Docket No. 50-346

License No. NPF-3

Serial No. 1219

December 17, 1985

TOLEDO EDISON

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Mr. John F. Stolz, Director
PWR Project Directorate No. 6
Division of PWR Licensing - B
United States Nuclear Regulatory Commission
Washington, D.C. 20555

Dear Mr. Stolz:

On March 12, 1985, the NRC issued an Information Request pursuant to 10 CFR 50.54(f) (Log No. 1716). This letter requested Toledo Edison (TED) to identify the actions and schedule planned to implement the NRC Structural and Geological Engineering Branch (SGEB) Staff position for 75 masonry walls. These walls had been qualified by TED under IE Bulletin 80-11 (Log No. 1-362) utilizing the energy balance technique at the Davis-Besse Nuclear Power Station Unit No. 1. The Staff position does not accept the qualification of masonry walls by the energy balance technique.

On April 25, 1985, TED representatives met with the NRC Staff and presented our proposed approach to resolve this issue. The NRC Staff requested that we provide a formal submittal of our method of qualifying these walls by linear elastic working stress analysis, and address four specific issues raised in the meeting.

In our September 23, 1985 submittal (Serial No. 1183) we responded to each of the four issues and provided our report titled "Masonry Wall Re-Evaluation Report to IE Bulletin 80-11, Davis-Besse Nuclear Power Station Unit No. 1". We also indicated that all of the walls except one (Wall 5367) had been demonstrated acceptable by the working stress approach. We proposed to modify Wall 5367 and utilize our Integrated Living Schedule Program process to schedule the modification.

Further analysis of Wall 5367 has been performed utilizing a two-way finite element model and the actual reinforcing steel strengths from material test reports. This further analysis demonstrated that Wall 5367 is acceptable by the working stress approach. Accordingly, we no longer plan to modify Wall 5367.

Enclosed are five (5) copies of Revision 1 of our report titled, "Masonry Wall Re-Evaluation, Report to IE Bulletin 80-11, Davis-Besse Nuclear Power Station Unit 1". This revision reflects the further analysis performed for Wall 5367.

A001 w/check $150.00 #461545 1/1

Add: AD - D. CRUTCHFIELD (ltr only)
EB (W. JOHNSTON)
RSB (THOMAS)
EICSB (PARR)
FOB (W. REGAN)

In addition, pursuant to the NRCs request of November 13, 1985 (Log No. 1860) and 10 CFR 170.12(c), enclosed is a check for the amount of $150.00 for the application fee.

Very truly yours

Joe Williams, Jr.

JW:DRW:CLM

cc: DB-1 NRC Resident Inspector

Docket No. 50-346
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December 17, 1985

Masonry Wall Re-Evaluation
Response to NRC IE Bulletin 80-11
Davis-Besse Nuclear Power Station Unit No. 1

Revision 1

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1.0 INTRODUCTION

In response (Reference 1) to IE Bulletin 80-11 for the Davis-Besse Nuclear Power Station, Unit No. 1, the capacity of 169 concrete masonry unit (CMU) walls were re-evaluated. Ninety-five of these walls were qualified by the working stress-elastic analysis technique. The remaining 74 walls were qualified inelastically by the energy balance technique.

The NRC staff in their Safety Evaluation Report (Reference 2) found the walls re-evaluated by the working stress method acceptable. However, the staff indicated the use of the energy balance technique is unacceptable without further confirmation of the methodology. Three approaches were suggested in the Safety Evaluation Report that could be used to re-evaluate the affected 74 CMU walls. In summary, these approaches are as follows:

1. Supplement the energy balance technique with a comprehensive test program.
2. Re-analyze the walls by linear elastic-working stress methods and repair the walls as needed.
3. Use a rigorous non-linear analysis technique, supplemented with confirmatory testing.

The second of these available alternatives, use of linear elastic methodology, is being adopted for this study.

The criteria established by Toledo Edison for the re-evaluation in response to IE Bulletin 80-11 required that all CMU walls be initially evaluated using elastic methods. The re-evaluation criteria concentrated on a simplified approach which (1) could be utilized for all CMU walls, (2) could be applied by different engineers with uniformity of results assured and (3) minimize the need for separate decision making for individual walls. Such an approach was necessary to expedite the total re-evaluation effort within the time limits specified by the bulletin, and simultaneously maintain necessary control and uniformity in the results. This approach was utilized to guarantee that each step of the analysis could be easily demonstrated to yield unquestionable conservative results. The methodology and criteria to accomplish these objectives had several substantial conservatisms beyond those normally imposed on CMU wall design, or necessary to meet minimum licensing requirements.

Those CMU walls that did not satisfy the elastic criteria were then re-evaluated using the energy balance technique. The energy balance technique has been successfully used in seismic design applications for many structures other than nuclear power plants. Since the energy balance technique utilizes the results from the elastic analysis, similar conservatisms exist for both techniques. On the basis that the energy balance technique was a recognized and acceptable evaluation method, the philosophy was to adopt a conservative elastic criteria recognizing such

an approach may artificially indicate some of the walls exceed allowable stresses. However, those walls which may have high reported stresses could be shown to have adequate reserve strength by the energy balance technique and therefore be considered acceptable.

On the basis that the energy balance technique is not acceptable to the NRC staff without further documentation, the original elastic working stress analysis was reviewed in comparison with current acceptable licensing positions. Generic conservatisms which exist in the analysis were identified and in some instances quantified, so that a more accurate estimate of actual wall stress could be determined and compared with the original evaluation. This approach is consistent with the intent of the second alternative included in the Safety Evaluation Report. Using this approach, the 74 CMU walls originally analyzed using the energy balance technique were re-evaluated using linear elastic-working stress methods. The conclusion is that all of the 74 walls are within allowable stresses in accordance with the acceptable licensing criteria. 1

2.0 BACKGROUND

2.1 Identification of Walls

A summary of the walls qualified by the energy balance technique is provided in Tables 1, 2 & 3. Included in the tables is the ratio of the maximum calculated reinforcing steel stress (f_s) obtained from the elastic analysis to the allowable stress (F_{all}). The wall capacities are controlled by the reinforcing steel stress in all cases. This assures

proper ductile action of the walls. The ductility ratio as determined by the energy balance procedure is also provided in Tables 1, 2 and 3.

All but one of the 74 walls are located in areas 6, 7 or 8 of the auxiliary building on the floors between Elevations 545 and 643 with approximately 68% of the walls in area 7. The remaining wall is located in the intake structure at Elevation 567. Plant grade is Elevation 585.

In the NRC Staff evaluation (Reference 2) of the Toledo Edison response to IE Bulletin 80-11, a total of 75 rather than 74 CMU walls are identified as being evaluated by the energy balance technique. A comparison of walls indicates the additional wall is Wall No. 2297. This is a composite wall comprised of two eight inch masonry wythes and a two inch space between the units which is filled with concrete. The wall was originally analyzed as two independent eight inch thick walls spanning vertically which produced a maximum ductility ratio of 2.07 and an overstressed top connection. Prior to final submittal of results to the NRC, a finite element analysis was performed using a composite thickness in order to obtain realistic reactions for the design of the modifications to the top connection. The finite element analysis produced a maximum tensile stress in the vertical reinforcing steel of 13.04 ksi and a maximum tensile stress in the horizontal reinforcing steel of 4.3 ksi. All masonry stresses also passed the working stress criteria.

2.2 Construction Details

Substantial QA records accumulated during the construction phase verify that the materials supplied were furnished in accordance with the applicable specifications. These records also show that regularly scheduled inspections of the walls while under construction were also performed. The inspections verified that the correct materials were being employed, that both vertical and horizontal reinforcing was properly placed and that the cells were correctly filled with concrete. Tests were performed in accordance with applicable ASTM standards to verify minimum material strengths and the actual test values are discussed further in Section 3.1.5.

The construction details for the subject CMU walls are summarized in Table 4 and shown in Figures 8 through 25 of Reference 1. Vertical reinforcing for walls constructed of eight or twelve inch block consist of two number five reinforcing bars at sixteen inch centers. These bars are located on opposite faces of a grouted cell, as shown in the referenced figures, to produce a doubly reinforced masonry section. For double wythe shield walls thicker than one foot-six inches, vertical reinforcing is increased from the two number five bars at sixteen inch centers per wythe. Depending on the wall thickness, reinforcing is as great as two number eight bars at sixteen inches in each wythe. Specific reinforcement requirements are included in Figure 24 of Reference 1. Horizontal joint reinforcement for eight and twelve inch block construction consists of, as a minimum, extra heavy truss wire

reinforcing at every course. Four inch thick walls are centrally reinforced with a number three reinforcing bar in every vertical joint.

All reinforcing is anchored at the CMU wall boundaries. At concrete slab interfaces, vertical bars are lapped (approximately 24 inches) with matching size all thread bars anchored with self-drilling expansion sleeves as shown in Figure 1 (taken from Figure 14, Reference 1). At steel beams, vertical bars are lapped with twenty-four inch long matching size all thread bars secured by sleeve nuts welded to the beam as shown in Figure 2 (Figure 15 of Reference 1).

Horizontal reinforcing is lapped with number three all thread bars at interfaces with concrete walls or columns as shown in Figure 3 (Figure 20 of Reference 1). The number three bars are secured by self-drilling expansion sleeves at concrete boundaries. Special details employing "Z" type rigid steel masonry wall anchors are employed at wall corners and wall tee intersections as shown in Figure 4 (Figure 12 of Reference 1).

2.3 Analytical Procedures

2.3.1 Seismic Analysis of Structures

With the exception of several cases involving pressure loads or pipe reactions acting on the CMU walls, the only lateral load which affects the performance of the CMU walls is the result of seismic considerations. To understand the behavior of these walls to lateral load it is therefore important to understand the basis

of these loads as obtained from the seismic analysis of the structures housing the CMU walls.

