER SUPPORT
W/UNIST. HEADER USING WELDED FOOT MEMBER
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B	P	F
12	10	235
12	20	225
12	30	215
12	40	200
12	50	185
12	60	165
12	70	130
12	80	70

S U P P O R T   C A P A C I T Y   F O R

SUPPORT TYPE 3 - A   ZONE IV

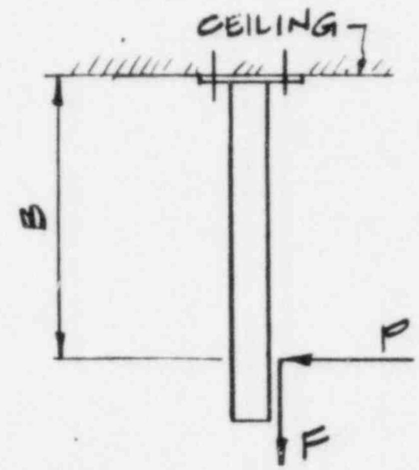
WALL MOUNTED TRIPLE CANTILEVER SUPPORT
W/UNIST. HEADER USING WELDED FOOT MEMBER
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B	P	F
12	10	255
12	20	245
12	30	235
12	40	220
12	50	205
12	60	185
12	70	165
12	80	135
12	90	85
18	10	110
18	20	35

S U P P O R T   C A P A C I T Y   F O R
SUPPORT TYPE 1-A ZONE I

CEILING MNTD. SNGL. CANT. WELD. FT. HGR.
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B (INCH)	P (LB)	F (LB)
12	20	150
12	30	125
12	40	90
12	50	25
15	20	135
15	30	100
15	40	35
18	20	115
18	30	60
21	20	85
24	20	45



ELEVATION

SUPPORT CAPACITY FOR
SUPPORT TYPE 1-A ZONE II

CEILING MNTD. SNGL. CANT. WELD. FT. HGR.
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B (INCH)	P (LB)	F (LB)
12	20	170
12	30	160
12	40	150
12	50	130
12	60	105
12	70	70
12	80	20
15	20	165
15	30	150
15	40	130
15	50	105
15	60	65
15	70	0
18	20	155
18	30	140
18	40	110
18	50	65
21	20	145
21	30	120
21	40	80
21	50	5
24	20	135
24	30	95
24	40	35
27	20	115
27	30	65
30	20	90
30	30	15

33	20	55
36	20	0

S U P P O R T   C A P A C I T Y   F O R
SUPPORT TYPE 1-A   ZONE III

CEILING MNTD. SNGL. CANT. WELD. FT. HGR.
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B (INCH)	P (LB)	F (LB)
12	20	185
12	30	180
12	40	175
12	50	170
12	60	160
12	70	145
12	80	130
12	90	115
12	100	95
12	120	40
15	20	180
15	30	175
15	40	170
15	50	155
15	60	145
15	70	125
15	80	105
15	90	80
15	100	45
18	20	180
18	30	170
18	40	160
18	50	145
18	60	125
18	70	100
18	80	70
18	90	25
21	20	175
21	30	165
21	40	150
21	50	130
21	60	100
21	70	65
21	80	10
24	20	170
24	30	155

24	40	135
24	50	110
24	60	70
24	70	15
27	20	160
27	30	145
27	40	120
27	50	80
27	60	25
30	20	150
30	30	130
30	40	95
30	50	45
33	20	140
33	30	110
33	40	70
36	20	125
36	30	90
36	40	30

SUPPORT CAPACITY FOR
SUPPORT TYPE 1-A ZONE IV

CEILING MNTD. SNGL. CANT. WELD. FT. HGR.
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B (INCH)	P (LB)	F (LB)
12	20	195
12	30	195
12	40	185
12	50	180
12	60	170
12	70	160
12	80	145
12	90	130
12	100	115
12	120	65
15	20	195
15	30	190
15	40	180
15	50	170
15	60	160
15	70	140
15	80	125
15	90	100
15	100	70
18	20	190
18	30	185
18	40	170
18	50	160
18	60	140
18	70	120
18	80	90
18	90	55
18	100	0
21	20	185
21	30	175
21	40	160
21	50	145
21	60	120
21	70	90
21	80	45
24	20	180

24	30	165
24	40	150
24	50	125
24	60	95
24	70	50
27	20	175
27	30	155
27	40	135
27	50	100
27	60	55
30	20	165
30	30	145
30	40	115
30	50	75
30	60	5
33	20	155
33	30	130
33	40	90
33	50	35
36	20	140
36	30	110
36	40	60

SUPPORT CAPACITY FOR
SUPPORT TYPE 1-A ZONE V

CEILING MNTD. SNGL. CANT. WELD. FT. HGR.
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B (INCH)	P (LB)	F (LB)
12	20	215
12	30	215
12	40	215
12	50	210
12	60	205
12	70	205
12	80	200
12	90	195
12	100	190
12	120	175
12	140	160
12	160	140
12	180	115
12	200	85
15	20	215
15	30	215
15	40	210
15	50	205
15	60	205
15	70	195
15	80	190
15	90	185
15	100	175
15	120	155
15	140	130
15	160	100
15	180	60
15	200	5
18	20	215
18	30	210
18	40	205
18	50	200
18	60	195
18	70	190
18	80	180
18	90	170
18	100	160
18	120	135
18	140	100
18	160	50

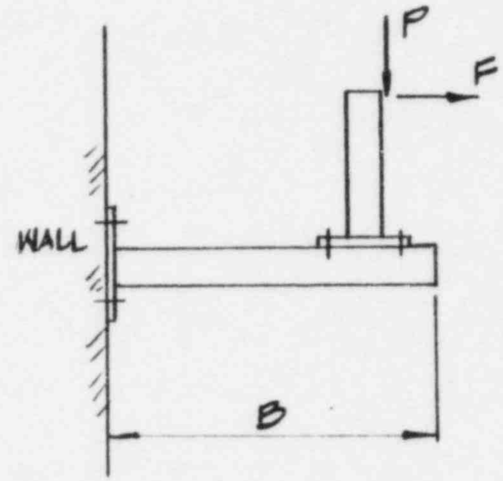
21	20	210
21	30	210
21	40	205
21	50	195
21	60	190
21	70	180
21	80	170
21	90	155
21	100	140
21	120	105
21	140	55
24	20	210
24	30	205
24	40	200
24	50	190
24	60	180
24	70	170
24	80	155
24	90	140
24	100	120
24	120	70
27	20	205
27	30	200
27	40	195
27	50	185
27	60	170
27	70	155
27	80	140
27	90	115
27	100	90
27	120	15
30	20	205
30	30	195
30	40	185
30	50	175
30	60	160
30	70	140
30	80	120
30	90	90
30	100	55
33	20	200
33	30	190
33	40	180
33	50	165
33	60	145
33	70	125
33	80	95
33	90	55

33	100	5
36	20	195
36	30	185
36	40	170
36	50	155
36	60	130
36	70	100
36	80	65
36	90	15

SUPPORT CAPACITY FOR
SUPPORT TYPE 2 - A ZONE I

WALL MOUNTED DOUBLE CANTILEVER SUPPORT
USING WELDED FOOT MEMBERS
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B	P	F
12	10	65
12	20	60
12	30	55
12	40	50
12	50	40
12	60	30
12	70	10
18	10	55
18	20	45
18	30	30
24	10	25



ELEVATION

SUPPORT CAPACITY FOR
SUPPOR⁻ TYPE 2 - A ZONE II

WALL MOUNTED DOUBLE CANTILEVER SUPPORT
USING WELDED FOOT MEMBERS
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B	P	F
12	10	105
12	20	100
12	30	95
12	40	85
12	50	75
12	60	60
12	70	35
18	10	95
18	20	85
18	30	65
18	40	40
24	10	75
24	20	50
30	10	35

S U P P O R T   C A P A C I T Y   F O R
SUPPORT TYPE 2 - A   ZONE III

WALL MOUNTED DOUBLE CANTILEVER SUPPORT
USING WELDED FOOT MEMBERS
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B	P	F
12	10	165
12	20	160
12	30	150
12	40	135
12	50	120
12	60	100
12	70	75
12	80	20
18	10	155
18	20	140
18	30	115
18	40	80
24	10	135
24	20	105
24	30	45
30	10	105
30	20	25
36	10	40

S U P P O R T   C A P A C I T Y   F O R
SUPPORT TYPE 2 - A   ZONE IV

WALL MOUNTED DOUBLE CANTILEVER SUPPORT
USING WELDED FOOT MEMBERS
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B	P	F
12	10	180
12	20	170
12	30	160
12	40	150
12	50	135
12	60	115
12	70	90
12	80	55
18	10	165
18	20	150
18	30	130
18	40	100
18	50	45
24	10	150
24	20	120
24	30	70
30	10	120
30	20	60
36	10	65

SUPPORT CAPACITY FOR
SUPPORT TYPE 2 A ZONE V

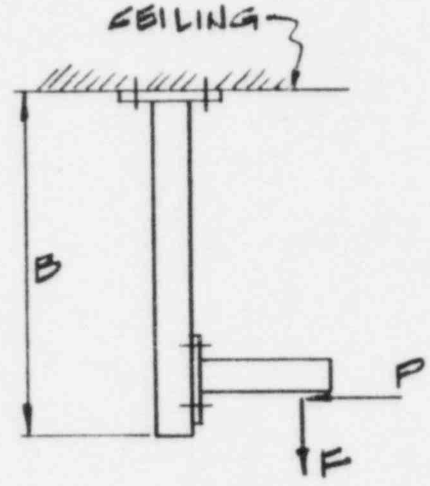
WALL MOUNTED DOUBLE CANTILEVER SUPPORT
USING WELDED FOOT MEMBERS
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B	P	F
12	10	310
12	20	300
12	30	285
12	40	265
12	50	240
12	60	215
12	70	185
12	80	140
12	90	75
18	10	295
18	20	270
18	30	240
18	40	195
18	50	135
24	10	275
24	20	230
24	30	170
24	40	30
30	10	245
30	20	170
36	10	200
36	20	25

S U P P O R T   C A P A C I T Y   F O R
SUPPORT TYPE 2 - A   ZONE I

CEILING MOUNTED DOUBLE CANTILEV. SUPPORT
USING WELDED FOOT MEMBER
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B	P	F
12	10	65
12	20	60
12	30	55
12	40	50
12	50	40
12	60	30
12	70	15
18	10	50
18	20	45
18	30	30
18	40	10
24	10	25
24	20	0



ELEVATION

S U P P O R T   C A P A C I T Y   F O R
SUPPORT TYPE 2 - A   ZONE II

CEILING MOUNTED DOUBLE CANTILEV. SUPPORT
USING WELDED FOOT MEMBER
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B	P	F
12	10	70
12	20	65
12	30	65
12	40	65
12	50	60
12	60	55
12	70	50
12	80	45
12	90	40
12	100	30
12	110	20
12	120	5
18	10	65
18	20	60
18	30	55
18	40	50
18	50	40
18	60	30
18	70	10
24	10	55
24	20	45
24	30	35
24	40	20
30	10	35
30	20	20
36	10	0

S U P P O R T   C A P A C I T Y   F O R
SUPPORT TYPE 2 - A ZONE III

CEILING MOUNTED DOUBLE CANTILEV. SUPPORT
USING WELDED FOOT MEMBER
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B	P	F
12	10	75
12	20	70
12	30	70
12	40	70
12	50	70
12	60	65
12	70	65
12	80	65
12	90	60
12	100	55
12	110	55
12	120	50
12	130	45
12	140	40
12	150	35
18	10	70
18	20	70
18	30	65
18	40	65
18	50	60
18	60	55
18	70	50
18	80	45
18	90	40
18	100	30
18	110	15
18	120	0
24	10	65
24	20	65
24	30	60
24	40	55
24	50	45
24	60	40
24	70	25
24	80	10
30	10	60
30	20	55

30	30	45
30	40	35
30	50	25
30	60	0
36	10	50
36	20	40
36	30	25
36	40	5

SUPPORT CAPACITY FOR
SUPPORT TYPE 2 - A ZONE IV

CEILING MOUNTED DOUBLE CANTILEV. SUPPORT
USING WELDED FOOT MEMBER
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B	P	F
----	-----	-----
12	10	80
12	20	75
12	30	75
12	40	75
12	50	75
12	60	70
12	70	70
12	80	70
12	90	65
12	100	65
12	110	60
12	120	55
12	130	50
12	140	50
12	150	45
18	10	75
18	20	75
18	30	70
18	40	70
18	50	65
18	60	60
18	70	60
18	80	50
18	90	45
18	100	35
18	110	30
18	120	15
18	130	0
24	10	70
24	20	70
24	30	65
24	40	60
24	50	55
24	60	45
24	70	35
24	80	20
24	90	5

30	10	65
30	20	60
30	30	55
30	40	45
30	50	30
30	60	15
36	10	55
36	20	45
36	30	35
36	40	20

