

UNITED STATES OF AMERICA  
NUCLEAR REGULATORY COMMISSION

BEFORE THE ATOMIC SAFETY AND LICENSING BOARD

In the Matter of	)	
	)	Docket Nos. 50-445 and
TEXAS UTILITIES ELECTRIC	)	50-446
COMPANY, ET AL.	)	
	)	(Application for
(Comanche Peak Steam Electric	)	Operating Licenses)
Station, Units 1 and 2)	)	

AFFIDAVIT OF ROBERT C. IOTTI AND  
JOHN C. FINNERAN, JR. REGARDING THE  
EFFECTS OF GAPS ON STRUCTURAL  
BEHAVIOR UNDER SEISMIC LOADING CONDITIONS

I, Robert C. Iotti, having been first duly sworn hereby depose and state, as follows: I am Chief Engineer of Applied Physics for Ebasco Services, Inc. In this position I am responsible for directing analytical work in diverse technical areas, including analyses of the response of piping and support systems to dynamic events, including earthquakes. I have been retained by Texas Utilities Generating Company to coordinate and oversee the technical activities performed to respond to the Licensing Board's December 28, 1983, Memorandum and Order (Quality Assurance for Design). A statement of my educational and professional qualifications was transmitted with Applicants' letter of May 16, 1984, to the Licensing Board in this proceeding.

I, John C. Finneran, Jr., hereby depose and state as follows: I am employed by Texas Utilities Generating Company as Project Pipe Support Engineer for the Comanche Peak Steam Electric Station. In this position I oversee the pipe support design activities of each organization performing pipe support design work for Comanche Peak. A statement of my educational and professional qualifications is in evidence as Applicants' Exhibit 142B.

Q. What is the purpose of this affidavit?

A. In this affidavit we address CASE's allegations regarding the effect of gaps (e.g., bolt hole tolerances) on the structural behavior of pipe supports under seismic loading conditions. This affidavit is in partial response to Item 9 of Applicants' Plan to respond to the Board's December 28, 1983, Memorandum and Order (Quality Assurance for Design).

Q. What is CASE's allegation regarding these effects?

A. CASE argues that bearing type connections are inappropriate for use as mechanisms for supporting structures during seismic events.

Q. What are the bases for this assertion?

A. There are two bases for CASE's argument. First, CASE argues that in a bearing type joint it is impossible to predict how many bolts are involved in the transfer of shear from the support to the wall.<sup>1</sup> Second, CASE argues that the presence

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<sup>1</sup> See CASE Proposed Findings of Fact and Conclusions of Law, Sections VII, XXI.

of gaps in the joints under dynamic conditions can be "disastrous."<sup>2</sup> In both instances, there is an underlying concern by CASE that the bolt holes in support base plates are "oversized," thus creating "gaps" that must be specially analyzed. We will address each of the issues separately. We will also put in perspective the question of "oversized" bolt holes.

Q. Before addressing these matters, please explain the difference between bearing and friction connections and the factors which seem to be of concern to CASE.

A. Bearing type connections are connections where shear forces between the two joined components (in most instances at Comanche Peak the base plates and the concrete), are reacted by bearing of the bolt surface on the surface of the bolt hole. Friction type connections are those where the same shear forces are expected to be fully reacted by friction forces created by preloading the connection bolts. For a baseplate, the friction forces exist between the baseplate and concrete surface and between the baseplate and bolt washer surface. It must be recognized that friction connections will ultimately revert to bearing connections when the shear load exceeds the friction developed at the interface plane. In the ultimate condition, all joints are bearing joints.

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<sup>2</sup> CASE Exhibit 763 at 3; Tr. 6605-6624.

Q. Do you agree with CASE's assertions concerning the distribution of shear in multiple bolt, bearing type connections?<sup>3</sup>

A. No. CASE's position is premised on an erroneous assessment of industry practice and bolt interaction theory. CASE argues that "... the usual procedure (industry practice) is to assume that two holes react the load regardless of the number of bolts in the pattern (for patterns of 4 bolts or more)". Not only is this not industry practice but it is contrary to sound engineering principles. This is readily apparent if one considers a pattern with, for example, twenty bolts. CASE would have one believe that it is industry practice to assume that only two bolts can react imposed shear loads. Obviously, if this were the case, and no more than two bolts could be counted on to react shear loads, every bolt in a multiple bolt pattern would have to be significantly oversized. If this were done, every connection designed for shear might as well be designed with only two bolts to begin with. This obviously is an illogical result, but one which flows directly from CASE's position.

CASE's argument is also contrary to sound engineering principles, recognized in authoritative texts concerning the design of bolted joints. "Plastic Design of Steel Frames,"

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<sup>3</sup> CASE Proposed Findings, Section VII at 10, 11, and Section XXI.



by Lynn S. Reedle (Attachment A), at pages 7-9, discusses the inherent plastic action that exists in elastic design. Therein it is noted (at 8) that there are "a number of examples . . . in which the ductility of steel has been counted upon in elastic design knowingly or not -- but certainly not through direct application of plastic design procedures." The author states (at 7) that "perhaps the outstanding example of this variance between elastic design assumptions and the actual truth is to be found in the ordinary riveted or bolted joint." The author goes on to explain that inelastic (plastic) action occurs to assure that all bolts will eventually participate in reacting the shear load.

Another text which addresses this matter is "Structural Design Guide to AISC Specifications for Buildings," authored by Paul F. Rice and Edward S. Hoffman (Attachment B). Rice and Hoffman explain (at 268) that in designing bolted connections for loading in shear "the use of an average capacity for each of the several connector elements sharing the total load is justified by allowing self-limiting localized stresses determined by an elastic joint analysis to exceed the yield point and create inelastic localized deformations of the connector materials, or by inelastic deformations of the connection elements (1.15.4)." "To illustrate this principle, Rice and Hoffman employ a two bolt example, designed to account for inelastic behavior

resulting from the cumulative effects of out-of-round holes, holes exceeding the bolt diameter by 1/16", and hole alignment. They conclude that both bolts share the load. In summary reliance on inelastic action in bolted connections to distribute shear actively to all bolts in the connection is a well recognized and valid assumption in elastic design.

Q. Do these excerpts agree with the positions taken by the NRC Staff in this proceeding?

A. Yes. As Dr. Chen stated (at Tr. 6884) "it is usual practice to assume that all the bolts will react equally to a shear load: the rationale being that even though initially one bolt or possibly two, let's say, out of a pattern of four will be engaged, some yielding of the first or second bolt will lead to equal load sharing eventually."

Q. Are the tolerances Applicants employ for bolt holes those normally characterized in the industry as "oversized" bolt holes?

A. No. The term "oversized" holes has a generally accepted meaning in the construction trade. The 8th Edition of the AISC Manual of Steel Construction is quite instructive on this point. At page 5-58 of the Manual, Paragraph 1.23.4.3 (Attachment C) states "Oversized holes may be used in any or all plies of friction-type connections, but they shall not be used in bearing-type connections." Table 1.23.4 on that

same page is quite clear concerning what the Code means by "oversized" holes. This table establishes tolerances for standard and oversized hole diameters.

Q. Do Applicants use "oversized" hole tolerances noted in the AISC Manual in the design of pipe support anchor bolts?

A. No. Applicants do not utilize the oversize hole tolerances for anchor bolts. Applicants use instead more stringent tolerances. Applicants' specified hole sizes meet the following requirements: holes for bolts up to 1" are to be  $d + 1/16$ " (where  $d$  is the diameter of the bolt), and holes for bolts 1" and over are to be  $d + 1/8$ ". AISC defines "oversized" as  $d + 3/16$ " for bolts up to and including  $7/8$ " diameter,  $d + 1/4$ " for 1" bolts, and  $d + 5/16$ " for bolts greater than or equal to 1 1/8" diameter. Although Applicants' criterion for bolts with diameters equal to or greater than 1" is 1/16" larger than the "standard" by AISC, it is 3/16" smaller than the AISC "oversized". Thus, Applicants' specifications for bolt hole tolerances can definitely not be called "oversized," as that term is generally used in the construction industry.

Q. What is the purpose of these bolt hole tolerances?

A. These types of tolerances are absolutely necessary to facilitate construction. As demonstrated in this affidavit, a reasoned consideration of the principles related to the distribution of loads in these connections demonstrates that such an approach is appropriate and acceptable.

Q. What test data are available regarding the capacity of anchor bolts to withstand the type of displacements which could occur in the transfer of shear loads?

A. There are two sets of data already in the record regarding bolt capacities in shear. Both Applicants' Exhibit 142D and the Cygna response to Doyle Question 16 contain data which demonstrate that bolts 1" and greater have more than enough capability to deflect a worst case 1/8" and still carry their full rated load capacity. (See Attachments B and C of Applicants' Exhibit 142D, and Enclosure D16-1 to Board Exhibit April 1984 No. 1 (Cygna testimony).) Attachment "B" of Applicants' Exhibit 142D indicates that the 1 1/4" super kwick Hilti bolt did not fail until approximately .7 inches displacement. Therefore, the inherent safety factor to failure for this bolt with a maximum displacement of .125" (1/8") would be .7 divided by .125, or 5.6. Enclosure D16-1 indicates failure of a 1" Hilti bolt at about .57" displacement. The safety factor inherent in this bolt would be .57 divided by .125, or 4.56. Further, Attachment "C" of Applicants' Exhibit 142D indicates a lower limit for slippage of a 1 1/4" Richmond Insert of about .4 inches. Thus, the safety factor in this case would be .4 divided by

.125 = 3.2. This test data is indicative of the displacement capacities of anchor bolts used at Comanche Peak. The data demonstrates that Applicants' bolts withstand the worst case slip of 1/8" that would be necessary to distribute shear forces equally to all bolts in the connection.