Input Time History

Two parameters are necessary to define seismic ground motion for the purpose of exciting a structure such as the Davis-Besse auxiliary building. One parameter is the magnitude of the earthquake which is conveniently expressed by the maximum peak acceleration, in terms of gravity (g). The second parameter is related to the frequency content of the earthquake and can best be represented by a design spectra. The design spectra, adjusted to a specific peak earthquake acceleration provides the entire definition of the earthquake necessary to proceed with a seismic analysis of both the primary structure (auxiliary building) and secondary system (CMU walls). Based on the methodology selected to conduct the seismic analysis of the masonry walls, it is necessary to obtain floor response spectra at appropriate building locations. At the time the Davis-Besse seismic analysis was conducted, a necessary intermediate step required an input time history to excite the structure. Since the earthquake motion is completely described by the design spectra, to assure conservatism, it is necessary to develop a time history having a response spectra which envelops the design spectra. The 1935 modified Helena, Montana E-W component time history was employed to envelop the modified Newmark Spectrum. As shown in Figure 5, the time history provides an irregular spectrum which exhibits

substantial exceedences at various frequencies throughout the frequency range, as compared with the modified Newmark Spectrum. The resulting floor response spectra reflects the irregularity of the time history design spectra but is smoothed by enveloping and broadening for design use. By utilizing this approach, the upper bound characteristic of the jagged time history design spectra is included as an additional conservatism in the floor spectra development. This phenomenon is particularly obvious in areas of the spectra away from the structure natural frequencies. Since the enveloped smooth spectra is used to obtain loads for the masonry walls, these conservatisms are directly reflected in the wall moments.

Mathematical Models

Mathematical models of the auxiliary building, which were dynamically excited by the input time history, are shown in Figure 6. Since the building is separated into individual areas by seismic joints, individual models were developed to represent the response of the various building areas. Since the structure is founded on rock at Elevation 545, with the rock having a shear modulus in excess of 5900 ft/sec., the analysis conservatively assumes the foundation as a fixed boundary. The portion of the auxiliary building structure at area 6 is supported on caissons founded in rock at Elevation 558 and extending to Elevation 585. Lateral soil springs between Elevations 567 and 585 represent lateral resistance to deflection provided by the structural

backfill surrounding the caissons. The intake structure seismic model is not shown. It is a cantilever with four masses and a fixed base, since the structure is founded on rock at Elevation 546. Each of the three auxiliary building models was excited in both horizontal directions as well as vertically.

2.3.2 Application to Masonry Walls

Elastic Analysis

The structural response of the masonry walls subjected to out-of-plane seismic inertia loads is based on the elastic behavior of reinforced masonry in flexure. See Table 5 for a summary of the elastic analysis. A FORTRAN computer code BLOCKWALLS was developed to analyze the CMU walls for the effects of external and seismic loads. This computer program, which was described in detail in Reference 1, analyzes walls as simplified three degree of freedom beam models. Use of a three degree of freedom model was verified by comparison of representative results with solutions from a nine degree of freedom model. This comparison shows excellent correlation between results from the three and nine degree of freedom models.

Seismic response of the walls is determined by the modal analysis technique used in conjunction with the response spectrum method. Final inertia loads are based on dynamic response of wall section properties (effective moment of inertia) obtained by an iterative

solution technique. A convergence criteria verifies that the assumed section properties result in similar inertial loading for two successive iterations.

The analysis is conducted by selecting the most severe vertical wall strip, which is limited in width to three times the nominal wall thickness (3t). Depending on support conditions, the top of the wall is modelled as either free or pinned. A pinned boundary condition is considered at the base of the wall except in those cases where the top is free. In this case, the bottom is considered fixed to satisfy stability requirements in the analysis. All external loads within the 3t strip, as well as portions of wall and attached external loads not otherwise supported in the vertical direction but outside the 3t strip, are imposed on the selected wall strip. However, the contributing stiffness and load carrying capability from adjacent wall areas outside this strip are not considered. Stresses are evaluated using the working stress method of analysis. The calculated stresses are checked against established allowables based on the Uniform Building Code, 1970 edition.

3.0 ANALYSIS CONSIDERATIONS

3.1.1 Input Time History

As previously indicated, a modified Helena E-W component time history was used to excite the various buildings, which under OBE

and SSE seismic conditions were considered to display 3 and 4% critical viscous damping, respectively. In addition to the conservatisms generated by the enveloping technique employed for the floor response spectra discussed in Section 2.3.1, additional conservatisms exist in the input time history. This can be best illustrated by comparison of the floor response spectra from the original analysis to analysis results obtained later (in 1980) using (1) the criteria imposed by Regulatory Guides 1.60 and 1.61 and (2) a g level increased from the site licensed level of 0.15g to 0.20g. A typical comparison of floor response spectra for these two analyses, shown in Figure 7, demonstrates that even for an increased g level, the Reg. Guide 1.60/1.61 response is less than that obtained from the original analysis. For the specific structure and floor level shown, a reduction to 80% of the original response was realized. Of course, the amount of reduction varies for other structures at different floor levels. A detailed review of the two analyses provides the reductions shown in Table 6. The analysis using Reg. Guide 1.60/1.61 criteria imposes response from a synthetic time history shown in Figure 8.

This analysis was not conducted specifically for this issue, nor does it represent a licensing commitment. Rather, these analysis results were requested by the ACRS during licensing activities to demonstrate that the analysis of record provides conservative results even if a 0.2g earthquake were to be considered. In the

spirit of honoring the previous interests of the NRC staff and ACRS, the evaluation presented in this report is based on the results of that analysis, imposing a 0.2g earthquake rather than the 0.15g earthquake and using Regulatory Guide 1.60/1.61 criteria. Otherwise all the licensing commitments of seismic analysis originally imposed on this site are maintained.

3.1.2 Modeling Techniques

Another area where substantial conservatisms exist in the re-evaluation presented in Reference 1 involves the modeling techniques used to analyze the walls.

Boundary Conditions

All previous analyses assumed pinned conditions at all boundaries except for cantilever conditions which were assumed fixed. However, as indicated in Section 2.2, except in the rare case where the wall is not attached at least partial restraint exists on all sides. Evaluating the actual stiffness of the end connections compared to that of the wall, and conservatively using the minimum expected moment redistribution, at least 20% of the maximum moment at the center of the wall will be transferred through the top and bottom supports. The result is that the effective moment which the wall experiences is reduced to at least 80% of that obtained when assuming pinned boundary conditions. Both top and bottom connections were checked to assure that the existing details will allow adequate transfer of at least 20% of the wall moment.

In some instances, due to the dimensions of the wall, it was more appropriate to analyze the wall as a horizontal strip rather than a vertical strip. This approach was used when it was obvious that due to the relative stiffness of the horizontal versus vertical spans a majority of the load would be transferred to the adjacent horizontal walls. In a manner similar to that for the vertical strip analysis, all of the load was conservatively assumed to be transferred in one direction.

Anchorage details at the edges for horizontal spans are sufficient for moment distribution. Horizontal reinforcing steel is either anchored into existing concrete walls by lapping with 3/8 inch diameter by two foot long all-thread bars inserted into expansion shields or into existing CMU walls by lapping with "Z" type rigid steel masonry anchors. The result is that moment distribution is realized such that an actual reduction of 67% of the original moment is experienced on the section as shown in Figure 10. Maximum moments occur at the connections rather than the mid-span of the wall.

Plate Action

A second conservatism resulting from the strip analysis is the disregard of plate action. More specifically, the width of the strip in these analyses is limited to three times the nominal masonry block thickness. To evaluate the conservative nature of this assumption, a finite element analysis using the BSAP Computer code was conducted on a wall panel representing typical dimensions of those found at the Davis-Besse Station (see Figure 11). To maintain a conservative basis for this study, the

panel was considered pinned continuously on top and bottom with both sides free, to minimize the amount of lateral load transfer. The results show that the effective width of a single direction vertical strip similar to that used in the original re-evaluation would have to be increased from 3t to 15t (12 feet) to produce the same moment and to approximately 19t (15 feet) to provide deflections similar to the finite element model responding to a concentrated load at the center of the span. The finite element model had orthotropic properties representing a proper ratio of the vertical to horizontal stiffness of the CMU walls.

Although this study provides no specific reduction factor which can be directly applied to the previous re-evaluation results, it does provide clues regarding conservatisms in the original analysis. The approach used in the original re-evaluation was to locate the 3t strip at the most severely loaded portion of wall, artificially and conservatively forcing the applied loads to be transferred only through the designated strip to wall boundaries. Very often the most severe wall section occurred because of a relatively large concentrated load or an adjacent blockout. Although adjacent less loaded portions of wall are available to transfer a portion of this load, its availability was conservatively ignored in the original strip analysis.

Perhaps a better means of determining the conservatism existing in the strip analysis is to compare the strip analysis results with that of plate analysis results on the same walls. The Davis-Besse Station has 30 walls which were analyzed by both strip and plate methods. Many of

these walls are not included in the 74 walls under consideration, since the plate analysis previously conducted, using the BSAP program, in accordance with procedures acceptable to the NRC staff as outlined in Reference 1 resulted in the walls being qualified by elastic analysis.

As shown in Table 7, results of those 30 plate analyses range from 3% to 84% of the response obtained from a strip analysis. It is estimated that the application of a plate analysis creates a reduction of at least 15% to 20% due solely to the redistribution of loads to all four wall boundaries and an additional reduction of approximately 20%, or greater, due to a more accurate distribution of concentrated loads. However, this estimate has not been confirmed by analysis. Substantial reductions also occur in some cases by defining a more accurate wall natural frequency, beyond the peak of the floor response spectra. In summary, a plate analysis, although more complex and time consuming than a strip analysis, provides more realistic results with substantially reduced responses.

Effects of Cutouts

In addition to other considerations, the plate analysis also considers blockouts in the wall. Therefore, reduction factors imposed on the strip analysis method were obtained from a comparison with plate analyses which included all types of blockouts. The finite element mesh sizes vary with individual walls in an attempt to provide adequate models that produce representative results. Although this modeling technique may not capture extremely local stress concentrations, it provides sufficient information

to access the overall behavior of the wall around the periphery of blockouts to evaluate the adequacy of existing designs.