S U P P O R T   C A P A C I T Y   F O R
SUPPORT TYPE 2 - A   ZONE   V

CEILING MOUNTED DOUBLE CANTILEV. SUPPORT
USING WELDED FOOT MEMBER
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

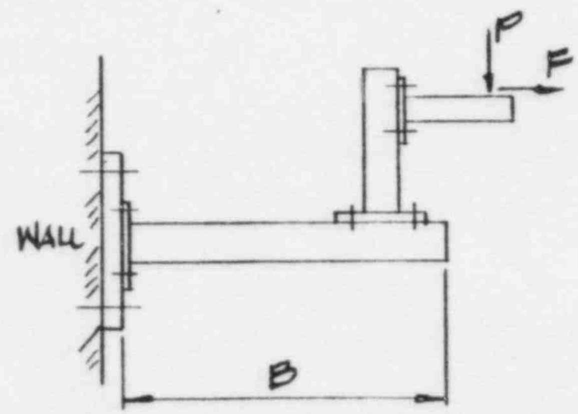
B	P	F
12	10	85
12	20	85
12	30	85
12	40	85
12	50	85
12	60	85
12	70	85
12	80	80
12	90	80
12	100	80
12	110	80
12	120	75
12	130	75
12	140	75
12	150	75
18	10	85
18	20	85
18	30	85
18	40	80
18	50	80
18	60	80
18	70	80
18	80	75
18	90	75
18	100	70
18	110	70
18	120	65
18	130	60
18	140	60
18	150	55
24	10	80
24	20	80
24	30	80
24	40	80
24	50	75
24	60	75
24	70	70
24	80	65
24	90	65
24	100	60
24	110	50

24	120	45
24	130	40
24	140	30
24	150	20
30	10	80
30	20	80
30	30	75
30	40	75
30	50	70
30	60	65
30	70	60
30	80	55
30	90	45
30	100	35
30	110	25
30	120	15
36	10	75
36	20	75
36	30	70
36	40	65
36	50	60
36	60	55
36	70	45
36	80	35
36	90	20
36	100	5

SUPPORT CAPACITY FOR
SUPPORT TYPE 3 A ZONE I

WALL MOUNTED TRIPLE CANTILEVER SUPPORT
W/UNIST. HEADER USING WELDED FOOT MEMBER
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B	P	F
----	-----	-----



ELEVATION

S U P P O R T   C A P A C I T Y   F O R
SUPPORT TYPE 3 - A   ZONE II

WALL MOUNTED TRIPLE CANTILEVER SUPPORT
W/UNIST. HEADER USING WELDED FOOT MEMBER
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B	P	F

S U P P O R T   C A P A C I T Y   F O R
SUPPORT TYPE 3 - A   ZONE III

WALL MOUNTED TRIPLE CANTILEVER SUPPORT
W/UNIST. HEADER USING WELDED FOOT MEMBER
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B	P	F
12	10	235
12	20	225
12	30	215
12	40	200
12	50	185
12	60	165
12	70	130
12	80	70

SUPPORT CAPACITY FOR

SUPPORT TYPE 3 - A ZONE IV

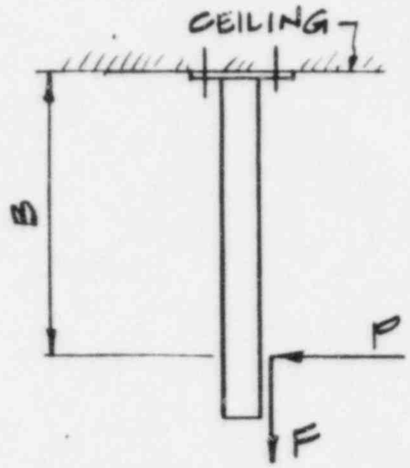
WALL MOUNTED TRIPLE CANTILEVER SUPPORT
W/UNIST. HEADER USING WELDED FOOT MEMBER
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B	P	F
12	10	255
12	20	245
12	30	235
12	40	220
12	50	205
12	60	185
12	70	165
12	80	135
12	90	85
18	10	110
18	20	35

SUPPORT CAPACITY FOR
SUPPORT TYPE 1-A ZONE I

CEILING MNTD. SNGL. CANT. WELD. FT. HGR.
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B (INCH)	P (LB)	F (LB)
12	20	150
12	30	125
12	40	90
12	50	25
15	20	135
15	30	100
15	40	35
18	20	115
18	30	60
21	20	85
24	20	45



ELEVATION

SUPPORT CAPACITY FOR
SUPPORT TYPE 1-A ZONE II

CEILING MNTD. SNGL. CANT. WELD. FT. HGR.
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B (INCH)	P (LB)	F (LB)
12	20	170
12	30	160
12	40	150
12	50	130
12	60	105
12	70	70
12	80	20
15	20	165
15	30	150
15	40	130
15	50	105
15	60	65
15	70	0
18	20	155
18	30	140
18	40	110
18	50	65
21	20	145
21	30	120
21	40	80
21	50	5
24	20	135
24	30	95
24	40	35
27	20	115
27	30	65
30	20	90
30	30	15

33	20	55
36	20	0

S U P P O R T   C A P A C I T Y   F O R
SUPPORT TYPE 1-A ZONE III

CEILING MNTD. SNGL. CANT. WELD. FT. HGR.
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B (INCH)	P (LB)	F (LB)
12	20	185
12	30	180
12	40	175
12	50	170
12	60	160
12	70	145
12	80	130
12	90	115
12	100	95
12	120	40
15	20	180
15	30	175
15	40	170
15	50	155
15	60	145
15	70	125
15	80	105
15	90	80
15	100	45
18	20	180
18	30	170
18	40	160
18	50	145
18	60	125
18	70	100
18	80	70
18	90	25
21	20	175
21	30	165
21	40	150
21	50	130
21	60	100
21	70	65
21	80	10
24	20	170
24	30	155

24	40	135
24	50	110
24	60	70
24	70	15
27	20	160
27	30	145
27	40	120
27	50	80
27	60	25
30	20	150
30	30	130
30	40	95
30	50	45
33	20	140
33	30	110
33	40	70
36	20	125
36	30	90
36	40	30

SUPPORT CAPACITY FOR
SUPPORT TYPE 1-A ZONE IV

CEILING MNTD. SNGL. CANT. WELD. FT HGR.
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B (INCH)	P (LB)	F (LB)
12	20	195
12	30	195
12	40	185
12	50	180
12	60	170
12	70	160
12	80	145
12	90	130
12	100	115
12	120	65
15	20	195
15	30	190
15	40	180
15	50	170
15	60	160
15	70	140
15	80	125
15	90	100
15	100	70
18	20	190
18	30	185
18	40	170
18	50	160
18	60	140
18	70	120
18	80	90
18	90	55
18	100	0
21	20	185
21	30	175
21	40	160
21	50	145
21	60	120
21	70	90
21	80	45
24	20	180

24	30	165
24	40	150
24	50	125
24	60	95
24	70	50
27	20	175
27	30	155
27	40	135
27	50	100
27	60	55
30	20	165
30	30	145
30	40	115
30	50	75
30	60	5
33	20	155
33	30	130
33	40	90
33	50	35
36	20	140
36	30	110
36	40	60

SUPPORT CAPACITY FOR
SUPPORT TYPE 1-A ZONE V

CEILING MNTD. SNGL. CANT. WELD. FT. HGR.
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B (INCH)	P (LB)	F (LB)
12	20	215
12	30	215
12	40	215
12	50	210
12	60	205
12	70	205
12	80	200
12	90	195
12	100	190
12	120	175
12	140	160
12	160	140
12	180	115
12	200	85
15	20	215
15	30	215
15	40	210
15	50	205
15	60	205
15	70	195
15	80	190
15	90	185
15	100	175
15	120	155
15	140	130
15	160	100
15	180	60
15	200	5
18	20	215
18	30	210
18	40	205
18	50	200
18	60	195
18	70	190
18	80	180
18	90	170
18	100	160
18	120	135
18	140	100
18	160	50

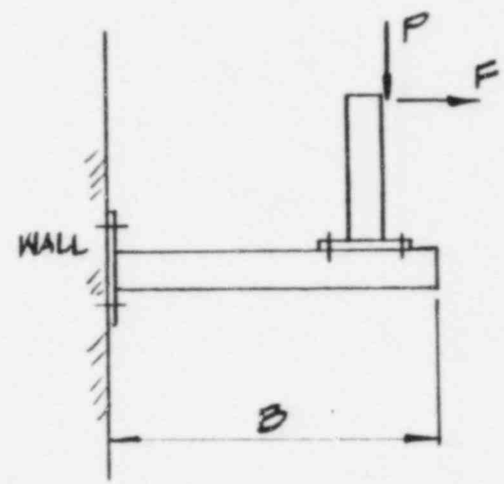
21	20	210
21	30	210
21	40	205
21	50	195
21	60	190
21	70	180
21	80	170
21	90	155
21	100	140
21	120	105
21	140	55
24	20	210
24	30	205
24	40	200
24	50	190
24	60	180
24	70	170
24	80	155
24	90	140
24	100	120
24	120	70
27	20	205
27	30	200
27	40	195
27	50	185
27	60	170
27	70	155
27	80	140
27	90	115
27	100	90
27	120	15
30	20	205
30	30	195
30	40	185
30	50	175
30	60	160
30	70	140
30	80	120
30	90	90
30	100	55
33	20	200
33	30	190
33	40	180
33	50	165
33	60	145
33	70	125
33	80	95
33	90	55

33	100	5
36	20	195
36	30	135
36	40	170
36	50	155
36	60	130
36	70	100
36	80	65
36	90	15

SUPPORT CAPACITY FOR
SUPPORT TYPE 2 - A ZONE I

WALL MOUNTED DOUBLE CANTILEVER SUPPORT
USING WELDED FOOT MEMBERS
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B	P	F
12	10	65
12	20	60
12	30	55
12	40	50
12	50	40
12	60	30
12	70	10
18	10	55
18	20	45
18	30	30
24	10	25



ELEVATION

S U P P O R T   C A P A C I T Y   F O R
SUPPORT TYPE 2 - A   ZONE II

WALL MOUNTED DOUBLE CANTILEVER SUPPORT
USING WELDED FOOT MEMBERS
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B	P	F
12	10	105
12	20	100
12	30	95
12	40	85
12	50	75
12	60	60
12	70	35
18	10	95
18	20	85
18	30	65
18	40	40
24	10	75
24	20	50
30	10	35

SUPPORT CAPACITY FOR
SUPPORT TYPE 2 - A ZONE III

WALL MOUNTED DOUBLE CANTILEVER SUPPORT
USING WELDED FOOT MEMBERS
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B	P	F
12	10	165
12	20	160
12	30	150
12	40	135
12	50	120
12	60	100
12	70	75
12	80	20
18	10	155
18	20	140
18	30	115
18	40	80
24	10	135
24	20	105
24	30	45
30	10	105
30	20	25
36	10	40

SUPPORT CAPACITY FOR
SUPPORT TYPE 2 - A ZONE IV

WALL MOUNTED DOUBLE CANTILEVER SUPPORT
USING WELDED FOOT MEMBERS
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B	P	F
12	10	180
12	20	170
12	30	160
12	40	150
12	50	135
12	60	115
12	70	90
12	80	55
18	10	165
18	20	150
18	30	130
18	40	100
18	50	45
24	10	150
24	20	120
24	30	70
30	10	120
30	20	60
36	10	65

S U P P O R T C A P A C I T Y F O R
SUPPORT TYPE 2 A ZONE V

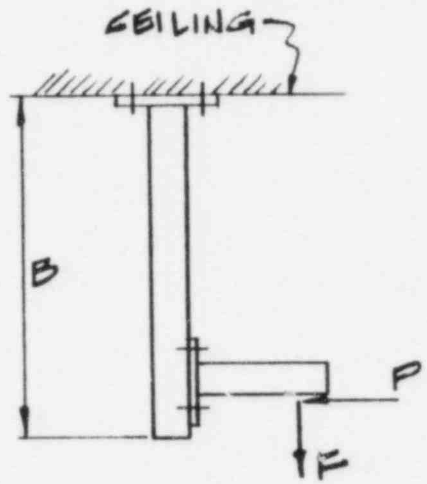
WALL MOUNTED DOUBLE CANTILEVER SUPPORT
USING WELDED FOOT MEMBERS
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B	P	F
12	10	310
12	20	300
12	30	285
12	40	265
12	50	240
12	60	215
12	70	185
12	80	140
12	90	75
18	10	295
18	20	270
18	30	240
18	40	195
18	50	135
24	10	275
24	20	230
24	30	170
24	40	30
30	10	245
30	20	170
36	10	200
36	20	25

SUPPORT CAPACITY FOR
SUPPORT TYPE 2 - A ZONE I

CEILING MOUNTED DOUBLE CANTILEV. SUPPORT
USING WELDED FOOT MEMBER
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B	P	F
12	10	65
12	20	60
12	30	55
12	40	50
12	50	40
12	60	30
12	70	15
18	10	50
18	20	45
18	30	30
18	40	10
24	10	25
24	20	0