Q. Is there anything else you would like to add regarding CASE's assertions concerning the distribution of shear loads in multiple bolt, bearing connections?

A. Yes. During cross-examination of Cygna in the April 1984 hearings, Mr. Doyle introduced a paper by James M. Fisher.<sup>4</sup> Mr. Doyle used a portion of that paper (page 87, "Shear and Anchor Bolts") in support of his position. Mr. Doyle argues that Fisher concurs with CASE's position that "not all bolts in a cluster" may be used to transfer shear. However, Mr. Doyle apparently overlooked the fact that Mr. Fisher is talking about anchor bolts used at the base of columns. As shown on p. 4-126 of the AISC Manual (Attachment D), anchor bolts used at the base of columns which have 1-inch to 2-inch diameter are permitted to have 1/2 inch oversize holes. In addition, Mr. Fisher recommends on (p. 87) that bolt hole sizes for anchor bolts should be 1.33 times the bolt diameter. These tolerances are much greater than those employed on pipe supports at CPSES. It is clear that the condition described by Fisher is not the same condition

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<sup>4</sup> CASE Exhibit 1001: Tr. at 6605-6624.

applicable to designing joints for supports. This is best illustrated by Figure 1. Figure 1 shows ultimate deflections in shear and tension for different size Hilti bolts. Also shown in Figure 1 are the deflections that would be permitted if the bolts were loaded to their allowable values. The true interaction curve is approximated by a straight line between the ultimate displacement in tension and in shear. Also shown is the actual curve derived from combined shear-tension tests. Similar curves are available in the Teledyne Report "Generic Response to U.S. NRC I&E Bulletin 79-02 Base Plate/Concrete Expansion Anchor Bolts." Although the linear approximation of the interaction curve is not conservative for high values of tension displacements (which correspond to near ultimate pull-out loads), it is more than adequate for shear displacements, which are the concern here. To illustrate what these test data indicate, let us take a 1" Hilti embedded 10 1/2" deep, and compute the margin of safety when loaded to its allowable values in tension or shear. In either case there is a margin of safety of 5. Let us now assume that only one bolt out of many in the connection is loaded in shear, as CASE argues should be done. Let us further assume that it will deflect through a bolt hole gap of 1/16" before the other bolts begin to take some of the shear. This bolt alignment is the worst case condition for imposing shear stresses in the bolt. (See Illustration 1.)



Let us further calculate the deflection of the Hilti bolt, which would result from thermal expansion of the tubular frame. A worst case of .0485 inches deflection is computed by the method of Applicants' Exhibit 142D for a 1" bolt, embedded to 10 1/2", (see Figure 2 of Applicants' Exhibit 142D).

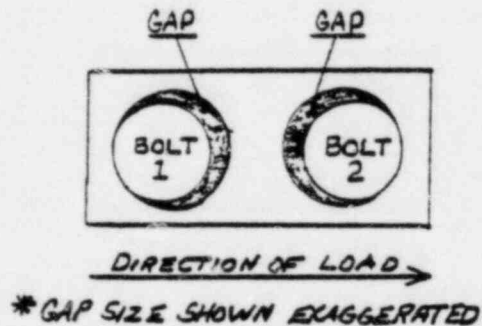


ILLUSTRATION 1

If the direction of the load is as shown in Illustration 1, bolt number 1 will take all the shear until it deflects through the gap. In this instance, the gap is .0625 inches (1/16"). If bolt number 1 is assumed to have already been loaded to its allowable value (7000 lbs) it would have deflected approximately .085 inches. Thus, the next bolt would have crossed the gap and started sharing the load, and the thermal expansion load would have already been totally relieved.

For the sake of being ultra-conservative, however, let us further assume a totally unrealistic scenario, i.e., the first bolt displaces as a result of its normal load, plus the thermal expansion load (which would have been relieved and thus is not truly additive to the other displacements),



plus the gap, before the second bolt engages. This would correspond to a gap for the second bolt of  $0.085 + .0485 + .0625 = 0.196$  inches. From Figure 1, even under this incredible condition, the margin of safety of the first bolt would be  $.55/.196 = 2.8$ . In reality, the margin of safety resulting from the  $1/16$ " bolt hole gap is 5 since the second bolt begins to share the load before the first bolt reaches the deflection at which the load would equal the allowable value. Even if the bolt holes are  $1/8$  larger than the bolt, the real factor of safety would still be 4.4. And if you were to play the same imaginary game of adding allowable displacement, thermal displacement and gap one would obtain a margin of safety of 2.1 ( $.55/.258$ ).

To illustrate why the Fisher paper made the recommendations it did, let us compare this situation to that examined in the paper. The gap condition examined by Fisher may be as high as  $1/2$  inch. The real margin of safety of the first bolt engaged would only be  $.55/.5$ , or 1.1, for a 1" bolt embedded  $10\ 1/2$  inches. This apparently is why Fisher recommends consideration of shear distribution on a limited number of bolts, rather than all bolts, where oversized holes such as he was examining may exist.

- Q. What is your response to CASE's concern<sup>5</sup> with the dynamic nature of the loadings?

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<sup>5</sup> CASE Exhibit 763 at 3. Tr. at 6605-6624.

A. CASE contends that the presence of gaps in joints under seismic conditions can be "disastrous." To the contrary, in a seismic event the first quarter cycle loading would cause preferentially loaded bolts to deflect in shear until the other bolts engage. Once the bolts have deflected, the gaps are uniform for all bolts. This is an over simplified explanation of what is a very complex phenomenon. To illustrate this point, refer to the above sketch. Bolt #1 will deflect until the second bolt engages (if the load is high enough to cause such deflection). As the load reverses, the locations of both bolts in the holes are now the same and both will take shear and deflect a like amount, and continue to do so, for the remainder of the dynamic event. Therefore, only during the first quarter cycle can there be preferentially loaded bolts. Thereafter, for subsequent cycles the load will be reacted by all bolts and there is no genuine concern for the capacity of the bolts to accept these loads.

Q. Are there any other matters that should be considered with respect to CASE's concern regarding the potentially adverse influence of gaps on a system's seismic response?<sup>6</sup>

A. Yes. It is important to recognize that the effect of gaps and other nonlinearities on the seismic response of systems cannot be defined in absolute terms. The effect is dependent on many factors, including the nature of the

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<sup>6</sup> Transcript at 13,705-13,717.

excitation (magnitude and distribution of frequencies), and the size, orientation and number of gaps. The situation is further complicated by the fact that nonlinearities introduce impacts and hence impact damping, which is known to be related to the coefficient of restitution ( $C_R$ ) via the formula:<sup>7</sup>

$$C_R = e^{-[2\pi\beta/\sqrt{1-\beta^2}]}$$

For steel on steel, average restitution coefficients of 0.6 would lead to impact damping factors ( $\beta$ ) of 8%, lower restitution coefficients would lead to progressively higher values of impact damping, and even with a very high restitution coefficient (0.8) the impact damping would be higher (3.5%) than the damping values recommended by the NRC in Regulatory Guide 1.61. This would mean that to account for just one of the effects of gaps, one would have to employ damping factors for portions of the system that exceed those specified by the regulations. Clearly, consideration of such effects would require complex analyses which depart from accepted practices.

Another fact that should be recognized is that while the gap is being transversed, little or no seismic input acceleration is being experienced. Also, depending on the nature of the gap (whether it is a gap in a bolt hole or the deadband in a mechanical snubber, or the play in a strut

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<sup>7</sup> HEDL-SA-1769, D.A. Barta, "Analyses of Piping Systems with Nonlinear Supports Subject to Seismic Loading", ASME P & PV Third National Congress, June 1979 (Attachment E).

assembly), a fraction of the seismic input may be introduced via friction (if permitted by the vertical excitation for a horizontal gap or vice versa). Thus, while transversing the gap, material damping takes place without a corresponding feed of energy from the seismic event. Obviously, the combination of intermittent energy input while damping continues produces a beneficial effect on the system response.

All of the above-described effects cannot be accounted for in the typical linear response spectrum analyses which are used to design the systems at CPSES, and are only accounted for with difficulty by performing nonlinear time history analyses. Thus, absolute generalizations as CASE contends should be made (CASE Proposed Findings at VII-11,12) simply are not possible.

Q. Is there a way to compare the results that would be obtained from a nonlinear analysis which considers the presence of gaps with those of the response spectrum analysis Applicants employ, in order to assess the effect of the gaps on seismic response?

A. In some respects, yes. However, in trying to compare results of nonlinear time history analyses with response spectrum analyses it becomes very difficult to distinguish between the effects of having conducted a full time history analyses (whether linear or nonlinear) versus a response spectrum analysis, and the effects of the gaps by

themselves. In other words, these two analytical approaches are sufficiently dissimilar that one may not discern whether particular results are attributable to differences in individual variables or assumptions (e.g., gaps) or the analytical techniques themselves.

In view of these uncertainties, it may be concluded that a more viable approach to assess the effect of gaps in designs which have been accomplished using response spectrum analyses (as in the case at CPSES), is to compare the support loads and pipe stresses predicted by the response spectrum (without gaps) with those which would be predicted by nonlinear time history analyses of each system.

To respond fully to this question, however, would be a virtually never ending and enormously expensive task.

Fortunately, a number of comparisons have been made between results obtained by response spectra analyses and nonlinear time history analyses which simulate the actual gaps in the system which provide reasonable evidence of the effect of gaps.