3.1.3 Response Spectra Modification

As discussed in Section 2.3.2, the vertical strip used for analysis is selected in a manner that maximizes force (mass) and minimizes available stiffness and strength, thereby producing a lower bound wall natural frequency and capacity. Since the actual wall natural frequency may be higher than predicted, and to assure that the seismic loads are conservative, the floor response spectra for masonry walls were modified as shown in Figure 9. This modification imposes the peak acceleration levels at all frequencies below the peak frequency. It is reasonable to expect that the wall natural frequency may be higher than predicted by the analytical methods selected since the stiffness denoted by the strip ignores contributing stiffness from the perpendicular direction in plate action. However, accompanying the increased stiffness is increased strength. Thus the potential for substantial overconservatism exists. If the vertical strip is representative of actual conditions and produces natural frequencies below that of the peak natural frequency, the seismic "g" level force can be substantially overestimated, resulting in a vast underestimaticu of the wall ability to resist a seismic event.

Conversely, in those cases where the actual wall natural frequency is higher than predicted, the modified floor response spectra account for the higher seismic loads, but the accompanying increase in wall strength is not considered. Thus the capacity of the wall may be substantially

underpredicted. Since the magnitude of these two effects vary from wall to wall a conservative generic reduction factor to account for this phenomenon is not practical. Nonetheless, on a case-by-case condition, additional conservatism exists.

3.1.4 Moment Combination

An additional conservatism occurs in the BLOCKWALL program related to the treatment of external moments. Specifically, the combination of external moment and maximum seismic inertial moment are combined as an absolute sum regardless of their location on the wall. Thus if a peak external moment is applied at the top of a wall and the peak inertial moment occurs at the bottom of the wall, stresses in the wall are evaluated for a bending moment equal to the absolute sum of the two.

Since conservatisms associated with external moment applications vary from wall to wall, depending upon the magnitude of external moments imposed, a generic factor cannot conservatively be applied to all walls. Again, on a case by case application this conservatism can be demonstrated.

3.1.5 Material Properties

Another area where valid conservatisms can be defined is the comparison of actual to assumed design strength of materials. In accordance with good design practice, the capacity of the masonry walls is controlled by flexure due to the strength of the reinforcing steel. In the original

re-evaluation, the minimum specified yield strength of the reinforcing steel was used as the basis to establish allowable stresses.

Vertical Reinforcing Steel

Table 8 presents a summary of yield and tensile strengths taken from certified material test results for all the No. 5 reinforcing bars used for masonry walls at Davis-Besse. The minimum yield strength of the No. 5 reinforcing steel is 50.6 ksi as compared to a minimum specified of 40 ksi. A minimum yield strength of 50.6 ksi, regardless of bar size, was used for all re-evaluations with the exception of CMU wall 5367. CMU wall 5367 is a four inch thick wall with one row of No. 3 reinforcing bars. Certified material test results for No. 3 reinforcing bars, used in masonary wall construction, show a minimum yield strength of 53.6 ksi (Reference Table 8A). See section 4.3 for a further discussion of CMU wall 5367. 1

Since the allowable stresses used in the re-evaluation were based on a proportion of yield strength, the capacity of the walls with No. 5 bars or larger are at least 25% greater than considered in the calculations. 1 This increased capacity of the walls can be expressed in terms of reducing the effective loads by a factor of 100/125 = 0.8, due to the linear aspect of the analysis. Therefore, the effect of the increased load capacity due to higher strength reinforcing steel is the same as reducing the loads to 80% of their original level, if the wall capacity is assumed to remain at its design level. Similar factors can be calculated for the larger diameter reinforcing bars. The minimum compressive strength, as determined by tests, of the masonry material,

masonry mortar and concrete fill is higher than the minimum specified of 2500 psi. A reduction factor to be applied to the loads to account for this increased strength could be calculated. However, since the wall capacity is primarily controlled by flexure, and reinforcing steel continues to control the design capacity even if the actual strength of steel is considered, further consideration of concrete or masonry strength is secondary.

Horizontal Reinforcing Steel

Reinforcing steel in the horizontal direction is DUR-O-WALL. DUR-O-WALL joint reinforcing as employed at Davis-Besse is an inherently ductile material. The ductile nature of this material permits the selection of allowable stresses which approach the minimum specified yield stress for extreme loading conditions such as the SSE.

Figures 12 and 13 show stress-strain relationships (from References 3 and 4) for cold-drawn wire typical of that used in the manufacture of masonry joint reinforcing (DUR-O-WALL). The tests reported in References 3 and 4 were performed on welded wire fabric (WWF) meeting the following wire properties:

a. Plain wires:

ASTM A 82 Standard Specification for Cold-Drawn Steel Wire for Concrete Reinforcement

b. Deformed wires:

ASTM A 496 Standard Specification for Deformed Steel Wire for Concrete Reinforcement

The joint reinforcing at Davis-Besse consists of 3/16-inch (0.1875 inch) diameter longitudinal deformed wire with 9 gage (0.148 inch diameter) plain web, both conforming to ASTM A 82. Therefore, the stress-strain curves for plain and deformed wire (ASTM A 82 and ASTM A 496) shown in Figures 12 and 13 are representative of DUR-O-WALL wire. These curves are based on stress-strain data selected from References 3 and 4 for wire size approximating the diameter of the longitudinal DUR-O-WALL wire (3/16 or 0.1875 inch). A comparison of the minimum required physical properties for ASTM A 82 and ASTM A 496 wire in this size range is shown in Table 9. The requirements are similar except for the bend tests, which are less restrictive for the deformed wire (ASTM A 496). A comparison of Figures 12 and 13 however, shows very little difference in stress-strain characteristics for deformed and plain wire. Both sets of curves reflect ductile behavior similar to that for ASTM A615 reinforcing steel.

Masonry and Concrete

The results of tests performed in accordance with applicable ASTM standards show the following. The concrete used as fill in the cells of the masonry units had a minimum compressive strength of 4000 psi at four days, the masonry mortar had a minimum compressive strength of 3300 psi at seven days, and the masonry units had a minimum

compressive strength based on net area of 2900 psi. All the tests results show all of the above described material to have compressive strengths above the minimum specified of 2500 psi.

3.1.6 Conduit Loads

A substantial portion of the external loads imposed on the CMU walls of the Davis-Besse Station occur as the result of seismic considerations from conduit supports. Lacking specific information within the time frame required to submit a formal response to IE Bulletin 80-11, it was generally assumed that all conduits were loaded to maximum allowable fill. However, large quantities of conduit are known to have much less than the maximum allowable fill. To quantify this consideration, a statistical analysis based on a reduced sample size, randomly selected, was conducted. The result (shown in Figure 14) is that if at least four conduits are present, there is a 95% confidence level that the average fill of conduits is no more than 30%, or approximately 75% the assumed conduit load considered in the re-evaluation. If the wall has six conduits, the confidence level increases to 98% that the average fill does not exceed 30%. As the number of conduits increase to 20, the average fill reduces to no greater than 25% fill resulting in a load of approximately 65% of that considered in the re-evaluation. Although this indicates that additional conservatism exists in many of the walls, it was not considered in the wall by wall re-evaluation.

4.0 Discussion of Review

4.1 Criteria for Review

Since the energy balance technique has not been accepted by the NRC to qualify masonry walls but substantial margins were known to exist in the analysis, the alternative to maintain an elastic analysis as provided in the NRC Safety Evaluation Report was adopted as reported herein. Many of the conservatisms identified are generic in nature since groups of walls exhibit the same properties. A simplified but acceptable approach is to quantify the generic conservatisms in the form of a common reduction factor applicable to a group of walls. These reduction factors are then directly applied to the controlling reinforcing steel stresses as determined in the original re-evaluation used to prepare Reference 2. If the revised reinforcing steel stress is less than 90% of the minimum specified yield stress the wall is acceptable for the SSE consideration. In general, the SSE load case controls the design. Exceptions will be discussed later.

Conservatisms have been identified in the following areas:

o Seismic loads

) - Input time history

- o BLOCKWALL analysis
  - (✓) - Boundary conditions imposed on the analytical model
  - Use of modified floor response spectra
  - Addition of absolute sum of external moments to wall moments obtained from inertia loads
  - (✓) - Use of vertical strip analysis based on 3 times the wall thickness
- (✓) o Use of minimum specified yield strength for reinforcing steel
- o Use of 100% design conduit fill

Of these factors, the items identified by a ( ) were utilized in re-evaluating of the 74 walls previously accepted by the energy balance technique.

4.2 Summary of Results

Results of the evaluation are presented in Tables 10, 11 and 12. These results show that, with five exceptions, all walls meet the elastic acceptance criteria. Five of these walls are the subject of further 1
detailed analysis.

4.3 Review of Specific Walls by Plate Analytical Techniques

CMU walls 1038, 237I, 5157, 5197 and 5367 were re-evaluated using the 1
computer code BSAP. The results are presented in Table 13. Benefit was made of several items as discussed previously and as outlined below:

- The floor response curves generated for a g input of 0.2 in accordance with Regulatory Guides 1.60 and 1.61 were employed.

- The minimum yield strength of the vertical reinforcing steel was
assumed to be 50.6 ksi for walls 1038, 237I, 5157 and 5197, and 1
53.6 ksi for wall 5367.

- Partial fixity was calculated and input for all boundary conditions.

- CMU wall 237I was walked down again and more precise attachment loads from conduit supports were calculated.

- A multiple mode response spectrum analysis was employed in which the results of the multiple modes were combined in a SRSS fashion.

As can be seen, all reinforcing bar and masonry stresses pass elastic acceptance criteria and further demonstrate the inherent conservatisms in a strip versus plate analysis.

4.4 Application of Results to OBE

The re-evaluation concentrated on the acceptability of the walls to withstand the SSE, in combination with other necessary loads, since this represents the most severe loading environment that will be imposed on the walls. If the walls are shown to withstand the SSE and not damage any adjacent safety-related equipment, safety will be ensured. However, in accordance with the criteria included in the SAR, load cases including the OBE are to be considered.