ELEVATION

SUPPORT CAPACITY FOR
SUPPORT TYPE 2 - A ZONE II

CEILING MOUNTED DOUBLE CANTILEV. SUPPORT
USING WELDED FOOT MEMBER
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B	P	F
12	10	70
12	20	65
12	30	65
12	40	65
12	50	60
12	60	55
12	70	50
12	80	45
12	90	40
12	100	30
12	110	20
12	120	5
18	10	65
18	20	60
18	30	55
18	40	50
18	50	40
18	60	30
18	70	10
24	10	55
24	20	45
24	30	35
24	40	20
30	10	35
30	20	20
36	10	0

S U P P O R T   C A P A C I T Y   F O R
SUPPORT TYPE 2 - A   ZONE III

CEILING MOUNTED DOUBLE CANTILEV. SUPPORT
USING WELDED FOOT MEMBER
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B	P	F
12	10	75
12	20	70
12	30	70
12	40	70
12	50	70
12	60	65
12	70	65
12	80	65
12	90	60
12	100	55
12	110	55
12	120	50
12	130	45
12	140	40
12	150	35
18	10	70
18	20	70
18	30	65
18	40	65
18	50	60
18	60	55
18	70	50
18	80	45
18	90	40
18	100	30
18	110	15
18	120	0
24	10	65
24	20	65
24	30	60
24	40	55
24	50	45
24	60	40
24	70	25
24	80	10
30	10	60
30	20	55

30	30	45
30	40	35
30	50	25
30	60	0
36	10	50
36	20	40
36	30	25
36	40	5

SUPPORT CAPACITY FOR
SUPPORT TYPE 2 - A ZONE IV

CEILING MOUNTED DOUBLE CANTILEV. SUPPORT
USING WELDED FOOT MEMBER
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B	P	F
12	10	80
12	20	75
12	30	75
12	40	75
12	50	75
12	60	70
12	70	70
12	80	70
12	90	65
12	100	65
12	110	60
12	120	55
12	130	50
12	140	50
12	150	45
18	10	75
18	20	75
18	30	70
18	40	70
18	50	65
18	60	60
18	70	60
18	80	50
18	90	45
18	100	35
18	110	30
18	120	15
18	130	0
24	10	70
24	20	70
24	30	65
24	40	60
24	50	55
24	60	45
24	70	35
24	80	20
24	90	5

30	10	65
30	20	60
30	30	55
30	40	45
30	50	30
30	60	15
36	10	55
36	20	45
36	30	35
36	40	20

SUPPORT CAPACITY FOR
SUPPORT TYPE 2 - A ZONE V

CEILING MOUNTED DOUBLE CANTILEV. SUPPORT
USING WELDED FOOT MEMBER
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

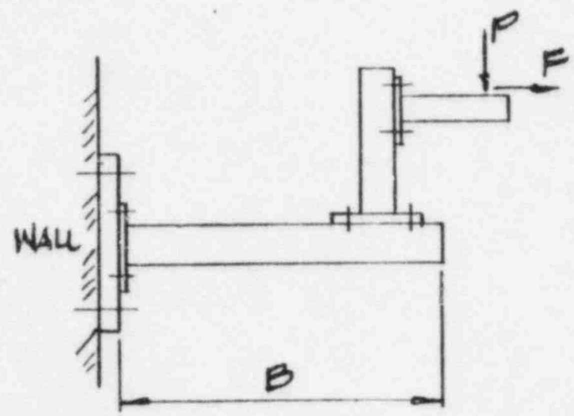
B	P	F
12	10	85
12	20	85
12	30	85
12	40	85
12	50	85
12	60	85
12	70	85
12	80	80
12	90	80
12	100	80
12	110	80
12	120	75
12	130	75
12	140	75
12	150	75
18	10	85
18	20	85
18	30	85
18	40	80
18	50	80
18	60	80
18	70	80
18	80	75
18	90	75
18	100	70
18	110	70
18	120	65
18	130	60
18	140	60
18	150	55
24	10	80
24	20	80
24	30	80
24	40	80
24	50	75
24	60	75
24	70	70
24	80	65
24	90	65
24	100	60
24	110	50

24	120	45
24	130	40
24	140	30
24	150	20
30	10	80
30	20	80
30	30	75
30	40	75
30	50	70
30	60	65
30	70	60
30	80	55
30	90	45
30	100	35
30	110	25
30	120	15
36	10	75
36	20	75
36	30	70
36	40	65
36	50	60
36	60	55
36	70	45
36	80	35
36	90	20
36	100	5

SUPPORT CAPACITY FOR
SUPPORT TYPE 3 - A ZONE I

WALL MOUNTED TRIPLE CANTILEVER SUPPORT
W/UNIST. HEADER USING WELDED FOOT MEMBER
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B	P	F



ELEVATION

S U P P O R T   C A P A C I T Y   F O R
SUPPORT TYPE 3 - A   ZONE II

WALL MOUNTED TRIPLE CANTILEVER SUPPORT
W/UNIST. HEADER USING WELDED FOOT MEMBER
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B	P	F

S U P P O R T   C A P A C I T Y   F O R
SUPPORT TYPE 3 - A   ZONE III

WALL MOUNTED TRIPLE CANTILEVER SUPPORT
W/UNIST. HEADER USING WELDED FOOT MEMBER
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B	P	F
12	10	235
12	20	225
12	30	215
12	40	200
12	50	185
12	60	165
12	70	130
12	80	70

SUPPORT CAPACITY FOR

SUPPORT TYPE 3 - A ZONE IV

WALL MOUNTED TRIPLE CANTILEVER SUPPORT
W/UNIST. HEADER USING WELDED FOOT MEMBER
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B	P	F
12	10	255
12	20	245
12	30	235
12	40	220
12	50	205
12	60	185
12	70	165
12	80	135
12	90	85
18	10	110
18	20	35

SUPPORT CAPACITY FOR
SUPPORT TYPE 3 - A ZONE V

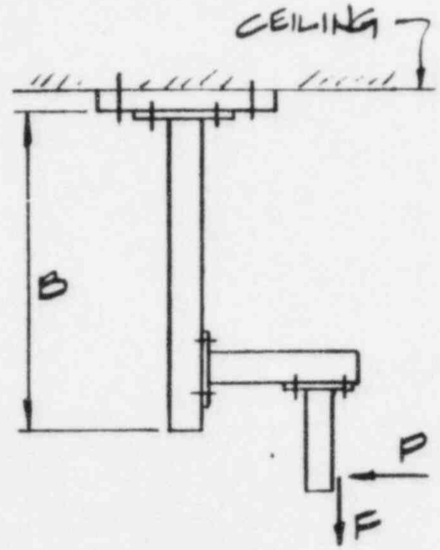
WALL MOUNTED TRIPLE CANTILEVER SUPPORT
W/UNIST. HEADER USING WELDED FOOT MEMBER
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B	P	F
12	10	475
12	20	460
12	30	445
12	40	425
12	50	400
12	60	375
12	70	345
12	80	315
12	90	275
12	100	225
12	110	160
12	120	30
18	10	450
18	20	425
18	30	400
18	40	370
18	50	335
18	60	290
18	70	235
18	80	155
24	10	410
24	20	380
24	30	335
24	40	245
24	50	75

SUPPORT CAPACITY FOR
SUPPORT TYPE 3 - A ZONE I

CEILING MOUNTED TRIPLE CANTILEV. SUPPORT
W/UNIST. HEADER USING WELDED FOOT MEMBER
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B	P	F



ELEVATION

SUPPORT CAPACITY FOR
SUPPORT TYPE 3 - A ZONE II

CEILING MOUNTED TRIPLE CANTILEV. SUPPORT
W/UNIST. HEADER USING WELDED FOOT MEMBER
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B	P	F
12	10	50
12	20	45
12	30	45
12	40	40
12	50	35
12	60	30
12	70	25
12	80	15
12	90	5

S U P P O R T   C A P A C I T Y   F O R

SUPPORT TYPE 3 - A   ZONE III

CEILING MOUNTED TRIPLE CANTILEV. SUPPORT
W/UNIST. HEADER USING WELDED FOOT MEMBER
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B	P	F
12	10	90
12	20	90
12	30	85
12	40	85
12	50	85
12	60	80
12	70	80
12	80	80
12	90	75
12	100	70
12	110	70
12	120	65
12	130	60
12	140	55
12	150	50
18	10	55
18	20	55
18	30	55
18	40	50
18	50	45
18	60	45
18	70	40
18	80	35
18	90	25
18	100	20
18	110	10
18	120	0

SUPPORT CAPACITY FOR
SUPPORT TYPE 3 - A ZONE IV

CEILING MOUNTED TRIPLE CANTILEV. SUPPORT
W/UNIST. HEADER USING WELDED FOOT MEMBER
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B	P	F
12	10	95
12	20	95
12	30	95
12	40	95
12	50	95
12	60	90
12	70	90
12	80	85
12	90	85
12	100	80
12	110	80
12	120	75
12	130	70
12	140	70
12	150	65
18	10	65
18	20	65
18	30	65
18	40	60
18	50	60
18	60	55
18	70	55
18	80	50
18	90	45
18	100	35
18	110	30
18	120	25
18	130	15
18	140	0
24	10	5
24	20	5
24	30	0

S U P P O R T   C A P A C I T Y   F O R
SUPPORT TYPE 3 - A   ZONE V

CEILING MOUNTED TRIPLE CANTILEV. SUPPORT
W/UNIST. HEADER USING WELDED FOOT MEMBER
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

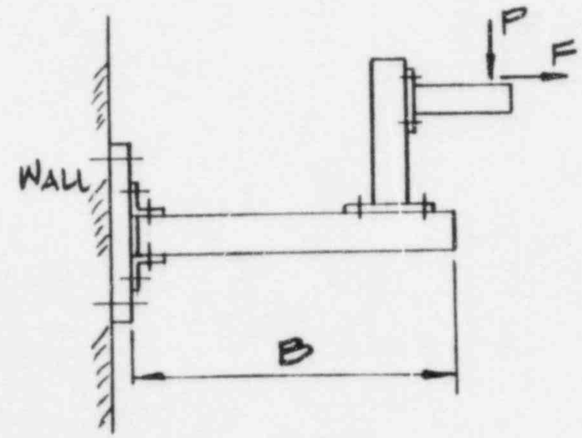
B	P	F
12	10	120
12	20	120
12	30	120
12	40	120
12	50	120
12	60	120
12	70	120
12	80	120
12	90	120
12	100	115
12	110	115
12	120	115
12	130	115
12	140	115
12	150	110
18	10	110
18	20	110
18	30	110
18	40	110
18	50	110
18	60	110
18	70	105
18	80	105
18	90	105
18	100	105
18	110	100
18	120	100
18	130	100
18	140	95
18	150	95
24	10	95
24	20	95
24	30	95
24	40	95
24	50	90
24	60	90
24	70	90
24	80	85
24	90	85
24	100	80
24	110	80

24	120	75
24	130	70
24	140	70
24	150	65
30	10	70
30	20	70
30	30	65
30	40	65
30	50	65
30	60	60
30	70	55
30	80	55
30	90	50
30	100	45
30	110	40
30	120	35
30	130	30
30	140	25
30	150	15
36	10	25
36	20	25
36	30	20
36	40	20
36	50	15
36	60	10
36	70	5

SUPPORT CAPACITY FOR
SUPPORT TYPE 3 - B ZONE I

WALL MOUNTED TRIPLE CANTILEVER SUPPORT
W/UNIST. HEADER USING UNISTRUT FITTINGS
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B	P	F
----	-----	-----



ELEVATION

SUPPORT CAPACITY FOR
SUPPORT TYPE 3 - B ZONE II

WALL MOUNTED TRIPLE CANTILEVER SUPPORT
W/UNIST. HEADER USING UNISTRUT FITTINGS
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B	P	F

S U P P O R T   C A P A C I T Y   F O R
SUPPORT TYPE 3 - B   ZONE III

WALL MOUNTED TRIPLE CANTILEVER SUPPORT
W/UNIST. HEADER USING UNISTRUT FITTINGS
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B	P	F
12	10	235
12	20	225
12	30	215
12	40	200
12	50	185
12	60	165
12	70	130
12	80	70

SUPPORT CAPACITY FOR
SUPPORT TYPE 3 - B ZONE IV

WALL MOUNTED TRIPLE CANTILEVER SUPPORT
W/UNIST. HEADER USING UNISTRUT FITTINGS
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B	P	F
12	10	255
12	20	245
12	30	235
12	40	220
12	50	205
12	60	185
12	70	165
12	80	135
12	90	85
18	10	110
18	20	35