Two general conclusions can be derived from these comparative studies and tests which simulate seismic conditions in actual piping systems which are apt to have some gaps in the supports, as would be present in any piping support system. The conclusions which can be reached are that:



a) The seismic response spectrum method, which ignores the nonlinearities, is more conservative than the non-linear time domain method (which includes gaps), and

b) The effect of gaps on reduction of response frequency is negligible due to the transient nature of the seismic acceleration loading.

Studies bearing out these conclusions are discussed below.

The study prepared by Barta (footnote 7) compares results of the response spectrum analyses of an FFTF piping system to those of the non-linear analyses which modelled the deadbands (gaps) of mechanical snubbers. The gap used was 0.120 which is comparable to or in excess of that existing in supports. Table 2 in that study compares the snubber loads obtained by both analyses. With the exception of two snubbers, all other support loads were higher in the response spectrum analysis, with the average being about 1.45 times larger.

A second example, taken from a study prepared by Badrian<sup>8</sup>, compares steam line stresses computed by response spectrum and non-linear time history methods. Table 3 in that reference provides the results. Here again, the response

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<sup>8</sup> Badrian, "A Seismic Analysis Review," Ebasco Services, Inc. (February, 1977) (Attachment F).

spectrum method is more conservative by approximately a factor of 2.3, although one point did show a higher stress for the non-linear model.

Finally a third study<sup>9</sup> compared results of response spectra analyses performed on 4 in., 16 in. and 28 in. pipelines against those obtained by nonlinear time history analyses of the same systems with varying gaps. Tables 3, 4 and 5 in this study illustrate the difference in support loads and piping stresses that result from several modelling assumptions. Once again, the results of the response spectrum analyses are generally conservative with respect to non-linear analyses.

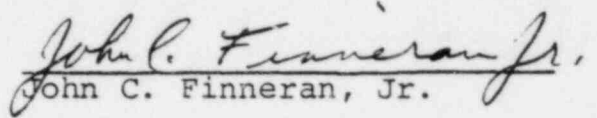
In sum, existing studies and analyses indicate that the effect of gaps are adequately bounded by analyses performed using the response spectrum methodology (such as is employed at Comanche Peak), and further analysis of these effects using nonlinear time history analysis would be unwarranted.

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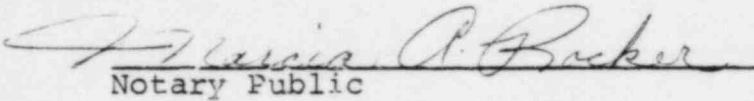
<sup>9</sup> Barta, Huang and Severud, "Seismic Analysis of Piping With Nonlinear Supports," Plant Analysis, FFTF Project, Westinghouse Hanford Company (Attachment G).



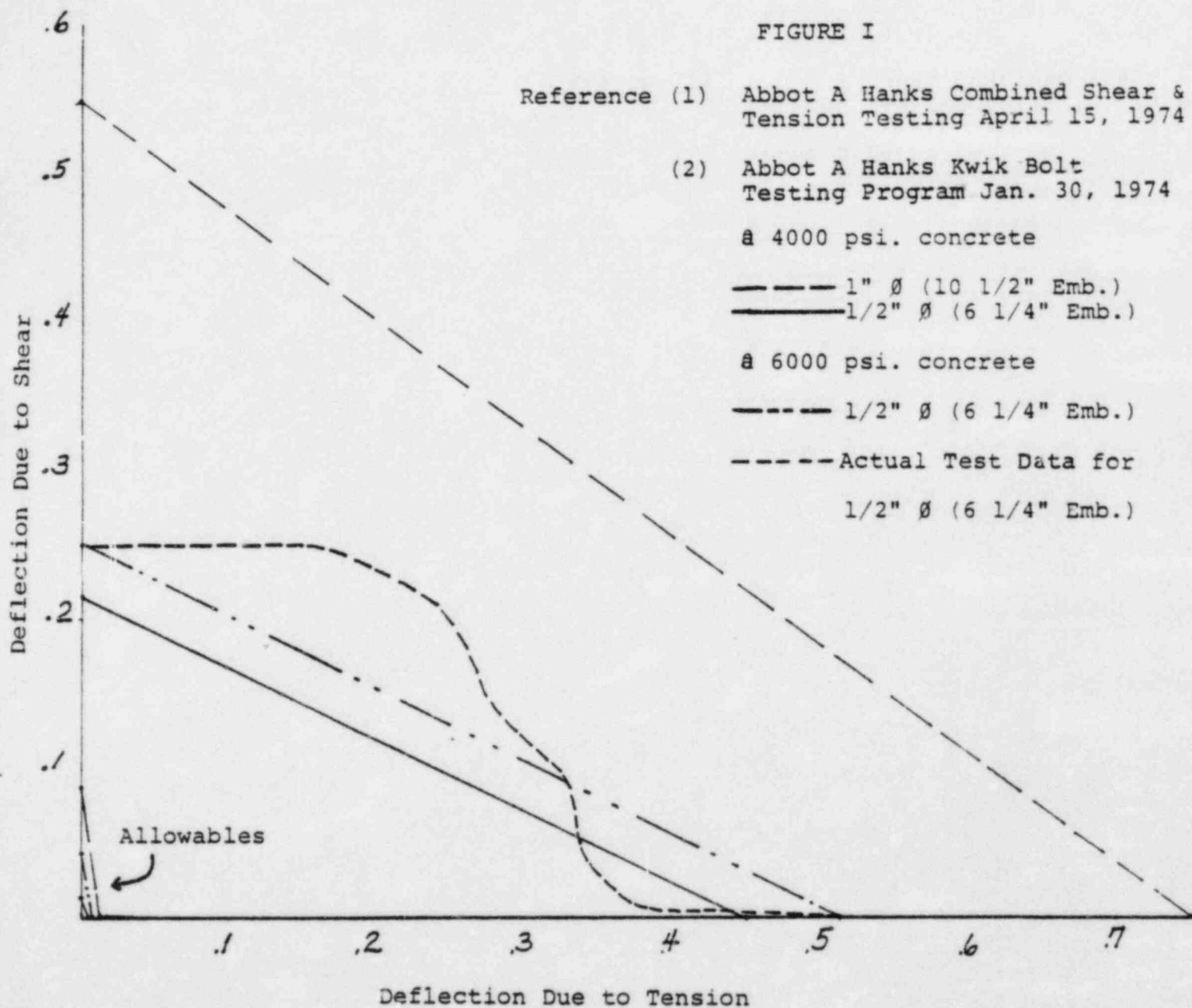
  
Robert C. Iotti

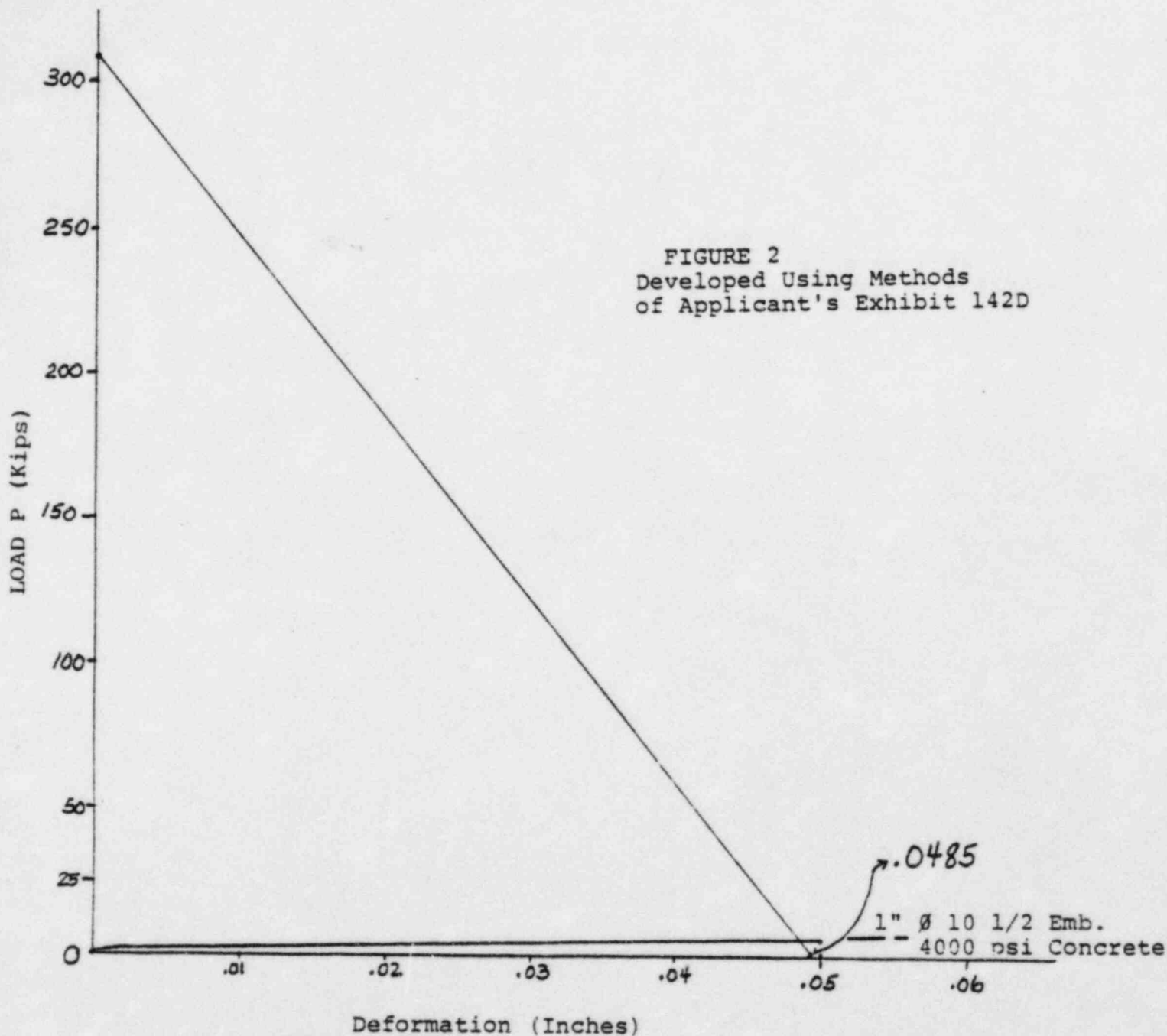
  
John C. Finneran, Jr.