As expected, the wall response to the OBE is less than that for the SSE. As discussed in Section 3.2.1, a 1980 seismic analysis was utilized in the re-evaluation. That analysis considered only the effects of an SSE. Rather than conduct separate analyses, the response from the 0.2g Regulatory Guides 1.60 and 1.61 analysis was adjusted downward to simulate the OBE earthquake. Linear scaling of the results is valid since structural response is linearly proportional to the input motion. These adjustments include corrections for (1) earthquake g level, (2) acceptable damping level of the main structure and (3) allowable damping level associated with the masonry wall.

Since the re-evaluation was conducted for a 0.2g earthquake and the OBE is specified as 0.08g, a reduction factor 0.4 can be applied to the SSE results.

The second factor accounts for the differential peak amplification of the building motion at a given floor level due to the difference in allowable

building damping. The SSE analysis using Regulatory Guide 1.60 input motion considered 7% damping for reinforced concrete structures in accordance with Regulatory Guide 1.61. However, only 4% damping is allowed for the OBE. Reference 5 provides peak amplification factors for the control points of the Regulatory Guide 1.60 response spectra as a function of building damping, reproduced as Figure 15 herein. At control point B (9 H_z) the amplification is 2.84 for 4% damping and is 2.27 for 7% damping. Thus the reduced damping would be expected to increase the peak floor response in the building by a factor of 2.84/2.27 = 1.25. A similar amplification occurs at control point C (2.5 H_z), but above 9 H_z the ratio of amplification reduces.

The third factor accounts for the change in amplification from a difference of 7% to 4% damping on the masonry wall. This was obtained by comparing the peak response of the 0.2g analysis for 4% and 7% response spectra curves. The results are summarized in Table 14, and indicate a general trend of increased 4% damping amplification with building elevation. With one exception (peak amplification of 1.50) the 4% damping results in a peak amplification of 1.45 over the 7% masonry wall damping consideration.

Combining all of these factors, the OBE response is expected to be 0.4 x 1.25 x 1.45 = 0.725 times the level of the SSE response. This is an upper bound response for the OBE. For example, the inclusion of soil structure interaction would have increased the damping levels of both the OBE and SSE so that the ratio of OBE building response to that from the

SSE would be reduced. Additionally, the amplification factor provided for the masonry wall damping is valid only at the peak of the spectra. At all other frequencies the ratio of OBE to SSE will be reduced.

Since the ratio of computed to allowable reinforcing steel stresses for OBE to SSE is 25/36 = 0.694, the possibility exists that in a few cases the OBE may be the controlling load case by a small factor. Comparing the reduction in response to reduction in allowable stresses from OBE to SSE, suggests that in certain cases, the ratio of computed to allowable stresses could be approximately 4-1/2% greater for the OBE case. All 74 CMU walls were examined to determine if OBE stresses exceed allowables. With the exception of the five cases where on initial re-evaluation the SSE allowables were exceeded, walls meet the OBE allowables. Reanalysis of these five walls as specified in Section 4.3 has 1
resolved the concern regarding these walls.

1

5.0 CONCLUSIONS

The purpose of this study was to review the original elastic-working stress analysis conducted on the subject 74 walls to identify conservatisms in excess of those necessary to meet minimum licensing commitments.

A number of conservatisms were identified and are summarized in Section 4.0. For some of these items, a "generic" lower bound level of conservatism can be easily and clearly identified for a group of walls.

For other items, a wall-by-wall review is necessary to quantify the amount of conservatism. In keeping with the intent of the study, and for clarity in presentation, only the following generic items were quantified.

- Use of a more current definition of input time history and associated damping factors (Reg. Guides 1.60/1.61).

- More realistic boundary conditions for the vertical and horizontal strip analysis.

- Use of correction factors to simulate plate analysis.

- Use of as-built reinforcing steel strength properties.

In addition, for certain walls correction factors were introduced to update analysis results so that they are consistent with the criteria originally imposed on the response to IE Bulletin 80-11. More specifically, although 4% and 7% critical damping is acceptable for analysis of CMU walls for OBE and SSE conditions respectively, the original re-evaluation results of some walls reported in the response to IE Bulletin 80-11 considered only 2% or 4% damping. Likewise, a correction factor was necessary to correct results of the original analysis using the Helena earthquake time history for OBE to SSE considerations. Contrary to the Regulatory Guide 1.60/1.61 analysis, and normal expectations, the peaks of the OBE floor response spectra in the

original seismic analysis were, in general, conservatively equal to or greater than the peaks of the SSE. However, the OBE allowables were lower, so that in those cases the OBE naturally would control. A factor of 0.7 was conservatively applied to adjust the original OBE results to SSE. In this manner, further evaluation would continue on an equal basis. Note that the discussion in Section 4.4 differs in that it evaluates the difference between an OBE and SSE using Regulatory Guide 1.60/1.61 input.

Considering these factors as applicable, for each of the 74 subject walls, all but five walls are acceptable. Further detailed analysis
conducted on these walls as reported in Section 4.3 bring these walls 1
within acceptable stress levels. The result is that all walls in question meet the acceptance criteria as specified in the Toledo Edison response to IE Bulletin 80-11 for elastic-working stress analysis.

REFERENCES

1. R. P. Crouse Letter to J. G. Keppler, NRC. Subject: Response to Item 2b and expanded response to Item 3 of IE Bulletin 80-11 for Davis-Besse Nuclear Power Station Unit No. 1, Toledo Edison, November 4, 1980, Serial No. 1-169.

2. Safety Evaluation Report, Masonry Wall Design, IE Bulletin 80-11, Davis-Besse Nuclear Power Station Unit 1, Docket No. 50-346, Structural and Geotechnical Branch, Structural Engineering Section A.

3. Investigation of Stress-Strain Characteristics of Plain Wire, Wire Reinforcement Institute, Wiss, Janney, Elstner & Associates (September 1969).

4. Investigation of Stress-Strain Characteristics of Plain Wire, Wire Reinforcement Institute, Wiss, Janney, Elstner & Associates (October 1969).

5. Design Response Spectra for Nuclear Power Plants, N. M. Newmark, J. A. Blume and K. K. Kapus.

TABLE 1
REVIEW OF WALLS EVALUATED BY ENERGY BALANCE PROCEDURE
ENERGY BALANCE BY VERTICAL WALL STRIP

ELEV.	AREA	WALL NO.	HEIGHT (ft.)	THICKNESS (in.)	DUCTILITY RATIO	fs / Fall	COMMENTS
545	7	1087	8-0	12	0.71	1.03	2% OBE (1), CANTILEVERED (2)
545	7	1147	8-6	12	0.93	1.03	
545	7	1197	15-0	8-8-8(4)	1.18	1.30	
545	7	1227	18-6	8-8-8	2.04	1.95	4% SSE (3)
545	7	1237	18-6	8-2-8	2.87	3.49	2% OBE
545	7	1267	18-0	8-8-8	2.07	1.97	4% SSE
545	7	1337	16-7	8-8-8	0.95	1.05	
545	8	1038	18-0	12-24-12	3.0	2.58	
545	8	1348	13-6	8-8-8	1.14	1.26	4% SSE
545	8	1428	13-6	8-8-8	0.83	0.91	4% SSE
565	7	2057	10-0	8-2-8	1.08	1.72	2% OBE
565	7	2067	10-0	8-2-8	0.71	1.02	2% OBE
565	7	2087	17-0	12	1.03	1.65	2% OBE
565	7	2107	18-1	12-12-12	1.10	1.22	
565	7	2147	18-1	12	1.45	2.21	2% OBE
565	7	2177	18-1	8-2-8	1.08	1.19	
565	7	2237	18-1	12-6-12	0.74	0.78	
565	7	2247	17-1	12-6-12	0.96	1.07	
565	7	2257	18-2	12	1.94	1.88	
565	7	2277	18-4	12	0.93	1.48	2% OBE
565	7	2317	15-5	12	1.00	1.11	
565	7	2337	10-0	12	1.17	1.28	Cantilevered
565	7	2367	18-2	12	1.76	1.77	
565	7	2447	16-10	12	1.48	1.55	
565	8	2018	17-0	12	0.99	1.60	
585	7	3227	16-10	12	2.19	1.90	
585	7	3257	16-11	12	1.16	1.84	2% OBE
585	7	3267	17-0	12	2.61	2.07	
585	7	3307	14-4	12	1.71	2.49	4% OBE
585	7	3367	17-0	12	1.80	1.69	
585	7	3407	14-9	12	2.81	2.39	
603	7	4917	9-6	8	1.50	1.58	
623	7	5017	11-5	8	2.17	2.03	
623	7	5107	11-5	12	1.35	1.45	
623	7	5127	14-0	8	2.10	1.99	
623	7	5147	13-11	12	1.49	1.56	
623	7	5157	14-0	8	4.30	3.07	
623	7	5187	14-0	8	2.56	2.25	
623	7	5197	14-0	8	2.48	2.21	
623	7	5277	14-0	8	2.88	2.42	
643	7	6087	14-2	12	1.36	1.46	
585	6	305D	13-0	12	1.36	1.46	
585	6	307D	13-1	12	0.79	0.85	
585	6	313D	8-6	12	1.30	2.02	2% OBE
576	Intake Struct.	237I	13-4	8	3.64	2.79	

Notes:
(1) Original evaluation based on OBE case considering 2% masonry wall damping (criteria allows 4%).
(2) Original evaluation considers wall section as a cantilevered beam with bottom support fixed and top free.
(3) Original evaluation based on SSE case considering 4% masonry wall damping (criteria allows 7%).
(4) 8-8-8 indicates multiple wythe wall with outside 8" reinforced masonry units with center 8" concrete fill.