SUPPORT CAPACITY FOR
SUPPORT TYPE 3 - B ZONE V

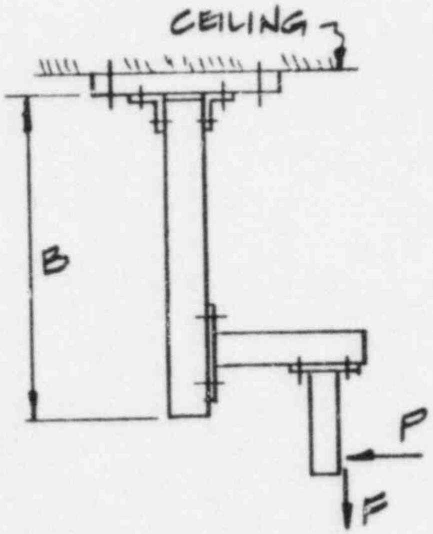
WALL MOUNTED TRIPLE CANTILEVER SUPPORT
W/UNIST. HEADER USING UNISTRUT FITTINGS
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B	P	F
12	10	475
12	20	460
12	30	445
12	40	425
12	50	400
12	60	375
12	70	345
12	80	315
12	90	275
12	100	225
12	110	160
12	120	30
18	10	450
18	20	425
18	30	400
18	40	370
18	50	335
18	60	290
18	70	235
18	80	155
24	10	410
24	20	380
24	30	335
24	40	245
24	50	75

S U P P O R T   C A P A C I T Y   F O R
SUPPORT TYPE 3 - B   ZONE   I

CEILING MOUNTED TRIPLE CANTILEV. SUPPORT
W/UNIST. HEADER USING UNISTRUT FITTINGS
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B	P	F
----	-----	-----



ELEVATION

S U P P O R T   C A P A C I T Y   F O R

SUPPORT TYPE 3 - B   ZONE II

CEILING MOUNTED TRIPLE CANTILEV. SUPPORT
W/UNIST. HEADER USING UNISTRUT FITTINGS
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B	P	F
12	10	50
12	20	45
12	30	45
12	40	40
12	50	35
12	60	30
12	70	25
12	80	15
12	90	5

SUPPORT CAPACITY FOR
SUPPORT TYPE 3 - 6 ZONE III

CEILING MOUNTED TRIPLE CANTILEV. SUPPORT
W/UNIST. HEADER USING UNISTRUT FITTINGS
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B	P	F
12	10	90
12	20	90
12	30	85
12	40	85
12	50	85
12	60	80
12	70	80
12	80	80
12	90	75
12	100	70
12	110	70
12	120	65
12	130	60
12	140	55
12	150	50
18	10	55
18	20	55
18	30	55
18	40	50
18	50	45
18	60	45
18	70	40
18	80	35
18	90	25
18	100	20
18	110	10
18	120	0

S U P P O R T   C A P A C I T Y   F O R
SUPPORT TYPE 3 - B   ZONE IV

CEILING MOUNTED TRIPLE CANTILEV. SUPPORT
W/UNIST. HEADER USING UNISTRUT FITTINGS
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B	P	F
12	10	95
12	20	95
12	30	95
12	40	95
12	50	95
12	60	90
12	70	90
12	80	85
12	90	85
12	100	80
12	110	80
12	120	75
12	130	70
12	140	70
12	150	65
18	10	65
18	20	65
18	30	65
18	40	60
18	50	60
18	60	55
18	70	55
18	80	50
18	90	45
18	100	35
18	110	30
18	120	25
18	130	15
18	140	0
24	10	5
24	20	5
24	30	0

S U P P O R T   C A P A C I T Y   F O R
SUPPORT TYPE 3 - B   ZONE   V

CEILING MOUNTED TRIPLE CANTILEV. SUPPORT
W/UNIST. HEADER USING UNISTRUT FITTINGS
3/8"HKB-WITH/2" EMBEDMENT
HORIZONTAL CONDUIT RUN

B	P	F
12	10	120
12	20	120
12	30	120
12	40	120
12	50	120
12	60	120
12	70	120
12	80	120
12	90	120
12	100	115
12	110	115
12	120	115
12	130	115
12	140	115
12	150	110
18	10	110
18	20	110
18	30	110
18	40	110
18	50	110
18	60	110
18	70	105
18	80	105
18	90	105
18	100	105
18	110	100
18	120	100
18	130	100
18	140	95
18	150	95
24	10	95
24	20	95
24	30	95
24	40	95
24	50	90
24	60	90
24	70	90
24	80	85
24	90	85
24	100	80
24	110	80

24	120	75
24	130	70
24	140	70
24	150	65
30	10	70
30	20	70
30	30	65
30	40	65
30	50	65
30	60	60
30	70	55
30	80	55
30	90	50
30	100	45
30	110	40
30	120	35
30	130	30
30	140	25
30	150	15
36	10	25
36	20	25
36	30	20
36	40	20
36	50	15
36	60	10
36	70	5

DETERMINE SUPPORT LOAD

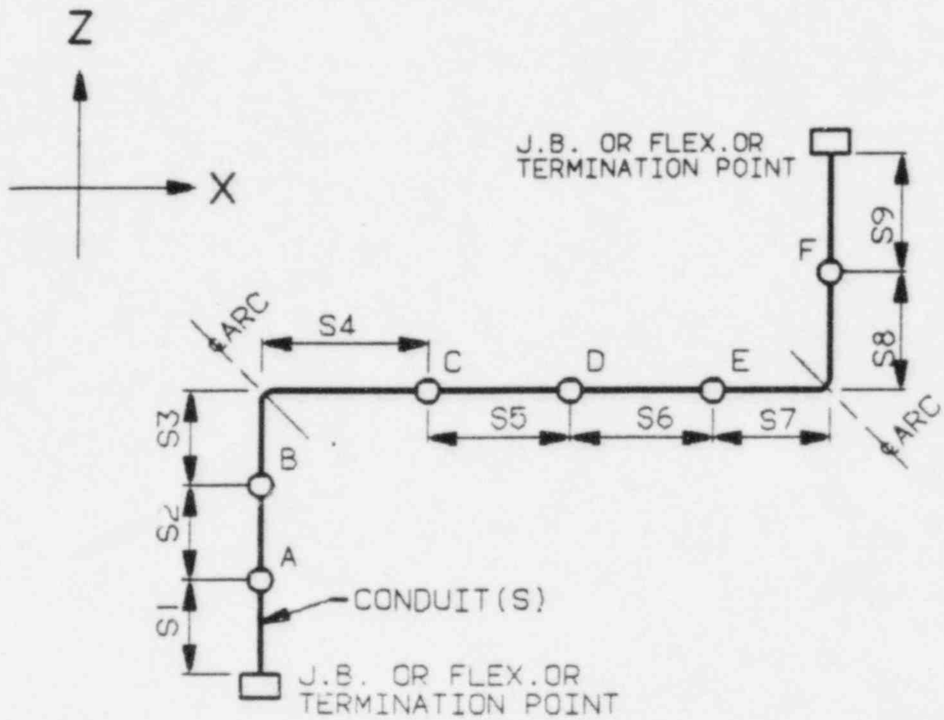
THE CONDUIT LOADS ACTING ON THE SUPPORTS (SUPPORT LOADS) ARE DEPENDING ON:

1. CONDUIT RUN SCHEMATIC

2. CONDUIT SUPPORT TYPE

SUPPORT LOAD:

A) SCHEME I -HORIZONTAL RUN WITH VERTICAL SUPPORTS ONLY



PLAN

LEGEND:

⊗ MULTI DIRECTIONAL SUPPORT

× VERTICAL / TRANSVERSE SUPPORT

O VERTICAL SUPPORT

J.B. JUNCTION BOX

FLEX. FLEXIBLE CONDUIT

CONDUIT RUN DIR.	TYPE OF SUPPORT	SUPPORTING LOAD IN DIRECTION		
		VERT.	HORIZ.	LONG.
HORIZ. RUN	⊗	✓	✓	✓
	×	✓	✓	-
	O	✓	-	-
VERT. RUN	⊗	✓	✓	✓
	×	-	✓	-
	O	-	-	-

a) TRIBUTARY SPAN IN Y (VERTICAL) DIRECTION

SUP'T "A" YA=S1+S2 /2

" "B" YB=1/2(S2+S3+S4)

" "C" YC=1/2(S3+S4+S5)

" "D" YD=1/2(S5+S6)

" "E" YE=1/2(S6+S7+S8)

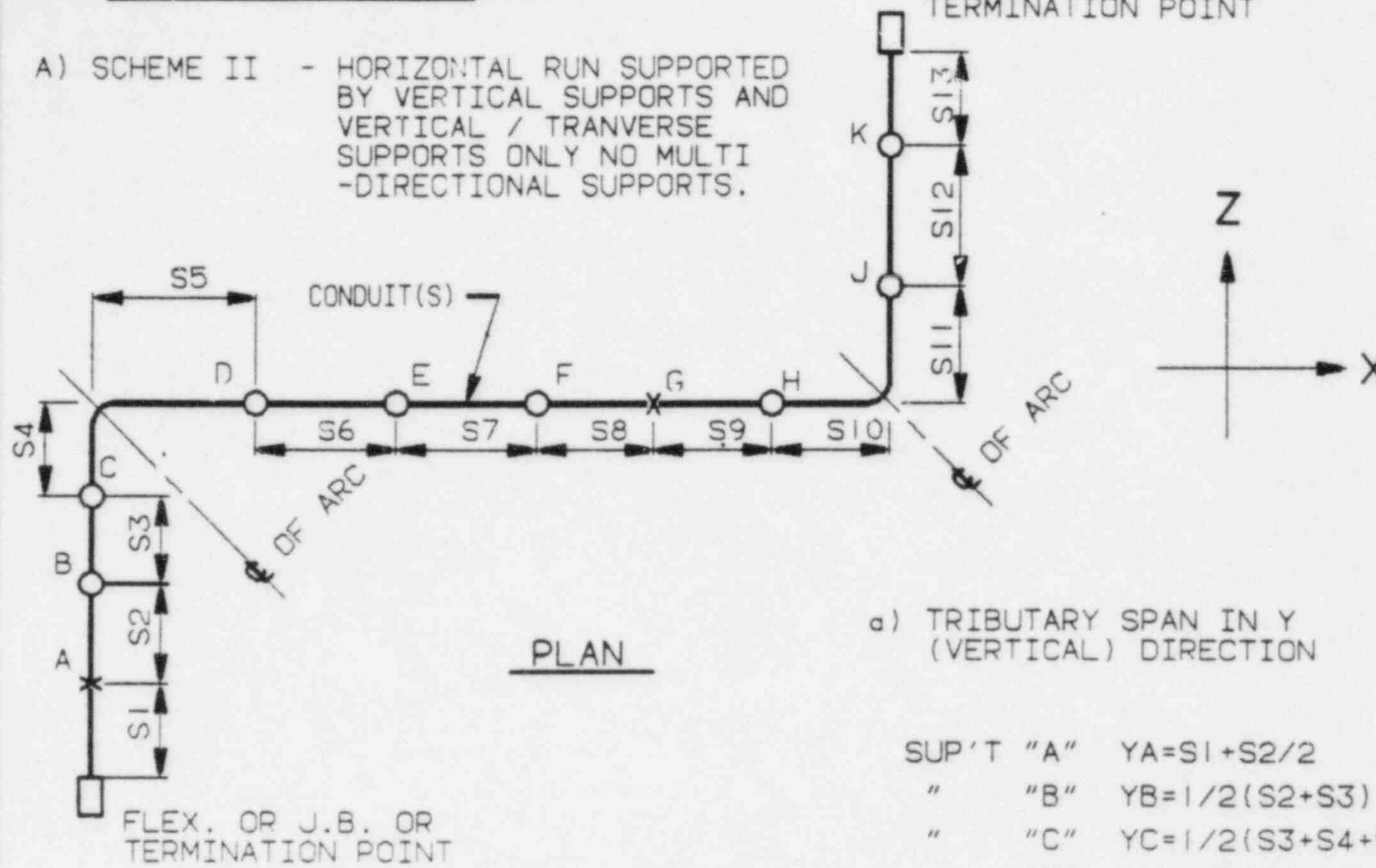
" "F" YF=1/2(S7+S8)+S9

SUPPORT LOAD= TRIBUTARY SPAN x TOTAL CONDUIT UNIT WEIGHT

CONDUIT RUN IS NOT RESTRAINED IN THE X AND Z DIRECTIONS.

SUPPORT LOAD:

A) SCHEME II - HORIZONTAL RUN SUPPORTED BY VERTICAL SUPPORTS AND VERTICAL / TRANVERSE SUPPORTS ONLY NO MULTI -DIRECTIONAL SUPPORTS.