Subscribed and sworn to before me this 18th day of May, 1984.

  
Notary Public

My Commission Expires May 31, 1987





ATTACHMENT A

# · PLASTIC DESIGN of STEEL FRAMES

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THIRD PRINTING, DECEMBER, 1964

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Library of Congress Catalog Card Number: 58-13454

Printed in the United States of America



would require careful study prior to the application of plastic design to them.

#### 1.4 TACIT ACCEPTANCE OF DUCTILE BEHAVIOR

All but the most recent texts and specifications in the field of structural steel design have required that the fiber stresses should nowhere exceed the yield point of the material when a specified overload is applied. As shown in Art. 1.1 this has been an appropriate criterion for simple beams because deformations start increasing rapidly at yield-point stress. But if it were argued that yielding could not be permitted in any part of a structure, then much of the past and present practice would be completely ruled out.

Both in buildings and in bridges, specifications allow the designer to use average stresses due to bending, shear, and bearing that result in actual local yielding. Such cases occur in pins and rivets and at local points. This local yielding results from stress concentrations that are neglected in the simple design formulas. Plastic action is thus depended upon to insure the safety of steel structures, and experience has shown that average or *nominal* maximum stresses form a satisfactory basis for design.

Perhaps the outstanding example of this variance between elastic design assumptions and the actual truth is to be found in the ordinary riveted or bolted joint. The assumption commonly is made that each fastener carries the same shear force. This is true for the case of two fasteners in a line. When more are added (Fig. 1.5), then as long as the joint remains elastic, the outer fasteners must carry the greater proportion of the load. For the example with four rivets, if each rivet transmitted the same load, then between rivets *C* and *D* one plate would carry perhaps three times the force in the other. Therefore it would stretch three times as much and would necessarily force the outer rivet (*D*) to carry more load than was assumed. The actual forces would look something like those shown under the heading "Elastic" in Fig. 1.5. What eventually happens is that the outer rivets yield, redistributing forces to the inner rivets until all forces are about equal as shown. Therefore the basis for design of a riveted joint is really its ultimate load and not the attainment of first yield.

To a greater extent than we may realize, the maximum *strength* of a structure has always been the dominant design criterion. When the usual permissible working stress has led to designs that were consistently too conservative, then that stress has been changed. Present design

procedures disregard local over-stressing at points of stress-concentration, etc. and long experience with similar structures so-designed shows that this is a safe procedure. Thus, the stresses that are calculated for elastic design purposes often are not true maximum stresses at all; they simply provide an index for structural design.

A number of examples will now be given in which the ductility of steel has been counted upon in elastic design—knowingly or not—but certainly not through direct application of plastic design procedures. The following listing is in two categories: (1) factors that are neglected because of the compensating effect of ductility, and (2) instances in which the working stresses have been revised because the "normal" value was too conservative. Following the listing, several examples in each category will be discussed in further detail.

I. Factors that are neglected:

- (1) Residual stresses (in the case of flexure) due to cooling after rolling.
- (2) Residual stresses resulting from the cambering of beams.
- (3) Erection stresses.
- (4) Foundation settlements.
- (5) Over-stress at points of stress-concentration (holes, etc.).
- (6) Bending stresses in angles connected in tension by one leg only.
- (7) Over-stress at points of bearing.
- (8) Nonuniform stress-distribution in splices, leading to design of connections on the assumption of a uniform distribution of stresses among the rivets, bolts, or welds. (Discussed above.)
- (9) Difference in stress-distribution arising from the "cantilever" as compared with the "portal" method of wind stress analysis.

II. Revisions in working stress due to reserve plastic strength:

- (10) Bending stress of 30 ksi in round pins.
- (11) Bearing stress of 40 ksi in pins in double shear.
- (12) Bending stress of 24 ksi in framed structures at points of interior support.

Consider item (1) for example. All rolled members contain residual stresses that are formed due to cooling after rolling or due to cold-straightening. Figure 1.6 shows a typical W-shape (sketch a) with a characteristic residual stress pattern (sketch b). When load-carrying bending stresses are applied (sketch c), the resulting strains are additive to the residual strains already present. As a result, the "final stress" (sketch d) could easily involve yielding at working load. In the example of Fig. 1.6 such yielding has occurred both at the compression flange tips and at the center of the tension flange (sketch e). Thus it is seen



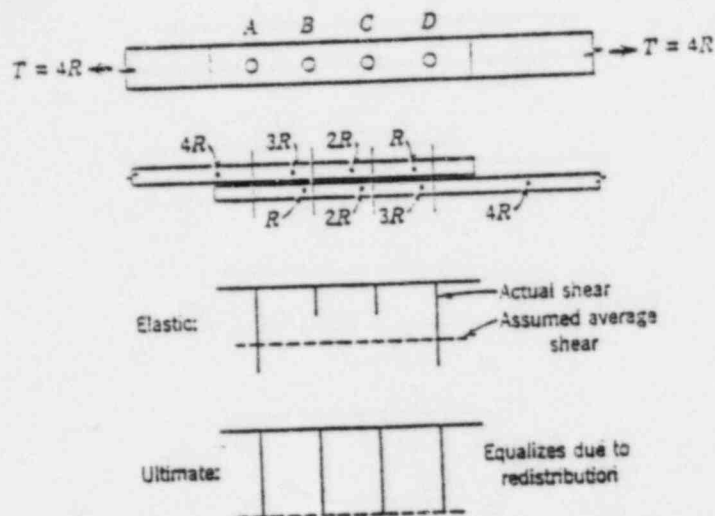


Fig. 1.5. Redistribution of shear in the fasteners of a lap joint.

that cooling residual stresses (whose influence is neglected and yet which are present in all rolled beams) may cause yielding in a beam even below the working load.

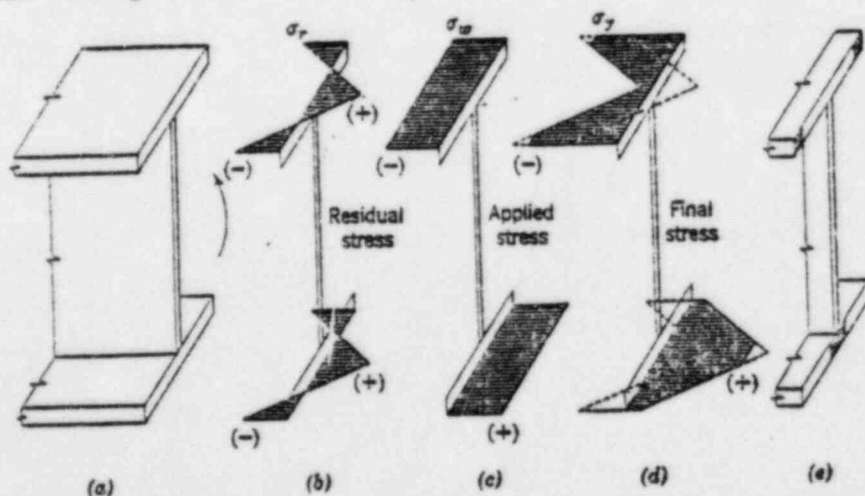


Fig. 1.6. Residual stresses due to cooling after rolling (b). When load-carrying stresses are applied (c), yielding may occur at elastic design working load (d) and (e).

As a justification for neglecting erection stresses (see item 3 on page 8), Fig. 1.7 shows how an erection force due to dimensional inaccuracy may introduce bending moments into a structure prior to the application

# Structural Design Guide to AISC Specifications for Buildings

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NEW YORK CINCINNATI ATLANTA DALLAS SAN FRANCISCO

Van Nostrand Reinhold Company Regional Offices:  
New York Cincinnati Atlanta Dallas San Francisco

Van Nostrand Reinhold Company International Offices:  
London Toronto Melbourne

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Library of Congress Catalog Card Number: 75-40491  
ISBN: 0-442-26904-8

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Manufactured in the United States of America

Published by Van Nostrand Reinhold Company  
450 West 33rd Street, New York, N.Y. 10001

Published simultaneously in Canada by Van Nostrand Reinhold Ltd.

15 14 13 12 11 10 9 8 7 6 5 4 3 2 1

Library of Congress Cataloging in Publication Data

Rice, Paul F. 1921-  
Structural design guide to AISC specifications for  
buildings.

Includes bibliographical references and index.

1. Structural design—Handbooks, manuals, etc.
  2. Building—Contracts and specifications. I. Hoffman, Edward S., 1920- joint author. II. Title.
- TA658.3.R52 690 75-40491  
ISBN 0-442-26904-8

joint as well as the shear, 1.15.5), or *semi-rigid* (transmitting a pre-determined fraction of the full moment capacity as a rigid joint and further loads in shear as a flexible joint with corresponding angle change to supply rotation for the additional loads, 1.15.5; 1.2).