TABLE 2

REVIEW OF WALLS EVALUATED BY ENERGY BALANCE PROCEDURE
ENERGY BALANCE BY HORIZONTAL STRIP

ELEV.	AREA	WALL NO.	SPAN (ft.)	THICKNESS (in.)	DUCTILITY RATIO	fs / Fall	COMMENTS
545	7	1157	5-0	8-8-8(4)	0.83	0.91	
565	7	2077	9-0	12	1.17	1.29	
565	7	2167	7-0	8-5-8	1.98	1.91	
565	7	2227	7-4	8-8-8	1.05	1.17	
565	7	2267	6-2	12	1.10	1.22	
565	7	2427	8-5	8	0.97	1.08	
585	7	3167	4-6	12	1.03	1.14	
585	7	3177	11-10	12	0.98	1.08	
585	7	3187	3-3	12	1.07	1.15	
585	7	3347	14-5	12	0.86	0.94	
585	7	3397	8-4	12	2.35	3.33	2% OBE (1)
585	7	3417	6-5	12	2.00	1.86	4% SSE (2)
603	6	4016	5-4	12	0.77	1.28	4% OBE
603	7	4647	3-4	12	1.40	1.26	CANTILEVERED (3)
623	7	5367	6-2	4	4.54	3.16	
643	7	6107	8-10	12	2.41	3.39	2% OBE

Notes: (1) Original evaluation based on OBE case considering 2% damping for masonry wall (criteria allows 4% damping).

(2) Original evaluation based on SSE case considering 4% damping for masonry walls (criteria allows 7% damping).

(3) Original evaluation considers wall section as a cantilever beam with one side fixed and other side free.

(4) 8-8-8 indicates multiple wythe wall with outside 8" reinforced masonry units with center 8" concrete fill.

TABLE 3

REVIEW OF WALLS EVALUATED BY ENERGY BALANCE PROCEDURE
ENERGY BALANCE BY TWO WAY ACTION

ELEV.	AREA	WALL NO.	HEIGHT (ft.)	SPAN (ft.)	THICKNESS (in.)	HORIZONTAL DUCTILITY RATIO	HORIZONTAL fs/Fall	VERTICAL DUCTILITY RATIO	VERTICAL fs/Fall
585	6	3036	16-5	13-0	12	-	0.62	0.88	1.39 (1)
585	7	3277	15-8	9-0	12	0.95	1.04	-	0.53
585	7	3287	16-10	8-0	12	0.95	1.06	-	0.89
585	7	3297	14-8	9-0	12	2.64	1.61	-	0.84
585	7	3357	14-8	22-10	12	0.98	1.09	-	0.61
603	6	4036	18-5	13-5	12	-	0.52	2.72	2.34
603	6	4046	18-5	10-0	12	-	1.01	0.76	1.16
603	6	4796 }	18-5	13-0	12	1.20	1.31	2.46	2.21
603	6	4886 }							
603	6	4896 }							
603	7	4867	8-2	1-8	12	1.40	1.26	-	0.39
585	6	304D	12-8	12-0	12	1.64	2.62	-	0.75 (1)
585	6	311D	23-1	10-3	8-2-8	0.89	1.54	0.85	1.34 (1)

(1) Qualified for OBE 4% Masonry Wall Damping. Remaining walls qualified for SSE 7% masonry wall damping.

TABLE 4

MASONRY WALL CONSTRUCTION DETAILS

MASONRY UNITS: ASTM C-90 GRADE N-1 (MINIMUM SPECIFIED COMPRESSIVE STRENGTH ON GROSS AREA = 1500 PSI FOR GROUTED UNITS AND 1350 PSI FOR PARTIALLY GROUTED OR HOLLOW UNITS)

MORTAR: ASTM C-476 TYPE PM (MINIMUM SPECIFIED COMPRESSIVE STRENGTH = 2500 PSI)

GROUT: ASTM C-476 (MINIMUM SPECIFIED STRENGTH = 2500 PSI)

REINFORCING STEEL: ASTM A615 GRADE 40 (MINIMUM SPECIFIED YIELD STRENGTH = 40000 PSI)

CONSTRUCTION DETAILS:

VERTICAL REINFORCING	8" BLOCK 2# 5 @ 16"	12" BLOCK 2# 5 @ 16"
HORIZONTAL REINFORCING	DUROWALL EXTRA HEAVY TRUSS TYPE PER ASTM A-82 SPACED AT 8"	

ANCHORAGE DETAILS:

FLOOR: LAPPED WITH ALL-THREAD BARS IN EXPANSION SHIELDS
WALLS: LAPPED WITH ALL-THREAD BARS IN EXPANSION SHIELDS
CEILINGS: LAPPED WITH ALL-THREAD BARS WELDED TO FLAT PLATE

STACKED BOND

TABLE 5

ELASTIC ANALYSIS OF MASONRY WALLS

METHODOLOGY
(UTILIZED COMPUTER PROGRAM BLOCKWALLS)

- o ANALYSIS CONSIDERS WALL AS 3 MASS BEAM WITH FIXED, PINNED OR FREE END CONDITIONS.
- o STIFFNESS DEVELOPED FOR BOTH CRACKED AND UNCRACKED TRANSFORMED SECTIONS.
- o NATURAL FREQUENCIES AND MODE SHAPES COMPUTED.
- o FLOOR RESPONSE SPECTRA READ INTO PROGRAM.
  - USED TO ESTABLISH EXCITED LEVELS FOR EACH MODE
- o MOMENT AND SHEAR DISTRIBUTION DETERMINED BY SRSS.
- o MAXIMUM MOMENTS, SHEAR, MASONRY COMPRESSIVE STRESS AND REINFORCING STEEL STRESS COMPUTED.
- o RESULTS COMPARED WITH ALLOWABLES.

TABLE 6

SEISMIC LOADS

REDUCTION FACTORS FOR USE OF REGULATORY GUIDE 1.60 (.2g) TIME HISTORY

AREA	DIRECTION	PEAK NATURAL FREQUENCY	REDUCTION FACTOR
6	N-S	6.7	0.8
	E-W	6.8	0.6
7	N-S	7.0	0.8 (1)
	E-W	5.2	1.0
8	N-S	9.1	0.6 (2)
	E-W	11.2	0.6 (3)

(1) REDUCTION FACTOR OF 0.85 FOR 1XXX AND 2XXX LEVEL WALLS

(2) REDUCTION FACTOR OF 0.9 FOR 1XXX AND 2XXX LEVEL WALLS

(3) REDUCTION FACTOR OF 0.8 FOR 1XXX AND 2XXX LEVEL WALLS

(4) 7% DAMPING USED FOR REGULATORY GUIDE 1.60 ANALYSIS

TABLE 7

COMPARISON OF PLATE TO STRIP ANALYSIS

WALL NO.	HEIGHT/WIDTH	REDUCTION FACTOR
1068	0.76	0.60
2207	1.27	0.36
2217	1.10	0.06
306D	1.10	0.19
308D	2.44	0.07
309D	2.55	0.08
310D	2.44	0.08
3247	1.49	0.84
3357	0.64	0.79
338D	2.61	0.08
4046	1.85	0.09
4137	0.65	0.03
5137	2.32	0.04
6037	1.90	0.27
6097	4.54	0.27
3016	1.29	0.41
3026	2.68	0.41
3036	1.27	0.41
3287	2.10	0.30
4036	1.38	0.29
4786 4906	2.68	0.42
4796 4886 4896	1.43	0.18
5207	0.74	
311D	2.26	0.06
3237	1.57	0.72
2297	0.81	0.21
4026	1.62	0.60

TABLE 8

MILL TEST REPORTS FOR MASONRY WALLS
NO. 5 REINFORCING BARS

ASTM SPECIFICATION	YIELD (KSI)	TENSILE (KSI)	QUANTITY OF STEEL REPRESENTED (TONS)
A-615 Grade 40	70.0	113.2	41.9
A-615 Grade 40	50.6	79.7	41.9
A-615 Grade 40	51.0	80.0	15.6
A-615 Grade 40	54.8	86.5	19.8
A-615 Grade 40	56.8	88.4	20.3
A-615 Grade 40	55.5	90.6	20.9
A-615 Grade 40	55.5	87.1	10.4
A-615 Grade 40	54.2	88.4	41.7
A-615 Grade 40	51.0	76.1	31.3
A-615 Grade 40	56.8	88.4	31.3
		TOTAL	275.1 TONS

TABLE 8A

MILL TEST REPORTS FOR MASONRY WALLS
No. 3 REINFORCING BARS

ASTM SPECIFICATION	YIELD (KSI)	TENSILE (KSI)		QUANTITY OF STEEL REPRESENTED (TONS)
A-615 Grade 40	53.6	81.6		0.3
A-615 Grade 40	53.6	81.6		0.9
			TOTAL	1.2 TONS

1

TABLE 9

COMPARISON OF MINIMUM REQUIRED PHYSICAL PROPERTIES FOR ASTM A 82 AND ASTM A 496 WIRE

	Plain Wire ASTM A 82	Deformed Wire ASTM A 496
Minimum Strength (ksi)		
General		
Yield	70	75
Ultimate	80	85
Welded Wire Fabric		
Yield	65 (1)	70
Ultimate	75 (1)	80
Bend Test		
Requirements		
Bend Angle	180 degrees	90 degrees
Pin diameter	One wire diameter (2)	Two wire diameters (3)