PLAN

a) TRIBUTARY SPAN IN Y (VERTICAL) DIRECTION

SUP'T	"A"	YA=S1+S2/2
"	"B"	YB=1/2(S2+S3)
"	"C"	YC=1/2(S3+S4+S5)
"	"D"	YD=1/2(S4+S5+S6)
"	"E"	YE=1/2(S6+S7)
"	"F"	YF=1/2(S7+S8)
"	"G"	YG=1/2(S8+S9)
"	"H"	YH=1/2(S9+S10+S11)
"	"J"	YJ=1/2(S10+S11+S12)
"	"K"	YK=S12/2+S13

b) TRIBUTARY SPAN IN X DIRECTION SUPPORT AT "A" ONLY

SUP'T "A" XA=S1+S2+.....S13

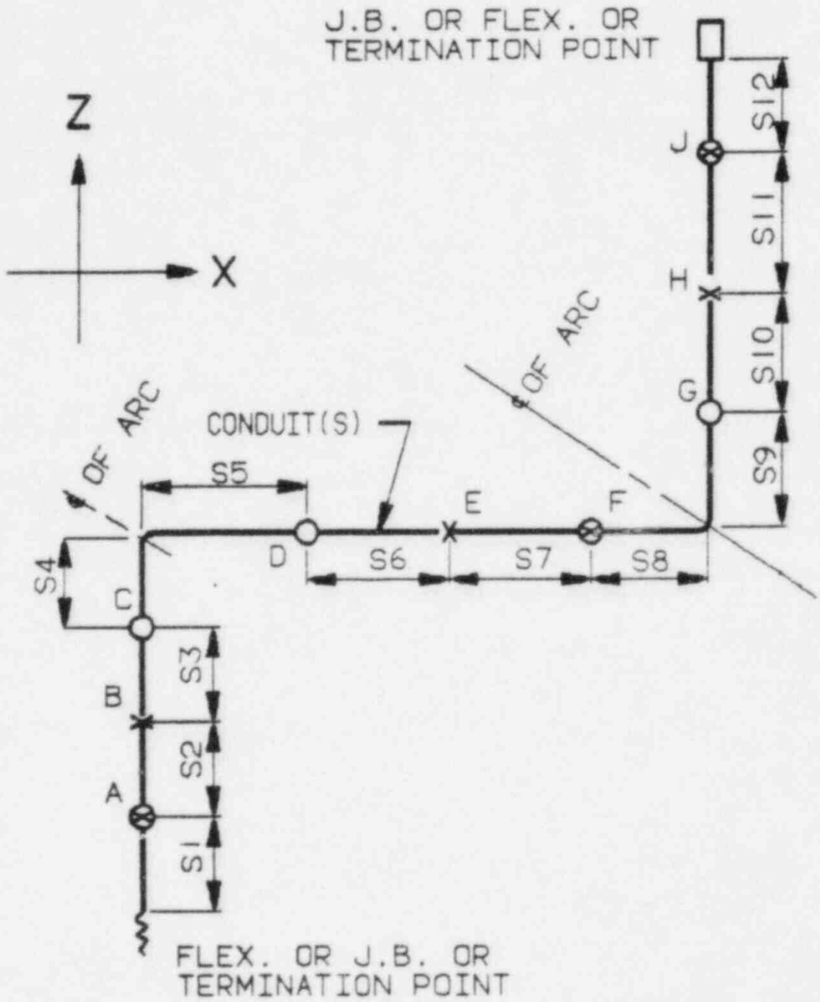
c) TRIBUTARY SPAN IN Z DIRECTION SUPPORT AT" G "ONLY

SUP'T "G" ZG=S1+S2+.....S13

SUPPORT LOAD=TRIBUTARY SPAN x TOTAL CONDUIT UNIT WEIGHT

SUPPORT LOAD:

A) SCHEME III - HORIZONTAL RUN SUPPORTED BY ALL THREE SUPPORT TYPES.



PLAN

a) TRIBUTARY SPAN IN Y (VERTICAL) DIRECTION

SUP'T	"A"	YA=S1+S2/2
"	"B"	YB=1/2(S2+S3)
"	"C"	YC=1/2(S3+S4+S5)
"	"D"	YD=1/2(S4+S5+S6)
"	"E"	YE=1/2(S6+S7)
"	"F"	YF=1/2(S7+S8+S9)
"	"G"	YG=1/2(S8+S9+S10)
"	"H"	YH=1/2(S10+S11)
"	"J"	YJ=S11/2+S12

b) TRIBUTARY SPAN IN X DIRECTION. SUPPORTS A,B,F,H & J ONLY

SUP'T "A" XA=S1+S2/2

SUP'T "B" XB=1/2(S2+S3+S4)

SUP'T "F" XF=1/2(S3+S4+S9+S10)+S5+S6+S7+S8

SUP'T "H" XH=1/2(S9+S10+S11)

SUP'T "J" XJ=S11/2+S12

c) TRIBUTARY SPAN IN Z DIRECTION. SUPPORTS A,E,F & J ONLY

SUP'T "A" ZA=(S1+S2+S3+S4)+1/2(S5+S6)

SUP'T "E" ZE= 1/2(S5+S6+S7)

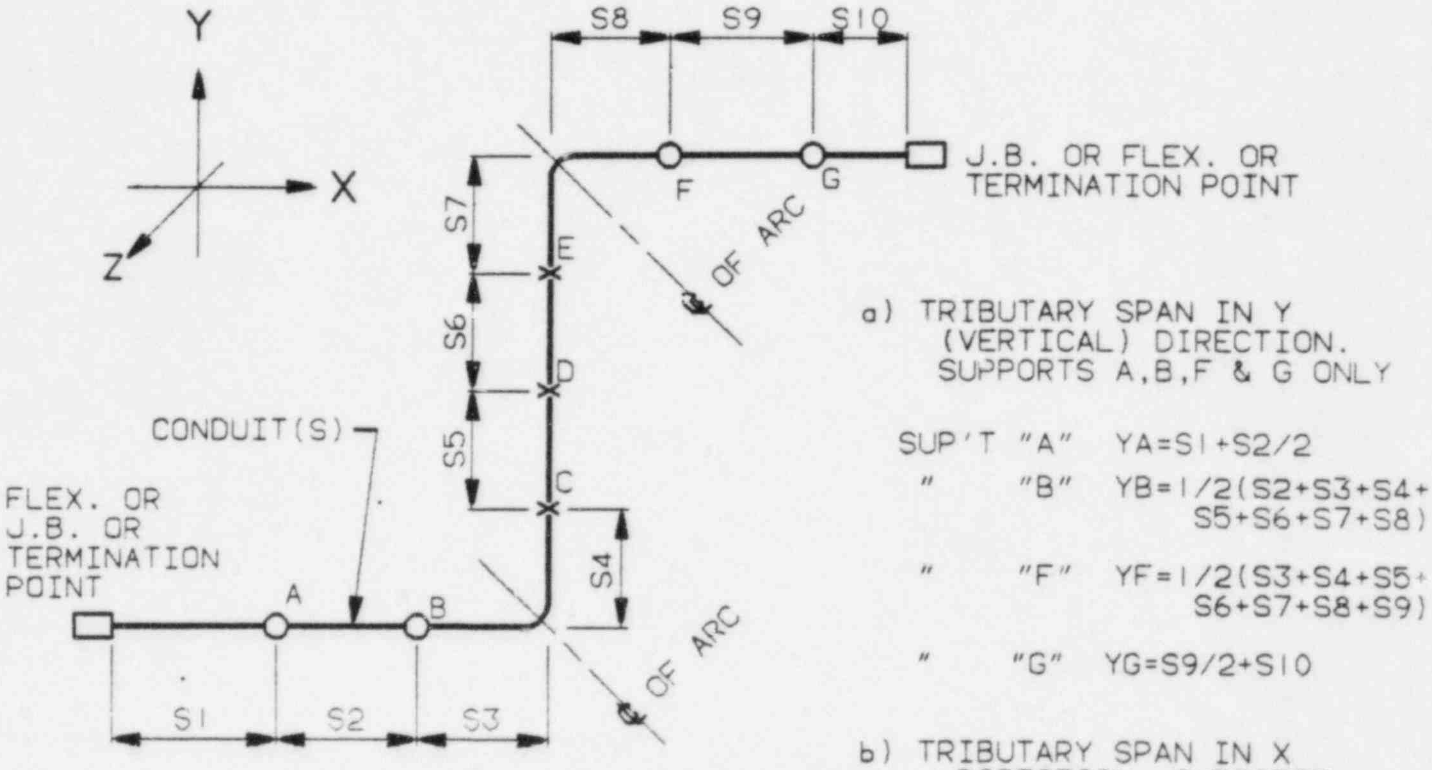
SUP'T "F" ZF= 1/2(S7+S8)

SUP'T "J" ZJ= S8/2+(S9+S10+S11+S12)

SUPPORT LOAD=TRIBUTARY SPAN x TOTAL CONDUIT UNIT WEIGHT.

SUPPORT LOAD:

B) SCHEME IV - VERTICAL RUN SUPPORTED BY VERTICAL SUPPORTS AND VERTICAL / TRANVERSE SUPPORTS ONLY, NO MULTI-DIRECTIONAL SUPPORTS



ELEVATION

a) TRIBUTARY SPAN IN Y (VERTICAL) DIRECTION. SUPPORTS A,B,F & G ONLY

SUP'T "A" $YA=S1+S2/2$

" "B" $YB=1/2(S2+S3+S4+S5+S6+S7+S8)$

" "F" $YF=1/2(S3+S4+S5+S6+S7+S8+S9)$

" "G" $YG=S9/2+S10$

b) TRIBUTARY SPAN IN X DIRECTION. SUPPORTS C,D & E ONLY

SUP'T "C" $XC=(S1+S2+S3+S4)+S5/2$

SUP'T "D" $XD=1/2\ (S5+S6)$

SUP'T "E" $XE=S6/2+(S7+S8+S9+S10)$.

c) TRIBUTARY SPAN IN Z DIRECTION. SUPPORTS C,D & E ONLY

SUP'T "C" $ZC=(S1+S2+S3+S4)+S5/2$

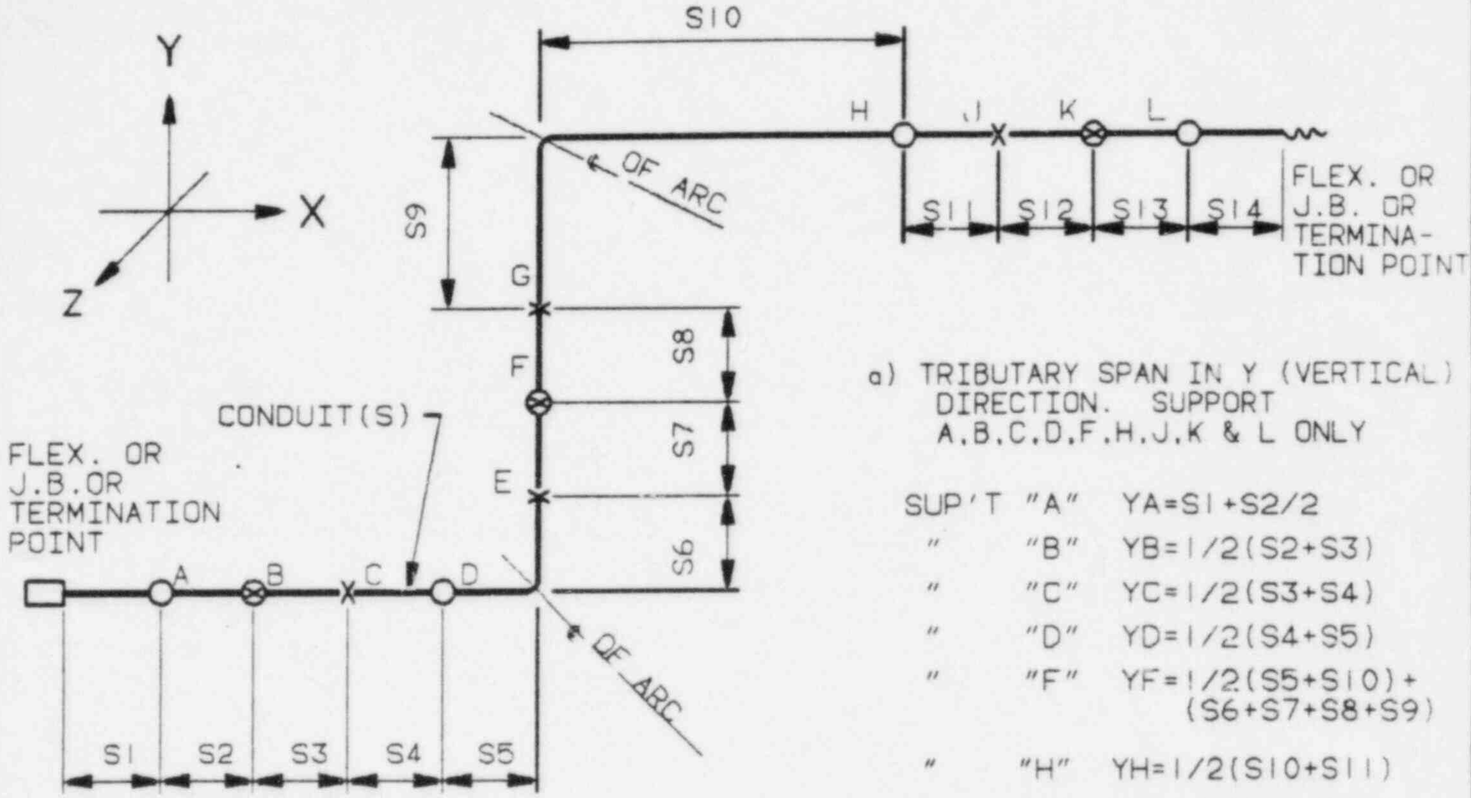
SUP'T "D" $ZD=1/2\ (S5+S6)$

SUP'T "E" $ZE=S6/2+(S7+S8+S9+S10)$

SUPPORT LOAD=TRIBUTARY SPAN x TOTAL CONDUIT UNIT WEIGHT.