**Flexible Connections.** "Flexible" connections are designed to transmit shear without exceeding allowable unit stresses on the connectors as a group or the connection as a whole. The use of an average capacity for each of several connector elements sharing the total load is justified by allowing self-limiting localized stresses determined by an elastic joint analysis to exceed the yield point and create inelastic localized deformations of the connector materials, or by inelastic deformations of the connection elements (1.15.5). The simplest examples of localized deformation occur in the assembly of bearing-type bolted connections where the cumulative tolerances permitted exist on (1) out-of-round in the bolts, (2) oversize holes ( $\frac{1}{16}$ "), and (3) center-to-center location of the holes in the different elements connected. The extreme degree of such inelastic action occurs with a two-bolt bearing-type connection where one bolt is loosely fitted and one is very tight. Until the material of the connected element surrounding the loaded bolt or the bolt yield and deforms ( $+\frac{1}{16}$ "), the load is not shared and a 50 percent adjustment will be developed as the load increases. For larger (and thus more important) members, more bolts or rivets will be required and the degree of adjustment required on each will be less. Lesser adjustments are required for a long line of bolts or rivets intended to share stress equally. Even if perfectly fitted, yielding and inelastic deformations occur, maximum at and beginning at the first loaded bolt or rivet, and decreasing to a minimum at the last. (See Figs. 5-

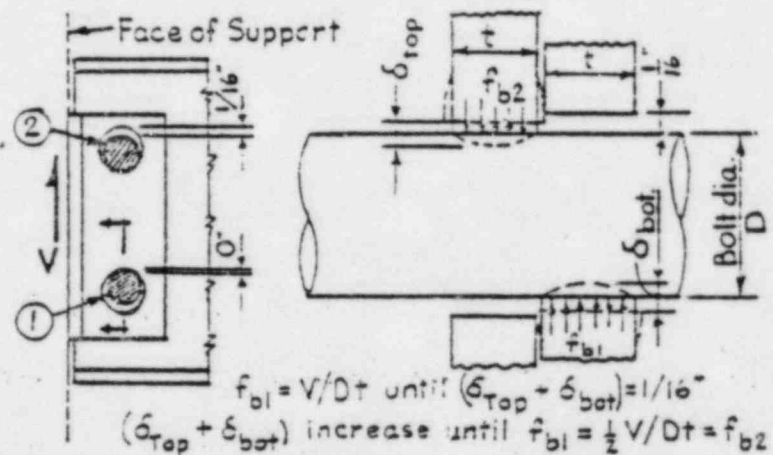


FIG. 5-1 Self-Limiting, Localized Deformations—Two Bolts.

and 5-2.) After this localized inelastic adjustment in the connectors for shear transmission, consider the inelastic adjustments that occur to reduce the "elastic theory" moments.

Inelastic deformation in the connection elements, typically angles, will occur and reduce the restraint which would transmit moment. The common double-angle shear bearing connection is extremely stiff longitudinally for the transmission of shear, and it depends upon the minor inelastic bearing deformations around each fastener to equalize the shear stresses in the fasteners. The same double-angle member is relatively flexible and will twist to permit a relatively large angular rotation reducing moment transmission. (See Fig. 5-3.)

Experience and tests confirm the practical assumptions of shear transfer only and the



TABLE 1.23.4

MAXIMUM SIZES<sup>a</sup> OF FASTENER HOLES, INCHES

Nominal Fastener Diameter (d)	Standard Hole Diameter	Oversized <sup>b</sup> Hole Diameter	Short-Slotted <sup>b</sup> Hole Dimensions	Long-Slotted <sup>b</sup> Hole Dimensions
$\leq \frac{7}{8}$	$d + \frac{1}{16}$	$d + \frac{1}{8}$	$(d + \frac{1}{16}) \times (d + \frac{1}{4})$	$(d + \frac{1}{16}) \times 2\frac{1}{2}d$
1	$1\frac{1}{16}$	$1\frac{1}{4}$	$1\frac{1}{8} \times 1\frac{1}{16}$	$1\frac{1}{8} \times 2\frac{1}{2}$
$\geq 1\frac{1}{8}$	$d + \frac{1}{16}$	$d + \frac{1}{8}$	$(d + \frac{1}{16}) \times (d + \frac{1}{4})$	$(d + \frac{1}{16}) \times 2\frac{1}{2}d$

<sup>a</sup> Sizes are nominal.  
<sup>b</sup> Not permitted for riveted connections.

gouges greater than  $\frac{1}{16}$ -inch that remain from cutting shall be removed by grinding. All re-entrant corners shall be shaped notch-free to a radius of at least  $\frac{1}{2}$ -inch.

### 1.23.3 Planing of Edges

Planing or finishing of sheared or thermally cut edges of plates or shapes will not be required unless specifically called for on the drawings or included in a stipulated edge preparation for welding.

### 1.23.4 Riveted and Bolted Construction—Holes

1.23.4.1 The maximum sizes of holes for rivets and bolts shall be as stipulated in Table 1.23.4, except that larger holes, required for tolerance on location of anchor bolts in concrete foundations, may be used in column base details.

1.23.4.2 Standard holes shall be provided in member-to-member connections, unless oversized, short-slotted, or long-slotted holes in bolted connections are approved by the designer. Oversized and slotted holes shall not be used in riveted connections.

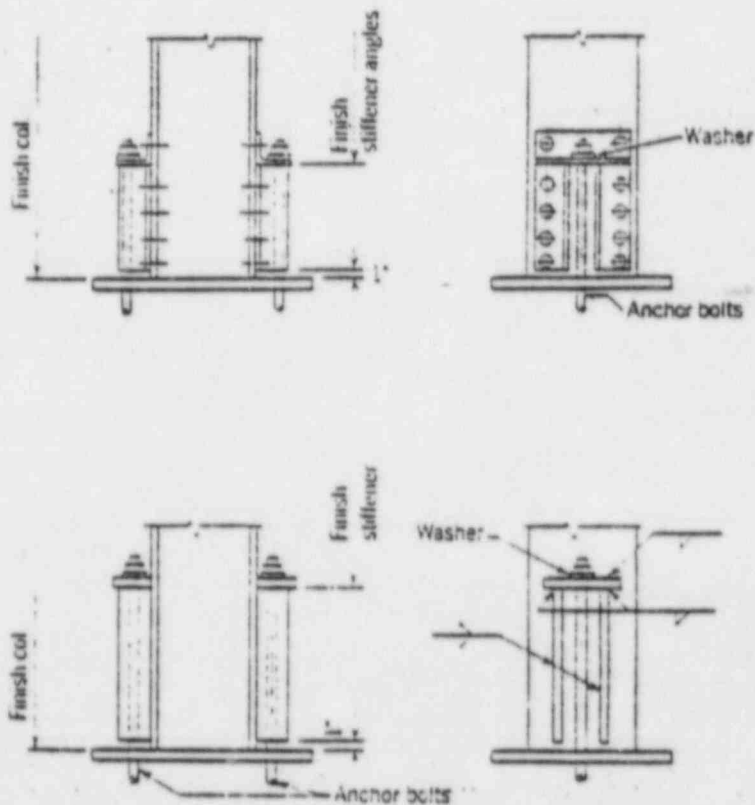
If the thickness of the material is not greater than the nominal diameter of the rivet or bolt plus  $\frac{1}{8}$ -inch, the holes may be punched. If the thickness of the material is greater than the nominal diameter of the rivet or bolt plus  $\frac{1}{8}$ -inch, the holes shall be either drilled from the solid, or sub-punched and reamed. The die for all sub-punched holes, and the drill for all sub-drilled holes, shall be at least  $\frac{1}{16}$ -inch smaller than the nominal diameter of the rivet or bolt. Holes in A514 steel plates over  $\frac{1}{2}$ -inch thick shall be drilled.

1.23.4.3 Oversized holes may be used in any or all plies of friction-type connections, but they shall not be used in bearing-type connections. Hardened washers shall be installed over oversized holes in an outer ply.

1.23.4.4 Short-slotted holes may be used in any or all plies of friction-type or bearing-type connections. The slots may be used without regard to direction of loading in friction-type connections, but the length shall be normal to the direction of the load in bearing-type connections. Washers shall be installed over short-slotted holes in an outer ply; when high-strength bolts are used, such washers shall be hardened.

# SUGGESTED DETAILS

## Column base plates



Base plates are normally detailed and shipped loose.

- Notes:
1. Hole sizes for anchor bolts are normally made oversize to facilitate erection as follows:  
 Bolts  $\frac{1}{2}$  to 1" —  $\frac{3}{16}$ " oversize  
 Bolts 1 to 2" —  $\frac{1}{4}$ " oversize  
 Bolts over 2" — 1" oversize
  2. The stability of a column with its loading should be considered at all stages of erection and its base designed accordingly for anchors and base plate.

AMERICAN INSTITUTE OF STEEL CONSTRUCTION

MASTER

HEDL-SA-1769

ANALYSIS OF PIPING SYSTEMS WITH NONLINEAR SUPPORTS  
SUBJECTED TO SEISMIC LOADING

D. A. Barta

March 1979

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June 1979 - San Francisco, Ca.

HANFORD ENGINEERING DEVELOPMENT LABORATORY  
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dynamic degrees of freedom. These interfaces are described by specific forms of forcing functions for the linear analysis. Since the dynamic matrix is inverted but once, option 2.5 is far more economical than the former, and it was used throughout this investigation.