(1) Wire size W1.2 (0.124 inch diameter) and larger

(2) Wire size W7 (0.299 inch diameter) and smaller

(3) Wire size D-6 (0.276 inch diameter) and smaller

TABLE 10

SUMMARY

VERTICAL STRIP ANALYSIS

WALL NO.	fs / Fall	SEISMIC ORIENT	REDUCTION FACTORS: ADJUSTMENT TO MATCH ACCEPTANCE CRITERIA DAMP.	SSE	SEISMIC TIME HISTORY	BOUND COND.	MAT PROP.	PLATE ANALYSIS	PRODUCT	REDUCED fs / Fall
2237	0.78	E	-	-	-	.8	.8	-	.64	0.50
307D	0.85	N	-	-	.8	.8	.8	-	.51	0.43
1428	0.91	N	.75	-	-	.9	.8	-	.54	0.49
2067	1.02	N	.7	.7	.85	.8	.8	-	.29	0.30
1087	1.03	N	.7	.7	.85	Cant.	.8	-	.31	0.32
1147	1.03	N	-	-	-	.8	.8	-	.64	0.66
1337	1.05	E	-	-	-	.8	.8	-	.64	0.67
2247	1.07	E	-	-	-	.8	.8	-	.64	0.68
2317	1.11	N	-	-	-	.8	.8	-	.64	0.71
2177	1.19	E	-	-	-	.8	.8	-	.64	0.76
2107	1.22	N	-	-	.85	.8	.8	-	.54	0.66
1348	1.26	E	.75	-	-	.8	.8	-	.48	0.60
2337	1.28	N	-	-	.85	Cant.	.8	-	.68	0.87
1197	1.30	E	-	-	-	.8	.8	-	.64	0.83
305D	1.46	E	-	-	-	.8	.8	-	.64	0.93
6087	1.46	E	-	-	-	.8	.8	-	.64	0.93
2277	1.48	N	.7	.7	.85	.8	.8	-	.27	0.40
2447	1.55	E	-	-	-	.8	.8	-	.64	0.99
5147	1.56	E	-	-	-	.8	.8	-	.64	1.00
4917	1.58	E	-	-	-	.8	.8	0.84	.54	0.84
2018	1.60	N	-	-	.9	.8	.8	-	.58	0.93
2087	1.65	N	.7	.7	.85	.8	.8	-	.29	0.44
3367	1.69	N	-	-	.8	.8	.8	-	.51	0.86
2057	1.72	E	.7	.7	-	.8	.8	-	.29	0.54
2367	1.77	E	-	-	-	.8	.8	0.84	.54	0.95
3257	1.84	E	.7	.7	-	.8	.8	-	.31	0.57
2257	1.88	N	-	-	.85	.8	.8	0.84	.46	0.86
3227	1.90	E	-	-	.97(2)	.8	.8	0.84	.52	0.99
1227	1.95	E	.75	-	-	.8	.8	-	.48	0.94
1267	1.97	E	.75	-	-	.8	.8	-	.48	0.95
5127	1.97	N	-	-	.8	.8	.8	0.84	.43	0.85
313D	2.02	N	.7	.7	.8	.8	.8	-	.25	0.51
5017	2.03	E	-	-	.93(2)	.8	.8	0.84	.50	1.02
3267	2.07	N	-	-	.8	.8	.8	0.84	.43	0.89
2147	2.21	N	.7	.7	.85	.8	.8	-	.29	0.59
5197	2.21	E	-	-	.93(2)	.8	.8	0.84	.50	1.10(1)
5187	2.25	N	-	-	.8	.8	.8	0.84	.43	0.97
3407	2.39	E	-	-	.68(2)	.8	.8	0.84	.37	0.87
5277	2.42	N	-	-	.79(2)	.8	.8	0.84	.42	1.01
3307	2.49	E	-	.7	-	.8	.8	0.84	.38	0.87
1038	2.58	E	-	-	-	.8	.8	0.84	.54	1.39(1)
2371	2.79	N	-	-	.8	.8	.8	0.84	.43	1.19(1)
1237	3.49	N	.7	.7	.85	.8	.8	-	.27	0.93
5157	3.07	N	-	-	.8	.8	.8	0.84	.43	1.32(1)

(1) See Table 13 (2) Reduction based on comparison of floor specific response spectra

TABLE 11

SUMMARY

HORIZONTAL STRIP ANALYSIS

WALL NO.	fs / Fall	SEISMIC ORIENT	REDUCTION FACTORS: ADJUSTMENT TO MATCH ACCEPTANCE CRITERIA DAMP.	SSE	SEISMIC TIME HISTORY	BOUND COND.	MAT PROP.	PLATE ANALYSIS	PRODUCT	REDUCED fs / Fall
4106	0.77	E	-	.7	.6	.7	-	.-	.29	0.23
3347	0.86	N	-	-	-	.7	-	-	.7	0.60
1157	0.91	E	-	-	-	.7	-	-	.7	0.64
2427	0.97	E	-	-	-	.7	-	-	.7	0.68
3177	0.98	E	-	-	-	.7	-	-	.7	0.69
3167	1.03	N	-	-	-	.7	-	-	.7	0.72
3187	1.07	N	-	-	-	.7	-	-	.7	0.75
2267	1.10	N	-	-	-	.7	-	-	.7	0.77
2227	1.17	N	-	-	.85	.7	-	-	.6	0.70
2077	1.29	E	-	-	-	.7	-	-	.7	0.90
4647	1.40	E	-	-	-	.7	-	-	.7	0.98
2167	1.91	N	-	-	.85	.7	-	.84	.5	0.97
3417	2.00	N	.75	-	-	.7	-	.84	.44	0.89
3397	2.35	E	.7	.7	-	.7	-	-	.34	0.80
5367	3.16	E	-	-	-	.7	-	.84	.59	1.99 (1)
6107	3.39	E	.7	.7	-	.7	-	.84	.29	0.97

(1) See Table 13

1

TABLE 12

SUMMARY

TWO WAY ANALYSIS

WALL NO.	CRITICAL DIRECTION	fs / Fall	SEISMIC ORIENT.	REDUCTION FACTORS: ADJUSTMENT TO MATCH ACCEPTANCE CRITERIA DAMP.	REDUCTION FACTORS: ADJUSTMENT TO MATCH ACCEPTANCE CRITERIA SSE	REDUCTION FACTORS: SEISMIC TIME HISTORY	REDUCTION FACTORS: BOUND COND.	REDUCTION FACTORS: MAT PROP.	REDUCTION FACTORS: REDUCTION FOR PLATE ANALYSIS	PRODUCT	REDUCED fs / Fall
3277	H	1.04	E	-	-	-	.8	-	-	.8	0.83
3287	H	1.06	E	-	-	-	.8	-	-	.8	0.85
3357	H	1.09	N	-	-	-	.8	-	-	.8	0.87
4046	V	1.16	E	-	-	.6	.8	-	-	.48	0.56
4867	H	1.26	N	-	-	-	.8	-	-	.8	1.01
3036	V	1.39	N	-	-	.8	.8	.8	-	.51	0.71
311D	H	1.54	N	.7	.7	.8	.8	-	-	.45	0.69
3297	H	1.61	E	-	-	.75	.8	-	-	.6	0.97
4796 4886 4896	V	2.21	N	-	-	.8	.8	.8	0.84	.43	0.95
4036	V	2.34	N	-	-	.8	.8	.8	0.84	.43	1.00
304D	H	2.62	E	-	.7	.6	.8	-	-	.34	0.88

TABLE 13

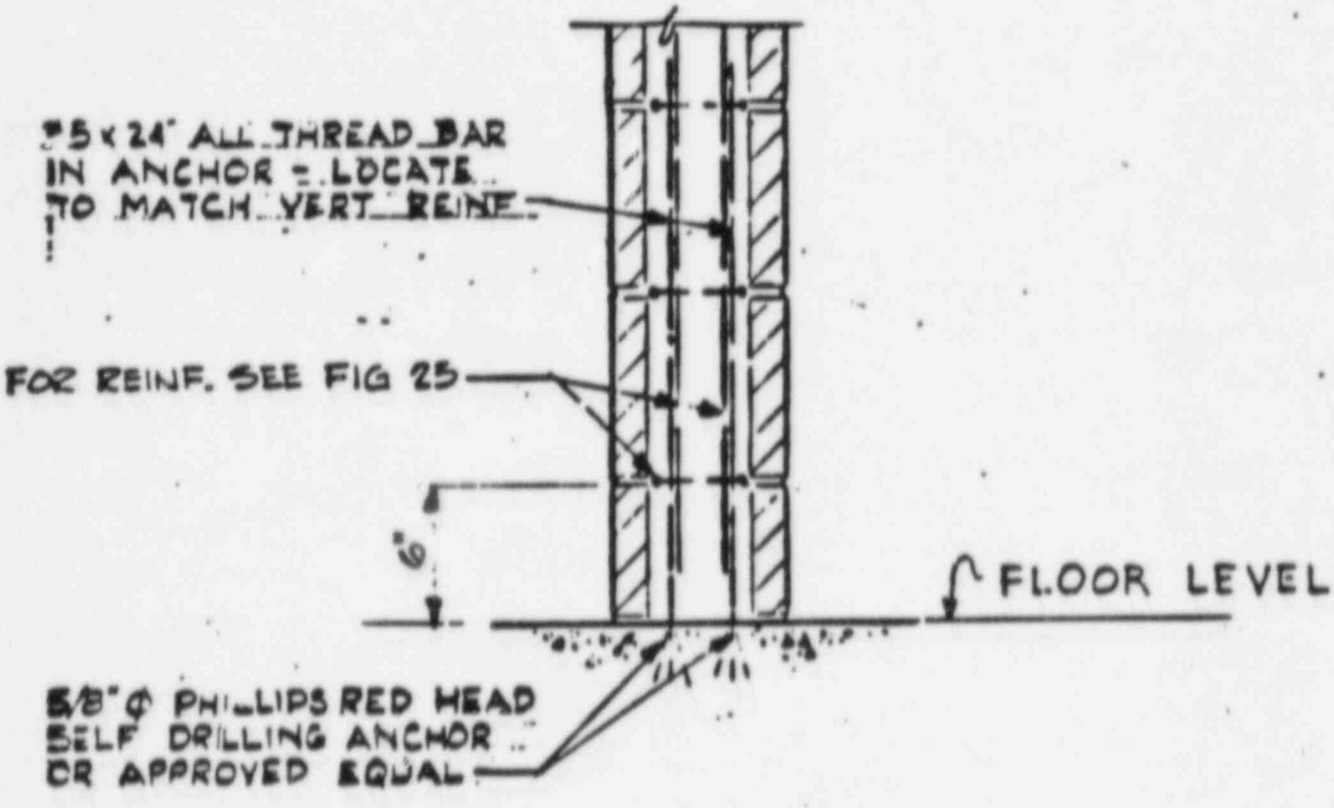
SUMMARY OF REVIEW OF SPECIFIC WALLS BY PLATE ANALYTICAL TECHNIQUES

WALL NO.	VERTICAL SPAN		HORIZONTAL SPAN		
	MAX. REBAR STRESS (KSI)	MAX. MASONRY STRESS (KSI)	MAX. REBAR STRESS (KSI)	MAX. MASONRY STRESS (KSI)	
1038	0.15	0.01	2.47	0.01	
2371	19.87	0.48	20.79	0.16	
5157	1.05	0.09	11.62	0.09	
5197	16.47	0.40	10.07	0.08	
5367	6.63	0.16	38.68	0.36	1