SUPPORT LOAD:

B) SCHEME V - VERTICAL RUN SUPPORTED BY ALL THREE SUPPORT TYPES.



ELEVATION

a) TRIBUTARY SPAN IN Y (VERTICAL) DIRECTION. SUPPORT A,B,C,D,F,H,J,K & L ONLY

SUP'T "A" YA=S1+S2/2
" "B" YB=1/2(S2+S3)
" "C" YC=1/2(S3+S4)
" "D" YD=1/2(S4+S5)
" "F" YF=1/2(S5+S10)+ (S6+S7+S8+S9)
" "H" YH=1/2(S10+S11)
" "J" YJ=1/2(S11+S12)
" "K" YK=1/2(S12+S13)
" "L" YL=S13/2+S14

b) TRIBUTARY SPAN IN X DIRECTION. SUPPORTS B,E,F,G & K ONLY

SUP'T "B" XB=(S1+S2+S3+S4+S5)+S6/2
SUP'T "E" XE=1/2(S6+S7)
SUP'T "F" XF=1/2(S7+S8)
SUP'T "G" XG=1/2(S8+S9)
SUP'T "K" XK=S9/2+(S10+S11+S12+ S13+S14)

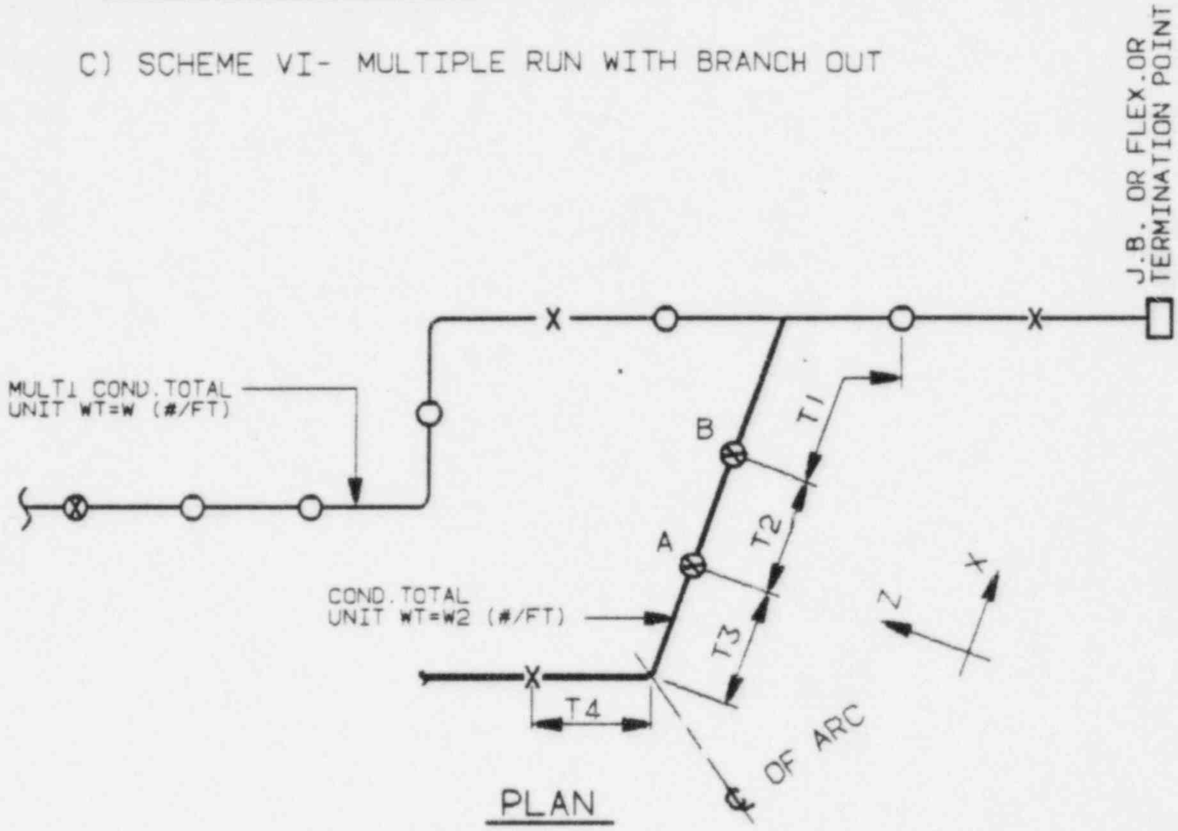
c) TRIBUTARY SPAN IN Z DIRECTION. SUPPORTS B,C,E,F,G,J & K ONLY

SUP'T "B" ZB=(S1+S2)+S3/2
SUP'T "C" ZC= 1/2(S3+S4+S5+S6)
SUP'T "E" ZE= 1/2(S4+S5+S6+S7)
SUP'T "F" ZF= 1/2(S7+S8)
SUP'T "G" ZG= 1/2(S8+S9+S10+S11)
SUP'T "J" ZJ= 1/2(S9+S10+S11+S12)
SUP'T "K" ZK= S12/2+(S13+ S14)

SUPPORT LOAD= TRIBUTARY SPAN × TOTAL CONDUIT UNIT WEIGHT

SUPPORT LOAD:

C) SCHEME VI- MULTIPLE RUN WITH BRANCH OUT



PLAN

a) TRIBUTARY LOAD IN Z DIRECTION

SUP'T "A" ZA=1/2(T2+T3+T4) W2 + WAZ

" "B" ZB=1/2(T1+T2)W2 + WBZ

WAZ, WBZ = NORMALIZED TRIBUTARY LOADS INDUCED BY TRAPEZE SUPPORT (SEE TABLES 1 TO 6)

b) TRIBUTARY LOAD IN X DIRECTION

SUP'T "A" XA=(T4+T3+T2/2)W2+ WAX

SUP'T "B" XB=1/2(T2+T1)W2 +WBX

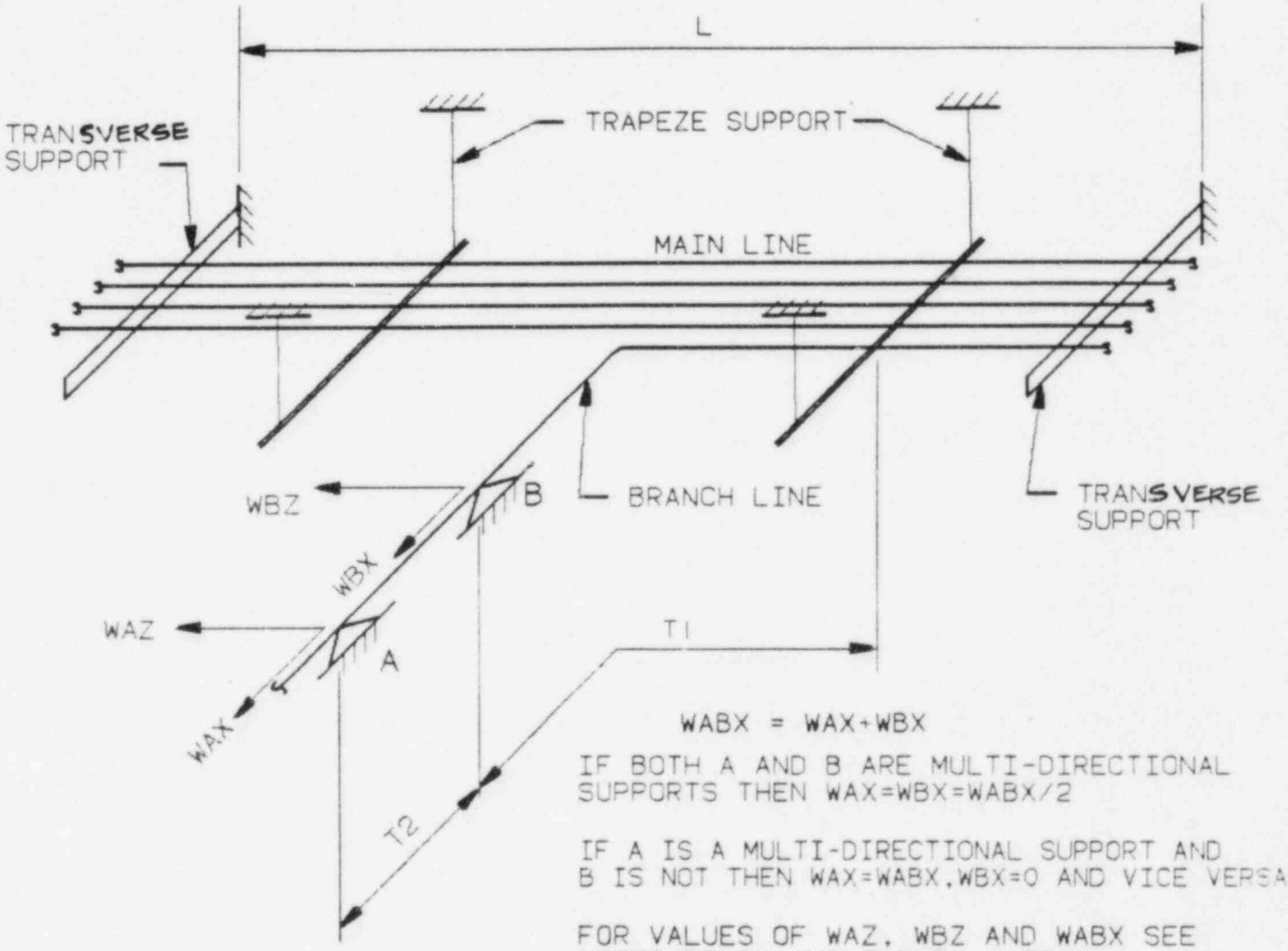
WAX, WBX = NORMALIZED TRIBUTARY LOADS INDUCED BY TRAPEZE SUPPORT (SEE TABLES 1 TO 6)

c) TRIBUTARY LOAD IN Y (VERTICAL) DIRECTION

SUP'T "A" YA=1/2(T2+T3+T4)W2

SUP'T "B" YB=1/2(T1+T2)W2

NORMALIZED TRIBUTARY LOADS AT BRANCH SUPPORTS INDUCED BY TRAPEZE SUPPORTS IN MAIN LINE.



WABX = WAX+WBX

IF BOTH A AND B ARE MULTI-DIRECTIONAL SUPPORTS THEN WAX=WBX=WABX/2

IF A IS A MULTI-DIRECTIONAL SUPPORT AND B IS NOT THEN WAX=WABX, WBX=0 AND VICE VERSA

FOR VALUES OF WAZ, WBZ AND WABX SEE NOTES BELOW AND TABLES 1 TO 6

MAIN LINE:

L - SPAN LENGTH BETWEEN TWO TRANVERSE OR MULTI-DIRECTIONAL SUPPORTS

W - TOTAL UNIT WEIGHT (WEIGHT PER UNIT LENGTH)

BRANCH LINE:

T1, T2 - CONDUIT LENGTHS

D - CONDUIT SIZE (SEE NOTES BELOW)

NOTES:

1. IF THE BRANCH LINE HAS MORE THAN ONE CONDUIT, THEN THE LOADS SHOULD BE INTERPOLATED OR EXTRAPOLATED BASED ON THE TOTAL CROSS SECTIONAL MOMENT OF INERTIA OF THE CONDUITS SHOWN IN THE TABLES DISREGARDING THE CONDUIT SIZES.
2. THE EFFECTIVE REACTION AT THE SUPPORT A OR B IS THE PRODUCT OF THE RELEVANT TRIBUTARY LOAD AND THE CORRESPONDING PEAK SEISMIC ACCELERATION.