## ANALYSIS RESULTS AND DISCUSSION

### Single Degree of Freedom Results

The response of the simplified spring/mass model shown in Figure 1 to the FFTF horizontal acceleration was determined with damping assumed to be 2% of critical. Gap sizes were varied from zero to 1.0 inch (2.54 cm) and the oscillator frequency, calculated by disregarding gap effects, was varied from 1.0 to 30. Hz. The results, which are shown in Figure 7, reveal relatively small load magnification for all gap sizes at frequencies lower than the 2.5 Hz. seismic spectrum peak. At frequencies higher than 5. Hz. load magnifications due to impact against hard structure become increasingly larger with increases in both natural frequency and gap size.

The response results shown in Figure 8 illustrate the effect of 20% of critical impact damping on response attenuation. The nonlinear responses are less than the 2% of critical damped linear response spectrum at all frequencies lower than 5. Hz. Furthermore, the nonlinear responses remain lower than the peak of the response spectrum up to a frequency of 30. Hz. The value of 20% critical damping for impact was chosen arbitrarily for this study. However, some fuel assembly impact tests [3] show maximum rebounds of less than 30%. The impact damping coefficient was shown in [3] to be related to the coefficient of restitution by the following equation:

$$C_R = e^{(-2\pi\delta/\sqrt{1-\delta^2})}$$

Where,  $\delta$  = Coefficient of Impact Damping

$C_R$  = Coefficient of Restitution

Maximum rebounds of less than 30% approximate a 20% of critical impact damping coefficient.

### Simulated Mechanical Snubber/Piping Results

A simulation of the combined civil structure, snubber and piping stiffness, shown in Figure 2, was used to assess the seismic response characteristics over a range of snubber stiffness and damping values which were determined by snubber characterization tests. The assumed piping mass and stiffness correspond to a 1.0 inch (2.54 cm) nominal diameter pipe with insulation and filled with sodium, and with a 141.6 inch (360. cm) span simply supported at each end to ground. A mechanical snubber with variable stiffness and damping values was assumed to be attached at one end to the piping at mid-span and at the other end to ground through civil support structure with variable stiffness. The snubber gap was assumed to be .030 inch (.076 cm) as measured by test. A range of snubber stiffness and damping values were used which correspond to a small snubber of the type that are used to support small piping. The seismic loading was applied through large base masses representing ground. Forces applied through the ground masses were scaled to produce base accelerations identical to the FFTF horizontal seismic motion. The civil support structure stiffness values were chosen to tune the system natural vibration frequencies to 2.5, 5.0 and 9.0 Hz.

TABLE 2  
COMPARISON OF LINEAR SPECTRA AND NONLINEAR ANALYSIS  
FFTF PIPING SYSTEM SNUBBER LOADS

SEISMIC SUPPORT NO.	SNUBBER LOADS ~ LBF*		Ratio
	LINEAR SPECTRA	NONLINEAR ANALYSIS	Linear Nonlinear
1 - X	5047	4140	1.2
1 - Y	3289	1423	2.3
2 - Z	5071	3152	1.6
3 - X	2552	1315	1.93
3 - Y	2391	1776	1.35
4 - X	4422	4376	1.01
4 - Y	1327	486	2.73
5 - Y	1819	2876	0.63
5 - Z	5865	5514	1.06
5A - Y	2092	2708	1.07
5A - Z	5026	5273	1.10
6 - Z	10817	4716	2.29
7 - X	9369	8536	1.10
7 - Y	4862	3760	1.29
7A - X	9835	8994	1.09
7A - Y	6086	4021	1.51
9 - X	3670	2238	1.64
9 - Z	4227	3516	1.20
10 - Z	6888	3418	2.01

\*1.0 LBF = 4.448 N.

ATTACHMENT F

APTR 12

3-E-2

11.06.23

A SEISMIC ANALYSIS

REVIEW

MORRIS BADRIAN

FEBRUARY • 1977

EBASCO SERVICES INCORPORATED

NEW YORK

TABLE 3

MAIN STEAM LINE (2MS-32-2SB) OBE STRESSES

Node Number (EQ)	Number (PS)	Uniform Response*		Time History Method		Ratio of Spectrum To History Methods
		Spectrum Method (PSI)		(PSI)	(SEC.)	
8	712	4,686		2,124	2.88	2.2
34	2	2,084		1,508	2.97	1.4
		1,153		835	2.97	1.4
35	4	1,234		683	2.95	1.8
		2,229		1,225	2.95	1.8
36	5	1,601		503	2.83	3.2
		886		287	2.83	3.1
37	6	4,609		1,976	2.95	2.3
				1,985	2.95	
38	8	1,461		1,744	3.12	.84
				1,744	3.12	
39	850	1,388		691	3.62	2.0
				692	3.62	
40	9	1,635		421	6.01	3.9
		2,956		747	6.01	3.9
41	10	3,291		1,075	5.78	3.1
		1,821		607	5.78	3.0
42	12	3,324		2,403	9.85	1.4

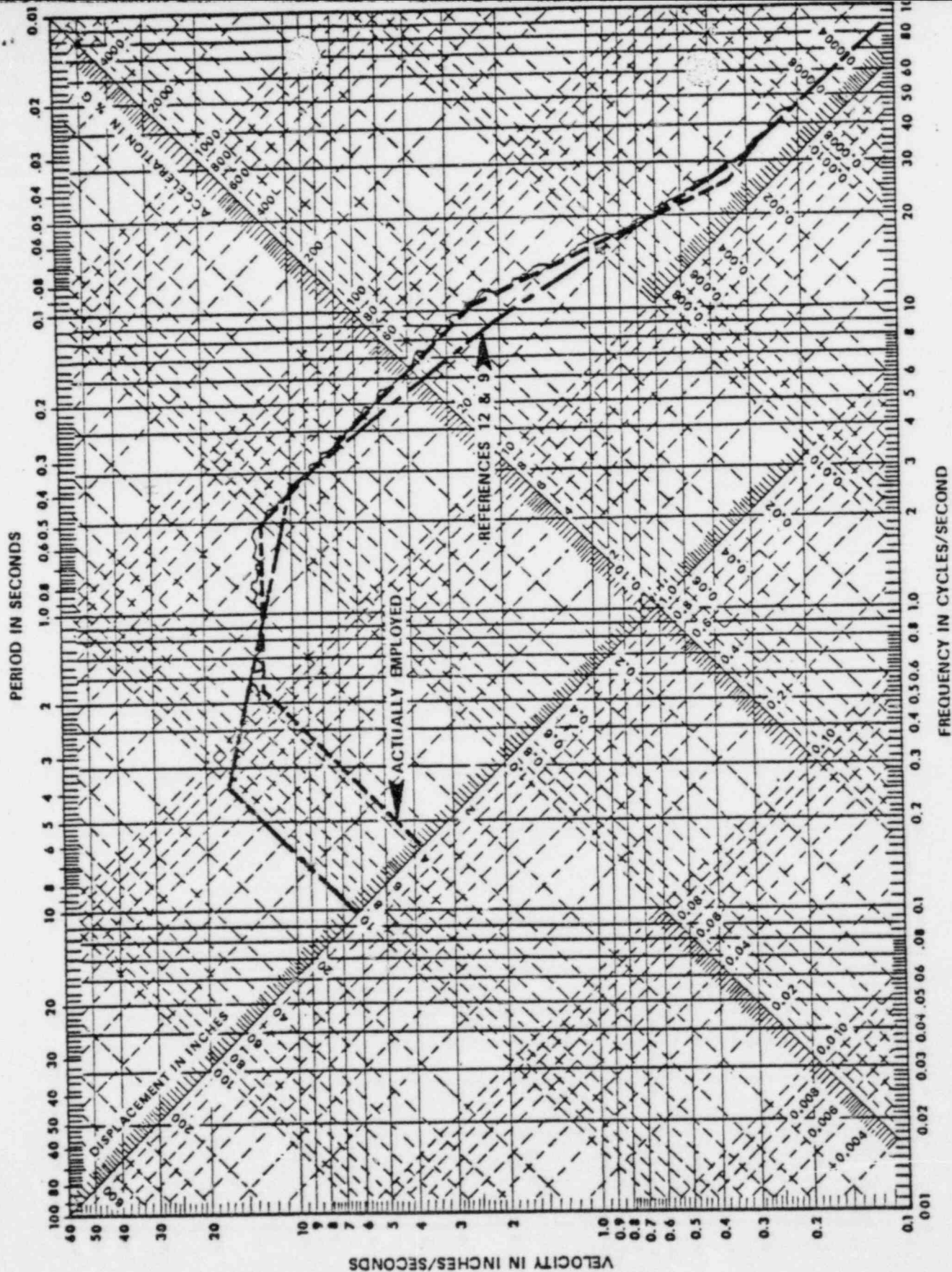
\* Does not include relative anchor displacement effects

Sources of Data

1. Uniform Response Spectrum Method - Pipestress 2010

Calculation Number 1157 (July 6, 1976)