TABLE 14

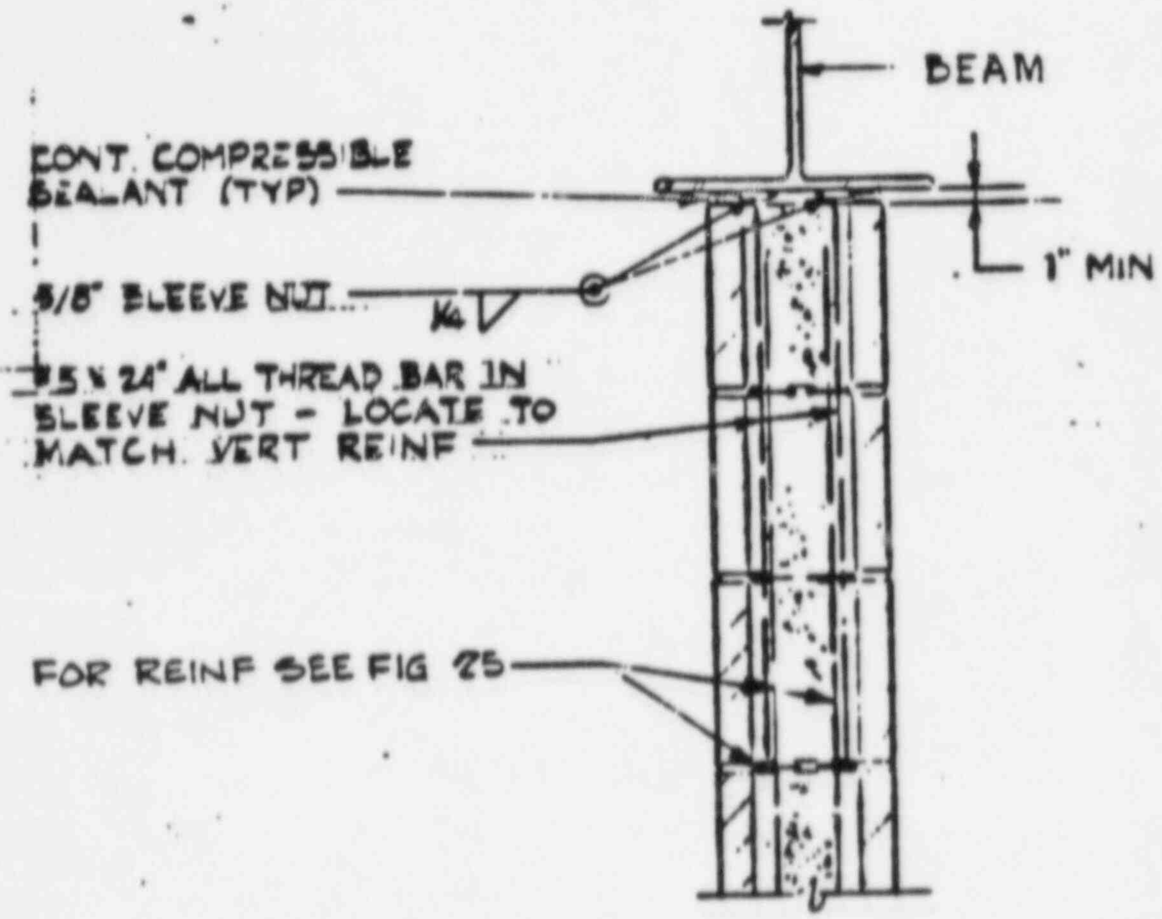
RATIO OF PEAK ACCELERATION OF CMU WALLS BETWEEN 4% AND 7% DAMPING

AUXILIARY BLDG. AREA	ELEV.	NORTH/SOUTH DAMPING 4% (Peak Accel g's)	7%	EXCITATION RATIO (g4%/g7%)	EAST/WEST DAMPING 4% (Peak Accel g's)	7%	EXCITATION RATIO (g4%/g7%)
6	585	2.4	1.9	1.26	2.6	2.25	1.16
	603	2.9	2.5	1.16	3.0	2.7	1.11
7	545			1.25			1.25
	565	0.85	0.67	1.27	0.9	0.6	1.50
	585	1.3	0.95	1.37	1.35	0.95	1.42
	603	2.05	1.45	1.41	2.15	1.5	1.43
	623	2.9	2.05	1.41	3.2	2.2	1.45
	643	3.8	2.6	1.45	4.2	2.9	1.45
8	545			1.25			1.25
	565	0.75	0.58	1.29	0.75	0.58	1.29