NORMALIZED TRIBUTARY LOADS AT BRANCH SUPPORTS INDUCED BY TRAPEZE SUPPORTS IN MAIN LINE

CONDUIT SIZE D (IN)	MOMENT OF INERTIA (IN**4)	LENGTHS T1 (FT)	T2 (FT)	WAZ = awL + bw + cL + d a -	b (FT)	c (LB/FT)	d (LB)
0.75	0.0370	2.	2.	0.1004	-0.21	1.97	31.
0.75	0.0370	4.	2.	0.0201	0.57	2.14	7.
0.75	0.0370	6.	2.	0.0163	0.64	1.33	13.
0.75	0.0370	8.	2.	0.0205	0.57	0.92	16.
0.75	0.0370	10.	2.	0.0191	0.63	0.77	18.
0.75	0.0370	2.	4.	0.0332	-0.05	0.94	9.
0.75	0.0370	4.	4.	0.0062	0.22	0.89	2.
0.75	0.0370	6.	4.	0.0059	0.24	0.55	4.
0.75	0.0370	8.	4.	0.0088	0.20	0.38	6.
0.75	0.0370	10.	4.	0.0091	0.20	0.30	7.
0.75	0.0370	2.	6.	0.0160	-0.01	0.58	4.
0.75	0.0370	4.	6.	0.0028	0.12	0.51	1.
0.75	0.0370	6.	6.	0.0030	0.13	0.31	2.
0.75	0.0370	8.	6.	0.0050	0.10	0.21	3.
0.75	0.0370	10.	6.	0.0055	0.10	0.18	4.
0.75	0.0370	2.	8.	0.0092	0.0	0.40	2.
0.75	0.0370	4.	8.	0.0015	0.08	0.33	0.
0.75	0.0370	6.	8.	0.0018	0.08	0.21	1.
0.75	0.0370	8.	8.	0.0033	0.06	0.14	2.
0.75	0.0370	10.	8.	0.0037	0.06	0.12	2.
0.75	0.0370	2.	10.	0.0058	0.01	0.30	1.
0.75	0.0370	4.	10.	0.0010	0.06	0.24	0.
0.75	0.0370	6.	10.	0.0012	0.06	0.15	1.
0.75	0.0370	8.	10.	0.0023	0.04	0.10	2.
0.75	0.0370	10.	10.	0.0027	0.04	0.08	2.

CONDUIT SIZE D (IN)	MOMENT OF INERTIA (IN**4)	LENGTHS T1 (FT)	T2 (FT)	WBZ = awL + bw + cL + d a -	b (FT)	c (LB/FT)	d (LB)
0.75	0.0370	2.	2.	0.1703	1.61	4.11	62.
0.75	0.0370	4.	2.	0.0883	2.39	2.73	66.
0.75	0.0370	6.	2.	0.0328	3.23	2.43	65.
0.75	0.0370	8.	2.	0.0251	3.33	2.06	67.
0.75	0.0370	10.	2.	0.0237	3.31	1.90	67.
0.75	0.0370	2.	4.	0.1434	2.77	1.33	90.
0.75	0.0370	4.	4.	0.0637	2.24	3.15	56.
0.75	0.0370	6.	4.	0.0285	2.48	2.66	48.
0.75	0.0370	8.	4.	0.0207	2.46	2.17	46.
0.75	0.0370	10.	4.	0.0181	2.42	1.84	46.
0.75	0.0370	2.	6.	0.1695	2.40	1.05	94.
0.75	0.0370	4.	6.	0.0651	2.01	3.36	51.
0.75	0.0370	6.	6.	0.0284	2.15	2.80	40.
0.75	0.0370	8.	6.	0.0190	2.11	2.21	38.
0.75	0.0370	10.	6.	0.0159	2.05	1.82	38.
0.75	0.0370	2.	8.	0.1824	2.24	0.98	95.
0.75	0.0370	4.	8.	0.0678	1.83	3.49	47.
0.75	0.0370	6.	8.	0.0283	1.97	2.87	36.
0.75	0.0370	8.	8.	0.0181	1.92	2.23	34.
0.75	0.0370	10.	8.	0.0146	1.85	1.80	33.
0.75	0.0370	2.	10.	0.1908	2.13	0.94	95.
0.75	0.0370	4.	10.	0.0696	1.72	3.57	45.
0.75	0.0370	6.	10.	0.0282	1.85	2.92	33.
0.75	0.0370	8.	10.	0.0175	1.80	2.25	31.
0.75	0.0370	10	10.	0.0138	1.71	1.80	30.

TABLE 2

NORMALIZED TRIBUTARY LOADS AT BRANCH SUPPORTS INDUCED BY TRAPEZE SUPPORTS IN MAIN LINE

CONDUIT SIZE D (IN)	MOMENT OF INERTIA (IN**4)	LENGTHS T1 (FT)	T2 (FT)	WABX = awL + bw + cL + d a -	b (FT)	c (LB/FT)	d (LB)
0.75	0.0370	2.	2.	0.2534	1.46	2.33	124.
0.75	0.0370	4.	2.	0.2337	1.54	3.76	106.
0.75	0.0370	6.	2.	0.2093	1.32	4.41	91.
0.75	0.0370	8.	2.	0.1816	1.55	4.67	81.
0.75	0.0370	10.	2.	0.1572	1.86	4.84	73.
0.75	0.0370	2.	4.	0.2433	1.48	2.73	118.
0.75	0.0370	4.	4.	0.2295	1.56	3.97	103.
0.75	0.0370	6.	4.	0.2057	1.31	4.49	89.
0.75	0.0370	8.	4.	0.1773	1.61	4.71	79.
0.75	0.0370	10.	4.	0.1552	1.87	4.84	72.
0.75	0.0370	2.	6.	0.2368	1.50	2.99	115.
0.75	0.0370	4.	6.	0.2268	1.58	4.10	101.
0.75	0.0370	6.	6.	0.2032	1.32	4.54	88.
0.75	0.0370	8.	6.	0.1743	1.64	4.74	78.
0.75	0.0370	10.	6.	0.1548	1.85	4.83	71.
0.75	0.0370	2.	8.	0.2320	1.53	3.19	113.
0.75	0.0370	4.	8.	0.2254	1.58	4.19	99.
0.75	0.0370	6.	8.	0.2013	1.34	4.58	87.
0.75	0.0370	8.	8.	0.1722	1.67	4.75	78.
0.75	0.0370	10.	8.	0.1544	1.84	4.83	71.
0.75	0.0370	2.	10.	0.2282	1.55	3.36	110.
0.75	0.0370	4.	10.	0.2246	1.58	4.25	99.
0.75	0.0370	6.	10.	0.1998	1.36	4.61	87.
0.75	0.0370	8.	10.	0.1706	1.69	4.77	77.
0.75	0.0370	10.	10.	0.1542	1.83	4.82	71.

CONDUIT SIZE D (IN)	MOMENT OF INERTIA (IN**4)	LENGTHS T1 (FT)	T2 (FT)	WAZ = awL + bw + cL + d a -	b (FT)	c (LB/FT)	d (LB)
1.00	0.0874	2.	2.	0.1991	-0.82	1.49	81.
1.00	0.0874	4.	2.	0.0709	-0.07	3.37	13.
1.00	0.0874	6.	2.	0.0230	0.84	2.75	6.
1.00	0.0874	8.	2.	0.0223	0.88	1.94	13.
1.00	0.0874	10.	2.	0.0275	0.78	1.48	17.
1.00	0.0874	2.	4.	0.0738	-0.34	0.92	27.
1.00	0.0874	4.	4.	0.0242	0.02	1.48	3.
1.00	0.0874	6.	4.	0.0080	0.33	1.16	1.
1.00	0.0874	8.	4.	0.0092	0.33	0.80	5.
1.00	0.0874	10.	4.	0.0127	0.26	0.60	7.
1.00	0.0874	2.	6.	0.0385	-0.18	0.65	13.
1.00	0.0874	4.	6.	0.0118	0.03	0.88	0.
1.00	0.0874	6.	6.	0.0039	0.19	0.68	0.
1.00	0.0874	8.	6.	0.0049	0.19	0.46	3.
1.00	0.0874	10.	6.	0.0075	0.14	0.34	4.
1.00	0.0874	2.	8.	0.0234	-0.11	0.49	8.
1.00	0.0874	4.	8.	0.0069	0.03	0.59	-0.
1.00	0.0874	6.	8.	0.0022	0.13	0.45	0.
1.00	0.0874	8.	8.	0.0032	0.12	0.31	2.
1.00	0.0874	10.	8.	0.0050	0.09	0.23	3.
1.00	0.0874	2.	10.	0.0156	-0.07	0.39	5.
1.00	0.0874	4.	10.	0.0044	0.03	0.43	-0.
1.00	0.0874	6.	10.	0.0013	0.09	0.33	0.
1.00	0.0874	8.	10.	0.0021	0.09	0.23	1.
1.00	0.0874	10.	10.	0.0036	0.06	0.16	2.

TABLE

NORMALIZED TRIBUTARY LOADS AT BRANCH SUPPORTS INDUCED BY
TRAPEZE SUPPORTS IN MAIN LINE

CONDUIT SIZE D (IN)	MOMENT OF INERTIA (IN**4)	LENGTHS T1 (FT)	T2 (FT)	WBZ = awL + bw + cL + d a -	b (FT)	c (LB/FT)	d (LB)
1.00	0.0874	2.	2.	0.3939	-1.57	3.29	166.
1.00	0.0874	4.	2.	0.0864	3.20	4.85	53.
1.00	0.0874	6.	2.	0.0743	2.99	3.35	64.
1.00	0.0874	8.	2.	0.0364	3.92	2.75	72.
1.00	0.0874	10.	2.	0.0286	4.14	2.30	81.
1.00	0.0874	2.	4.	0.1869	1.11	3.01	80.
1.00	0.0874	4.	4.	0.1199	1.68	2.30	83.
1.00	0.0874	6.	4.	0.0437	2.91	2.93	56.
1.00	0.0874	8.	4.	0.0266	3.21	2.38	60.
1.00	0.0874	10.	4.	0.0245	3.20	2.12	62.
1.00	0.0874	2.	6.	0.1231	3.33	1.94	87.
1.00	0.0874	4.	6.	0.1146	1.57	2.77	76.
1.00	0.0874	6.	6.	0.0418	2.68	3.05	53.
1.00	0.0874	8.	6.	0.0277	2.82	2.62	51.
1.00	0.0874	10.	6.	0.0231	2.83	2.29	51.
1.00	0.0874	2.	8.	0.1518	3.02	0.82	114.
1.00	0.0874	4.	8.	0.1102	1.59	3.13	72.
1.00	0.0874	6.	8.	0.0429	2.50	3.26	49.
1.00	0.0874	8.	8.	0.0278	2.62	2.79	45.
1.00	0.0874	10.	8.	0.0223	2.61	2.39	45.
1.00	0.0874	2.	10.	0.1690	2.82	0.63	120.
1.00	0.0874	4.	10.	0.1073	1.61	3.38	68.
1.00	0.0874	6.	10.	0.0442	2.37	3.42	45.
1.00	0.0874	8.	10.	0.0278	2.48	2.90	41.
1.00	0.0874	10.	10.	0.0217	2.47	2.45	41.

CONDUIT SIZE D (IN)	MOMENT OF INERTIA (IN**4)	LENGTHS T1 (FT)	T2 (FT)	WABX = awL + bw + cL + d a -	b (FT)	c (LB/FT)	d (LB)
1.00	0.0874	2.	2.	0.3152	2.23	0.64	183.
1.00	0.0874	4.	2.	0.2925	3.07	1.86	164.
1.00	0.0874	6.	2.	0.2814	1.87	2.92	142.
1.00	0.0874	8.	2.	0.2640	1.23	3.58	125.
1.00	0.0874	10.	2.	0.2488	0.82	3.96	113.
1.00	0.0874	2.	4.	0.3017	1.93	1.15	171.
1.00	0.0874	4.	4.	0.2894	3.05	2.10	160.
1.00	0.0874	6.	4.	0.2781	1.81	3.09	139.
1.00	0.0874	8.	4.	0.2612	1.14	3.68	122.
1.00	0.0874	10.	4.	0.2465	0.78	4.00	111.
1.00	0.0874	2.	6.	0.2916	1.78	1.54	163.
1.00	0.0874	4.	6.	0.2871	3.05	2.26	157.
1.00	0.0874	6.	6.	0.2758	1.77	3.19	136.
1.00	0.0874	8.	6.	0.2589	1.10	3.75	120.
1.00	0.0874	10.	6.	0.2448	0.75	4.03	110.
1.00	0.0874	2.	8.	0.2839	1.69	1.83	156.
1.00	0.0874	4.	8.	0.2851	3.06	2.40	155.
1.00	0.0874	6.	8.	0.2741	1.75	3.27	135.
1.00	0.0874	8.	8.	0.2572	1.08	3.81	119.
1.00	0.0874	10.	8.	0.2435	0.74	4.05	109.
1.00	0.0874	2.	10.	0.2779	1.63	2.05	152.
1.00	0.0874	4.	10.	0.2834	3.07	2.51	153.
1.00	0.0874	6.	10.	0.2728	1.73	3.34	133.
1.00	0.0874	8.	10.	0.2559	1.06	3.85	118.
1.00	0.0874	10.	10.	0.2425	0.72	4.07	108.