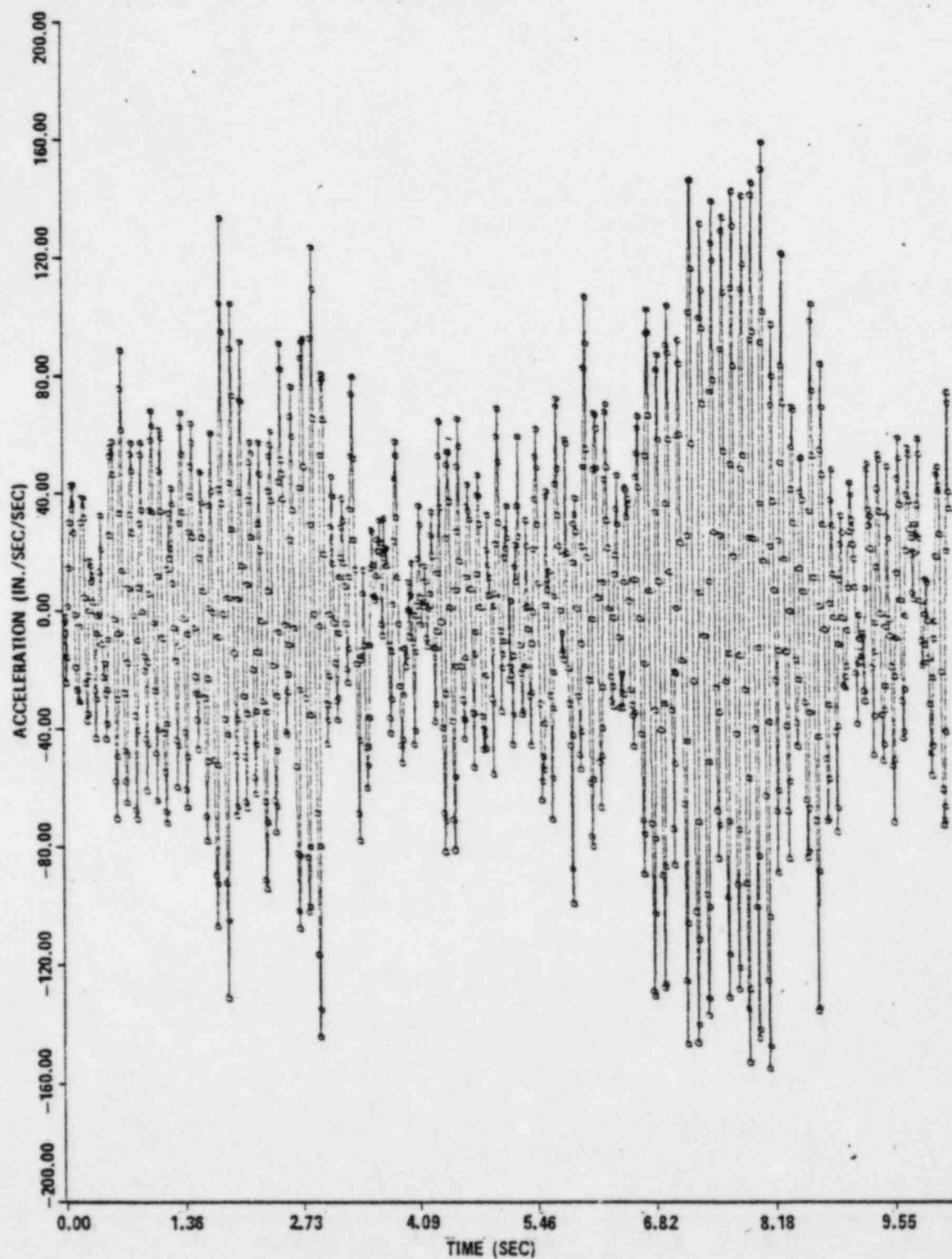
2. Time-History Method - PLAST run "LOOP23D" (December 27, 1976)



COMPARISON OF DBE  
HORIZONTAL BEDROCK SPECTRA

FIGURE 3

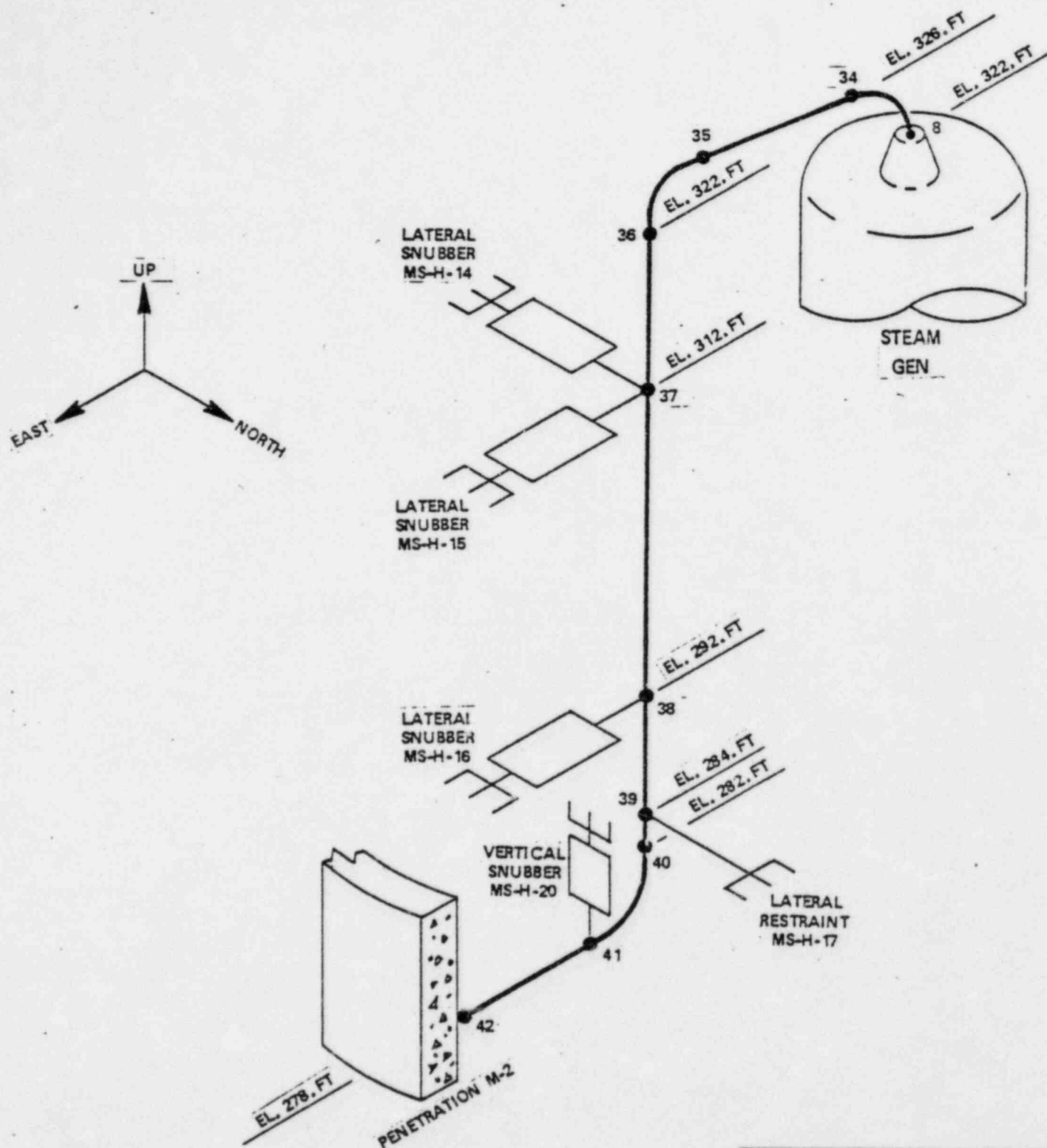




CONTAINMENT BUILDING  
(INTERNAL STRUCTURE - ELEVATION 302.0 FT)  
10/76 NORTH-SOUTH FLOOR ACCELERATION HISTORY

FIGURE 8





REFERENCE: IC-FW-MS-RC-2 (3-1-76)  
LOOP 23D NODE NUMBERS

EQUIPMENT MODEL  
(MAIN STEAM LINE PORTION)

FIGURE 9

## SEISMIC ANALYSIS OF PIPING WITH NONLINEAR SUPPORTS

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Westinghouse Hanford Company  
Richland, Washington

## ABSTRACT

The modeling and results of nonlinear time-history seismic analyses for three sizes of pipelines restrained by mechanical snubbers are presented. Numerous parametric analyses were conducted to obtain sensitivity information which identifies relative importance of the model and analysis ingredients. Special considerations for modeling the pipe clamps and the mechanical snubbers based on experimental characterization data are discussed. Comparisons are also given of seismic responses, loads and pipe stresses predicted by standard response spectra methods and the nonlinear time-history methods.

## INTRODUCTION

In the high temperature and seismic environment of a nuclear power plant, such as the Fast Flux Test Facility (FFTF), mechanical type snubbing devices are used to restrain piping motions during seismic events. The snubber is connected at one end to the building support structure and at the other end to piping through a clamp assembly. During normal plant operating conditions, these snubbers offer no significant resistance to thermal growth of the piping. Under seismic conditions the snubbers lock up at very low acceleration levels. Dynamic characterization tests were performed at HEDL on mechanical snubbers to determine their stiffness, dynamic loss of motion due to free play in the support linkages, and conditions of lockup. The tests also disclosed that the snubbers dissipate a large amount of energy per cycle which could provide a significant amount of piping system damping that may exceed that commonly used for seismic response spectra analyses of nuclear piping.

As described in U. S. Nuclear Regulatory Commission Guides, such as in [1], [2], two methods can be used in the seismic analysis, i.e., the response spectra or the time-history method. Due to theoretical limitations and computer implementation difficulties, the response spectra method can not be used to evaluate some of the essential support characteristics such as free play and local damping at snubber locations. Thus, the response spectra method is limited to linear analysis. On the other hand, the time-history method is able to include support nonlinearities and local damping in the analysis but with

Numbers in parentheses indicate references listed at the end of the paper.

TABLE 3. 4 INCH PIPELINE

HANGER NO.	HANGER LOAD ~ LB. (1 LBF = 4.448 N)												
	RUN 1	RUN 2	RUN 3	RUN 4	RUN 5	RUN 6	RUN 7	RUN 8	RUN 9	RUN 10	RUN 11	RUN 12	RUN 13
H - 9Y	94	134	107	39	39	39	38	39	39	39	38	39	28
H - 8X	62	168	105	0	16	0	0	0	0	0	0	72	109
H - 7Y	77	192	144	51	51	51	50	51	51	50	51	53	44
H - 6X	529	946	431	70	107	89	151	70	70	65	79	480	518
H - 5Y	105	125	91	33	33	33	33	26	39	30	33	34	43
H - 5Z	439	448	200	0	42	23	33	0	0	0	0	198	199
H - 4U	67	152	113	23	22	22	22	19	25	22	22	25	35
H - 4Z	191	269	156	0	13	0	0	0	0	0	0	128	214
H - 3Y	73	175	115	31	31	31	31	25	36	29	31	31	36
H - 3Z	52	176	99	0	22	9	13	0	0	0	0	97	94
H - 2Y	36	270	211	48	48	48	48	38	55	48	47	55	43
H - 2Z	79	183	142	0	13	0	0	0	0	0	0	84	113
H - 1X	142	359	180	20	60	40	65	15	23	14	26	210	241
H - 1Y	54	131	82	22	22	22	22	18	25	22	21	21	45
LOCATION	MAXIMUM PIPING STRESS ~ PSI (1 KSI = 6.89 MPa)												
NODE 1		12.1										3.3	

- NOTES:
1. PIPESD analysis with rigid supports and using enveloping design seismic spectra of Figures 10 and 11.
  2. PIPESD analysis with support and clamp flexibility and using enveloping seismic spectra.
  3. ANSYS model with support and clamp flexibility same as in RUN 2 but using calculated seismic spectra.
  4. Nonlinear, time domain analysis with support and clamp flexibility same as in RUN 3 and combined with snubber stiffness and damping characteristics.  
Snubber gaps = .030 in. (.076 cm).
  5. Same as RUN 4 but with snubber gaps = .005 in. (.013 cm).
  6. Same as RUN 4 but with snubber gaps = .015 in. (.038 cm).
  7. Same as RUN 4 but with snubber damping reduced 50%.
  8. Same as RUN 4 but with clamp stiffness decreased 30%.
  9. Same as RUN 4 but with clamp stiffness increased 30%.
  10. Same as RUN 4 but with time history compressed 10%.
  11. Same as RUN 4 but with time history expanded 10%.
  12. Same as RUN 4 but with snubber damping scaled in proportion to the energy across the snubber.
  13. Structural parameters same as in RUN 3 but with spatial components of seismic motion applied separately and spatial responses combined according to Regulatory Guide 1.92, Article 2.2.

TABLE 4. 15 INCH PIPELINE

SEISMIC SUPPORT NO.	SEISMIC SUPPORT LOAD ~ LB. (1 LBF = 4.448 N)						
	LINEAR ANALYSIS	NONLINEAR ANALYSIS					
		RUN 1	RUN 2	RUN 3	RUN 4	RUN 5	RUN 6
H - 1 X	6283	91	95	87	99	133	1390
H - 1 Y	3347	104	154	113	153	135	726
H - 2 X	5074	949	1200	965	1207	1224	1224
H - 3 X	10786	569	634	557	629	684	907
H - 3 Y	2645	328	403	317	334	395	812
H - 4 X	29512	3165	3170	3044	2826	2882	3579
H - 4 Y	25456	352	324	408	364	394	580
H - 5 X	2883	0	0	0	0	0	942
H - 5 Y	12566	2189	2178	2262	2172	2190	2262
H - 5A X	6926	0	0	12	6	15	882
H - 5A Y	9357	3237	3262	3236	2989	2999	3262
H - 6 X	36525	3896	3996	3719	4157	4150	4160
H - 6 Y	77212	1526	1613	1408	1707	1672	4378
H - 7 X	10441	1983	2219	2189	2241	2232	3818
H - 7A X	85026	3275	3304	3158	3212	3209	5048
H - 7A Y	14240	2240	2187	2153	2203	2209	4472
H - 10Z	31668	6272	5945	6204	6168	6167	6281
PIPING STRESS ~ PSI (1 PSI = 6.89 KPa)							
ELBOW NO.							
1	16185	3267	5738	3043	5767	5718	6432
6	15844	3567	3902	2573	3739	3714	4098
8	36765	13526	15353	14076	15462	15451	15666
9	56989	22031	22644	21310	22131	22182	23076
11	63906	7831	8196	7779	7965	8007	9189
13	57404	35037	35737	34223	35357	35372	36058

NOTES: RUN 1 - PIPESD analysis with support flexibility, using 2% damped DBE seismic spectra and 10% method modal summation.

RUN 2 - Nonlinear, time history analysis with snubber stiffness and damping characteristics, snubber gap = .030 in (.076 cm). Average clamp stiffness. Normal time history.

RUN 3 - Same as RUN 2 but with time history compressed 10%.

RUN 4 - Same as RUN 2 but with time history expanded 10%.

RUN 5 - Same as RUN 3 but with minimum clamp stiffness.

RUN 6 - Same as RUN 3 but with maximum clamp stiffness.

RUN 7 - Maximum responses from RUNS 2-6 due to z-axis loading combined with responses due to x-axis and y-axis loadings according to Regulatory Guide 1.92, Article 2.2.

TABLE 5. 28 INCH PIPELINE

HANGER LOAD ~ LBS. (1 LBF = 4.448 N)												
HANGER NO.	LINEAR ANALYSIS		NONLINEAR ANALYSIS									
	RUN 1	RUN 2	RUN 3	RUN 4	RUN 5	RUN 6	RUN 7	RUN 8	RUN 9	RUN 10	RUN 11	RUN 12
H - 2 X	33944	16041	5826	5854	8259	5816	5831	5502	6108	17387	11584	10266
H - 2 Y	9406	9749	4593	4667	6119	4583	4602	4376	4889	15752	8336	7042
H - 2AX	24016	13271	5040	5001	5927	5079	5021	5039	5133	14563	10773	11074
H - 2AY	8599	13247	3439	3496	7486	3438	3440	3368	3502	10419	6776	6594
H - 3 X	11100	30095	5475	5647	13221	5475	5476	5217	5666	32725	23603	24862
H - 5 X	4949	8976	2657	2688	4491	2665	2646	2453	2801	8602	5945	5676
H - 6 X	7919	27628	5534	5645	11743	5538	5530	5244	5783	29380	22729	22216
H - 7 X	5596	17521	2371	2376	4772	2266	2419	2152	2465	16055	5962	8870
H - 7 Y	7588	7496	1739	1770	3752	1667	1828	1584	1821	5849	4460	4281
H - 9 X	4654	20374	1722	1846	2860	1719	1718	1604	1811	15917	5815	7924
H - 9 Y	6501	26646	1523	1674	7804	1525	1520	1402	1697	18368	9033	8492
H - 11X	4876	14215	2891	2993	6318	2890	2892	2669	3034	14155	11180	8866
H - 12Z	5580	20670	3585	3671	6973	3579	3607	3416	3754	15809	12332	11534

PIPING ELBOW STRESS ~ PSI (1 PSI = 6.89 KPa)												
ELBOW NO.												
1	7892	20598	7513	7500	—	7510	7515	7748	7451	28320	15163	16712
5	16214	17521	8617	8564	—	8630	8608	8174	8774	21053	14720	10631
6	10745	16779	6376	6318	—	8362	6380	5892	6565	29304	16305	15338
7	8745	26512	5328	5315	—	5335	5320	5698	5162	25104	13388	10861
10	13068	28703	6865	7038	—	6866	6861	7642	7442	24536	15974	12403
12	8854	19308	5037	4982	—	5033	5037	4895	5214	16959	12354	10648

- NOTES: RUN 1 - PIPE2D linear analysis with reactor vessel ledge flexibility but with rigid pipe supports.  
 RUN 2 - PIPE2D linear analysis with reactor vessel ledge flexibility and with flexible pipe supports.  
 RUN 3 - ANSYS nonlinear analysis with reactor vessel ledge flexibility, average clamp stiffness, snubber test data of stiffness and damping characteristics, snubber gaps = 0.030 in (0.076 cm).  
 RUN 4 - Same as RUN 3 but with snubber gaps = 0.015 in (0.038 cm).  
 RUN 5 - Same as RUN 3 but with RUN 2 support stiffnesses and snubber dampings = 200 lb-sec/in (350 N-sec/cm).  
 RUN 6 - Same as RUN 3 but with minimum biaxial clamp stiffnesses at H-7 and H-9.  
 RUN 7 - Same as RUN 3 but with maximum biaxial clamp stiffnesses at H-7 and H-9.  
 RUN 8 - Same as RUN 3 but with time history compressed 10%.  
 RUN 9 - Same as RUN 3 but with time history expanded 10%.  
 RUN 10 - Same as RUN 3 but with zero damping and 1.8 times the horizontal acceleration/time history.  
 RUN 11 - Same as RUN 10 but with snubber damping scaled down in proportion to the energy across the snubber.  
 RUN 12 - Same as RUN 11 but with spatial components of seismic motion applied separately and spatial responses combined according to Regulatory Guide 1.92 Article 2.2.