NORMALIZED TRIBUTARY LOADS AT BRANCH SUPPORTS INDUCED BY
TRAPEZE SUPPORTS IN MAIN LINE

CONDUIT	MOMENT OF	LENGTHS		WAZ = awL + bw + cL + d			
SIZE D	INERTIA	T1	T2	a	b	c	d
(IN)	(IN**4)	(FT)	(FT)	-	(FT)	(LB/FT)	(LB)
1.50	0.3100	2.	2.	0.3358	0.91	-0.91	211.
1.50	0.3100	4.	2.	0.2649	-2.48	4.06	87.
1.50	0.3100	6.	2.	0.1232	-0.44	5.46	11.
1.50	0.3100	8.	2.	0.0464	1.58	4.98	-5.
1.50	0.3100	10.	2.	0.0245	2.11	4.03	4.
1.50	0.3100	2.	4.	0.1491	-0.13	-0.01	85.
1.50	0.3100	4.	4.	0.1021	-0.94	2.07	28.
1.50	0.3100	6.	4.	0.0469	-0.13	2.49	0.
1.50	0.3100	8.	4.	0.0180	0.65	2.22	-5.
1.50	0.3100	10.	4.	0.0088	0.91	1.80	-1.
1.50	0.3100	2.	6.	0.0883	-0.23	0.19	47.
1.50	0.3100	4.	6.	0.0549	-0.50	1.36	13.
1.50	0.3100	6.	6.	0.0251	-0.06	1.52	-2.
1.50	0.3100	8.	6.	0.0096	0.38	1.33	-4.
1.50	0.3100	10.	6.	0.0043	0.54	1.09	-2.
1.50	0.3100	2.	8.	0.0593	-0.22	0.24	30.
1.50	0.3100	4.	8.	0.0346	-0.31	0.98	7.
1.50	0.3100	6.	8.	0.0155	-0.03	1.05	-2.
1.50	0.3100	8.	8.	0.0059	0.25	0.91	-3.
1.50	0.3100	10.	8.	0.0025	0.36	0.75	-2.
1.50	0.3100	2.	10.	0.0428	-0.19	0.25	21.
1.50	0.3100	4.	10.	0.0237	-0.21	0.76	4.
1.50	0.3100	6.	10.	0.0105	-0.01	0.78	-2.
1.50	0.3100	8.	10.	0.0039	0.18	0.67	-3.
1.50	0.3100	10.	10.	0.0015	0.26	0.55	-1.

CONDUIT	MOMENT OF	LENGTHS		WBZ = awL + bw + cL + d			
SIZE D	INERTIA	T1	T2	a	b	c	d
(IN)	(IN**4)	(FT)	(FT)	-	(FT)	(LB/FT)	(LB)
1.50	0.3100	2.	2.	0.6844	0.63	-1.43	431.
1.50	0.3100	4.	2.	0.3864	-2.34	6.43	152.
1.50	0.3100	6.	2.	0.1391	3.10	7.45	59.
1.50	0.3100	8.	2.	0.0797	3.97	6.18	61.
1.50	0.3100	10.	2.	0.0694	3.92	5.21	70.
1.50	0.3100	2.	4.	0.4575	-1.53	0.39	260.
1.50	0.3100	4.	4.	0.1624	1.65	4.49	72.
1.50	0.3100	6.	4.	0.0889	2.97	3.51	67.
1.50	0.3100	8.	4.	0.0783	2.62	3.06	66.
1.50	0.3100	10.	4.	0.0467	3.43	2.80	67.
1.50	0.3100	2.	6.	0.3578	-1.81	1.19	189.
1.50	0.3100	4.	6.	0.0951	3.58	3.21	69.
1.50	0.3100	6.	6.	0.1093	1.94	2.48	80.
1.50	0.3100	8.	6.	0.0654	2.57	2.92	56.
1.50	0.3100	10.	6.	0.0287	3.52	2.76	54.
1.50	0.3100	2.	8.	0.2891	-1.30	1.64	147.
1.50	0.3100	4.	8.	0.1233	2.92	1.76	104.
1.50	0.3100	6.	8.	0.1108	1.59	2.72	75.
1.50	0.3100	8.	8.	0.0523	2.81	3.16	51.
1.50	0.3100	10.	8.	0.0272	3.41	2.82	53.
1.50	0.3100	2.	10.	0.2288	0.16	1.92	116.
1.50	0.3100	4.	10.	0.1395	2.46	1.63	112.
1.50	0.3100	6.	10.	0.1093	1.53	2.94	72.
1.50	0.3100	8.	10.	0.0496	2.81	3.32	49.
1.50	0.3100	10.	10.	0.0292	3.25	2.89	52.

NORMALIZED TRIBUTARY LOADS AT BRANCH SUPPORTS INDUCED BY
TRAPEZE SUPPORTS IN MAIN LINE

CONDUIT SIZE D (IN)	MOMENT OF INERTIA (IN**4)	LENGTHS T1 (FT)	T2 (FT)	WABX = awL + bw + cL + d a -	b (FT)	c (LB/FT)	d (LB)
1.50	0.3100	2.	2.	0.3008	10.56	-2.04	285.
1.50	0.3100	4.	2.	0.2871	10.63	-1.22	268.
1.50	0.3100	6.	2.	0.2790	10.40	-0.39	253.
1.50	0.3100	8.	2.	0.2872	8.47	0.41	236.
1.50	0.3100	10.	2.	0.2957	6.50	1.05	221.
1.50	0.3100	2.	4.	0.3158	8.30	-1.64	276.
1.50	0.3100	4.	4.	0.2834	10.76	-1.05	265.
1.50	0.3100	6.	4.	0.2778	10.28	-0.22	249.
1.50	0.3100	8.	4.	0.2881	8.15	0.56	233.
1.50	0.3100	10.	4.	0.2971	6.17	1.16	217.
1.50	0.3100	2.	6.	0.3220	6.85	-1.29	267.
1.50	0.3100	4.	6.	0.2803	10.87	-0.90	262.
1.50	0.3100	6.	6.	0.2769	10.20	-0.08	247.
1.50	0.3100	8.	6.	0.2886	7.93	0.68	231.
1.50	0.3100	10.	6.	0.2980	5.94	1.25	215.
1.50	0.3100	2.	8.	0.3238	5.86	-0.97	258.
1.50	0.3100	4.	8.	0.2771	10.97	-0.74	259.
1.50	0.3100	6.	8.	0.2756	10.15	0.05	244.
1.50	0.3100	8.	8.	0.2888	7.77	0.77	229.
1.50	0.3100	10.	8.	0.2987	5.76	1.31	213.
1.50	0.3100	2.	10.	0.3234	5.15	-0.69	250.
1.50	0.3100	4.	10.	0.2744	11.05	-0.61	257.
1.50	0.3100	6.	10.	0.2745	10.13	0.15	242.
1.50	0.3100	8.	10.	0.2888	7.66	0.85	227.
1.50	0.3100	10.	10.	0.2990	5.64	1.37	212.

CONDUIT SIZE D (IN)	MOMENT OF INERTIA (IN**4)	LENGTHS T1 (FT)	T2 (FT)	WAZ = awL + bw + cL + d a -	b (FT)	c (LB/FT)	d (LB)
2.00	0.6660	2.	2.	0.3953	4.22	-2.35	295.
2.00	0.6660	4.	2.	0.4567	-3.83	3.29	189.
2.00	0.6660	6.	2.	0.2799	-3.00	7.03	45.
2.00	0.6660	8.	2.	0.1406	-0.05	7.55	-11.
2.00	0.6660	10.	2.	0.0654	2.13	6.84	-22.
2.00	0.6660	2.	4.	0.1909	0.78	-0.70	127.
2.00	0.6660	4.	4.	0.1865	-1.63	1.99	67.
2.00	0.6660	6.	4.	0.1112	-1.14	3.37	11.
2.00	0.6660	8.	4.	0.0557	0.04	3.48	-10.
2.00	0.6660	10.	4.	0.0256	0.94	3.10	-13.
2.00	0.6660	2.	6.	0.1204	0.08	-0.24	74.
2.00	0.6660	4.	6.	0.1046	-0.92	1.44	33.
2.00	0.6660	6.	6.	0.0613	-0.61	2.13	3.
2.00	0.6660	8.	6.	0.0306	0.04	2.14	-8.
2.00	0.6660	10.	6.	0.0136	0.56	1.90	-9.
2.00	0.6650	2.	8.	0.0851	-0.13	-0.05	50.
2.00	0.6660	4.	8.	0.0678	-0.60	1.10	19.
2.00	0.6660	6.	8.	0.0391	-0.38	1.52	-0.
2.00	0.6660	8.	8.	0.0194	0.04	1.50	-7.
2.00	0.6660	10.	8.	0.0084	0.39	1.32	-7.
2.00	0.6660	2.	10.	0.0641	-0.19	0.04	36.
2.00	0.6660	4.	10.	0.0478	-0.42	0.89	12.
2.00	0.6660	6.	10.	0.0272	-0.26	1.16	-1.
2.00	0.6660	8.	10.	0.0134	0.04	1.12	-6.
2.00	0.6660	10.	10.	0.0056	0.28	0.99	-6.

TABLE 6

NORMALIZED TRIBUTARY LOADS AT BRANCH SUPPORTS INDUCED BY TRAPEZE SUPPORTS IN MAIN LINE

CONDUIT SIZE D (IN)	MOMENT OF INERTIA (IN**4)	LENGTHS T1 (FT)	T2 (FT)	WBZ = awL + bw + cL + d a -	b (FT)	c (LB/FT)	d (LB)
2.00	0.6660	2.	2.	0.8101	6.77	-4.34	603.
2.00	0.6660	4.	2.	0.7037	-7.19	5.37	291.
2.00	0.6660	6.	2.	0.3548	-1.50	9.63	88.
2.00	0.6660	8.	2.	0.1610	3.36	9.72	28.
2.00	0.6660	10.	2.	0.0922	4.69	8.49	31.
2.00	0.6660	2.	4.	0.5926	0.49	-1.68	389.
2.00	0.6660	4.	4.	0.3728	-3.18	4.41	146.
2.00	0.6660	6.	4.	0.1502	2.06	5.82	51.
2.00	0.6660	8.	4.	0.0793	3.43	5.12	46.
2.00	0.6660	10.	4.	0.0803	2.86	4.15	61.
2.00	0.6660	2.	6.	0.4980	-1.44	-0.49	300.
2.00	0.6660	4.	6.	0.2336	-0.22	4.05	91.
2.00	0.6660	6.	6.	0.0893	3.43	4.15	51.
2.00	0.6660	8.	6.	0.0999	2.28	3.06	67.
2.00	0.6660	10.	6.	0.0778	2.54	2.98	62.
2.00	0.6660	2.	8.	0.4368	-2.19	0.25	247.
2.00	0.6660	4.	8.	0.1572	1.98	3.63	70.
2.00	0.6660	6.	8.	0.1081	2.81	2.62	80.
2.00	0.6660	8.	8.	0.1070	1.69	2.70	70.
2.00	0.6660	10.	8.	0.0661	2.57	3.03	52.
2.00	0.6660	2.	10.	0.3890	-2.35	0.79	208.
2.00	0.6660	4.	10.	0.1149	3.40	3.15	72.
2.00	0.6660	6.	10.	0.1291	2.09	2.18	93.
2.00	0.6660	8.	10.	0.1033	1.61	2.90	66.
2.00	0.6660	10.	10.	0.0558	2.79	3.21	47.

CONDUIT SIZE D (IN)	MOMENT OF INERTIA (IN**4)	LENGTHS T1 (FT)	T2 (FT)	WABX = awL + bw + cL + d a -	b (FT)	c (LB/FT)	d (LB)
2.00	0.6660	2.	2.	0.2784	18.43	-3.33	346.
2.00	0.6660	4.	2.	0.2985	15.80	-2.73	336.
2.00	0.6660	6.	2.	0.2833	16.63	-2.23	326.
2.00	0.6660	8.	2.	0.2845	15.65	-1.51	310.
2.00	0.6660	10.	2.	0.2986	13.54	-0.91	297.
2.00	0.6660	2.	4.	0.3161	14.68	-3.05	342.
2.00	0.6660	4.	4.	0.2977	15.68	-2.59	333.
2.00	0.6660	6.	4.	0.2799	16.73	-2.05	322.
2.00	0.6660	8.	4.	0.2849	15.41	-1.36	307.
2.00	0.6660	10.	4.	0.3011	13.13	-0.77	294.
2.00	0.6660	2.	6.	0.3383	12.12	-2.75	336.
2.00	0.6660	4.	6.	0.2968	15.60	-2.47	331.
2.00	0.6660	6.	6.	0.2769	16.82	-1.90	319.
2.00	0.6660	8.	6.	0.2850	15.25	-1.24	304.
2.00	0.6660	10.	6.	0.3028	12.84	-0.65	291.
2.00	0.6660	2.	8.	0.3518	10.27	-2.47	328.
2.00	0.6660	4.	8.	0.2959	15.54	-2.36	329.
2.00	0.6660	6.	8.	0.2745	16.89	-1.78	316.
2.00	0.6660	8.	8.	0.2850	15.13	-1.15	302.
2.00	0.6660	10.	8.	0.3040	12.61	-0.57	289.
2.00	0.6660	2.	10.	0.3602	8.86	-2.21	321.
2.00	0.6660	4.	10.	0.2949	15.50	-2.26	327.
2.00	0.6660	6.	10.	0.2725	16.94	-1.68	314.
2.00	0.6660	8.	10.	0.2848	15.04	-1.06	300.
2.00	0.6660	10.	10.	0.3049	12.44	-0.49	287.