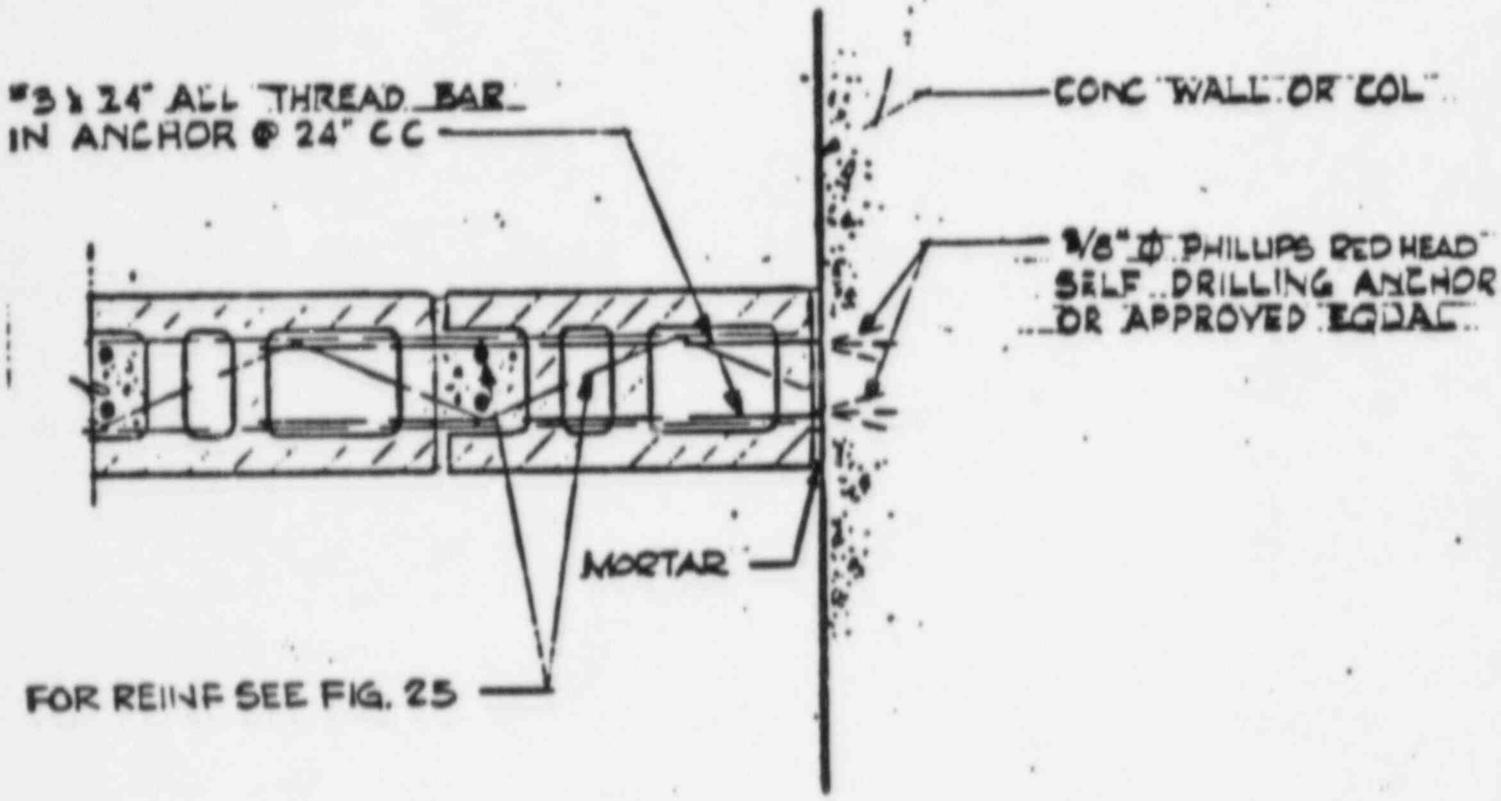
SECTION @ FLOOR LEVEL

FIGURE 1



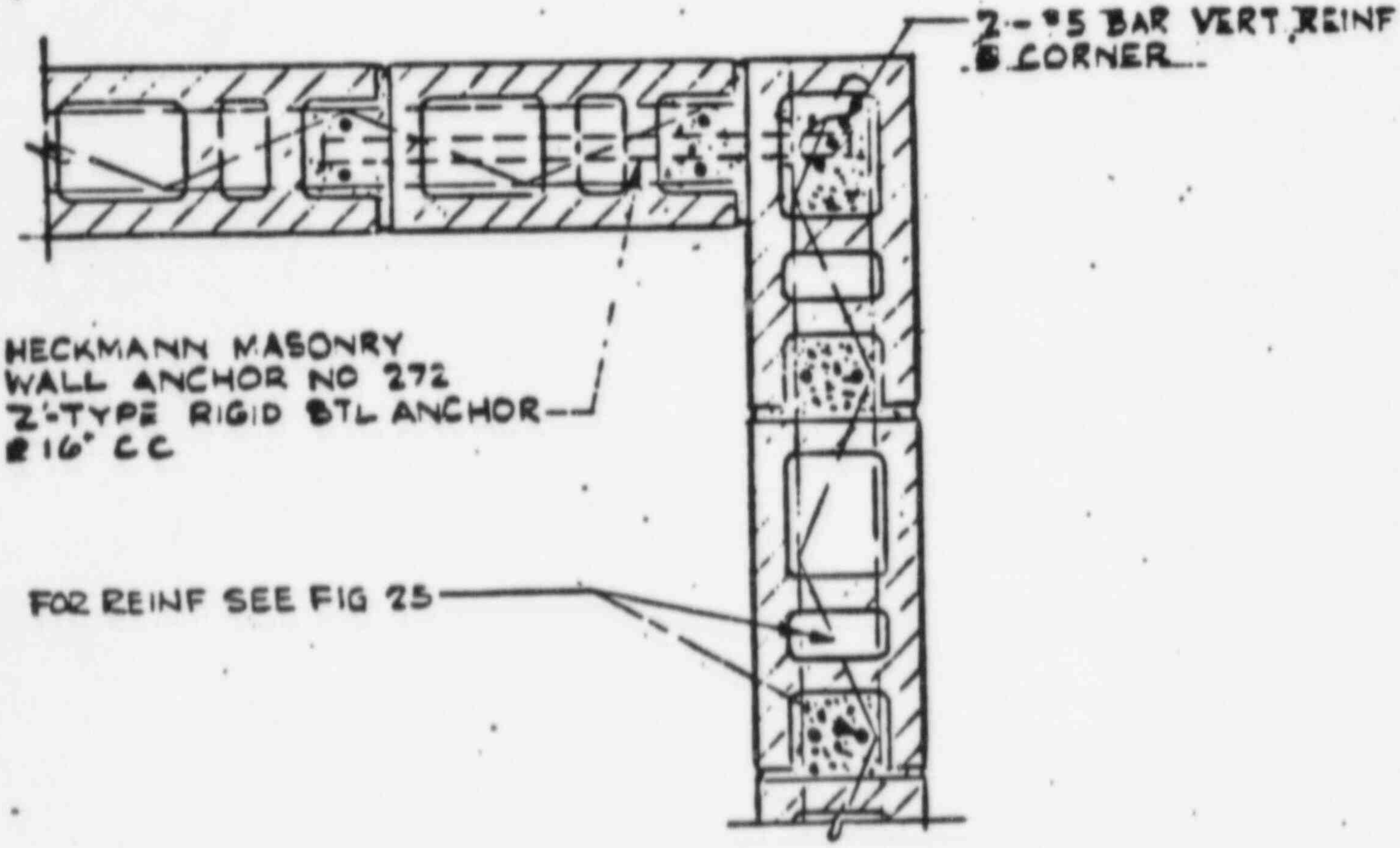
SECTION @ STEEL BEAM

FIGURE 2



PLAN @ CONC WALL OR COLUMN INTERSECTION

FIGURE 3



PLAN @ WALL CORNER

FIGURE 4

DAVIS BESSE

HORIZONTAL ACCELERATION DESIGN SPECTRA

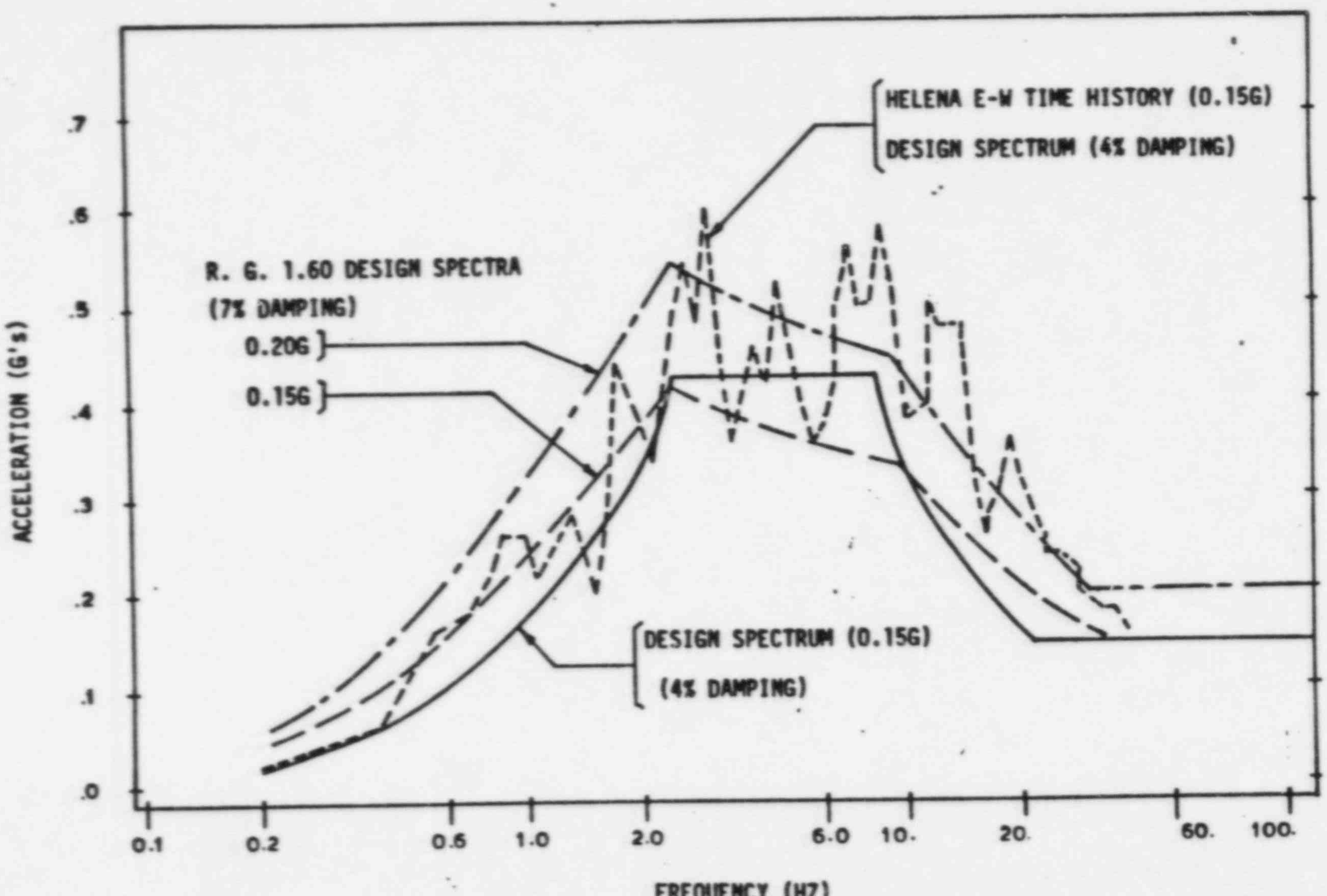


FIGURE 5

[DAVIS BESSE

[AUXILIARY BUILDING

SEISMIC MODELS

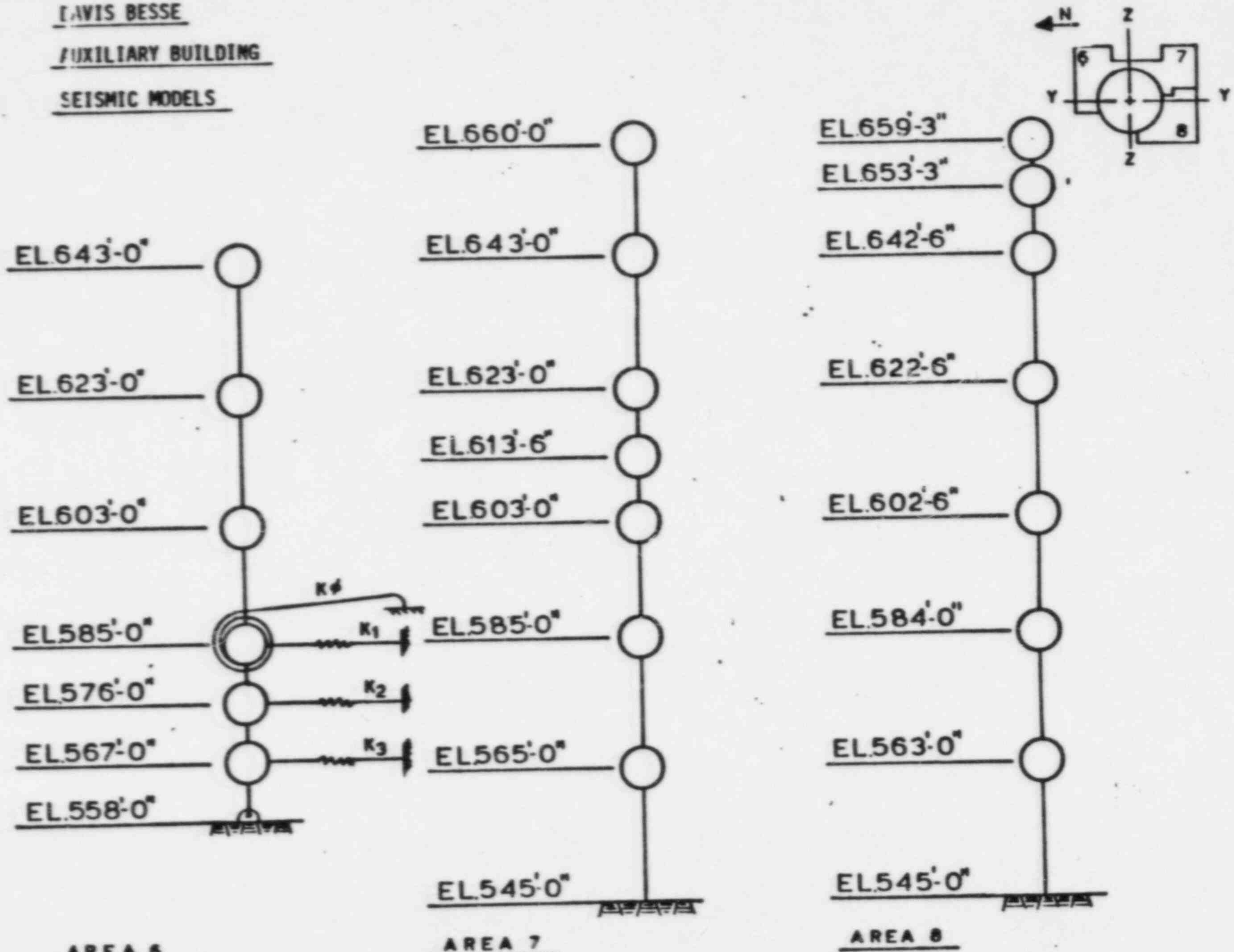
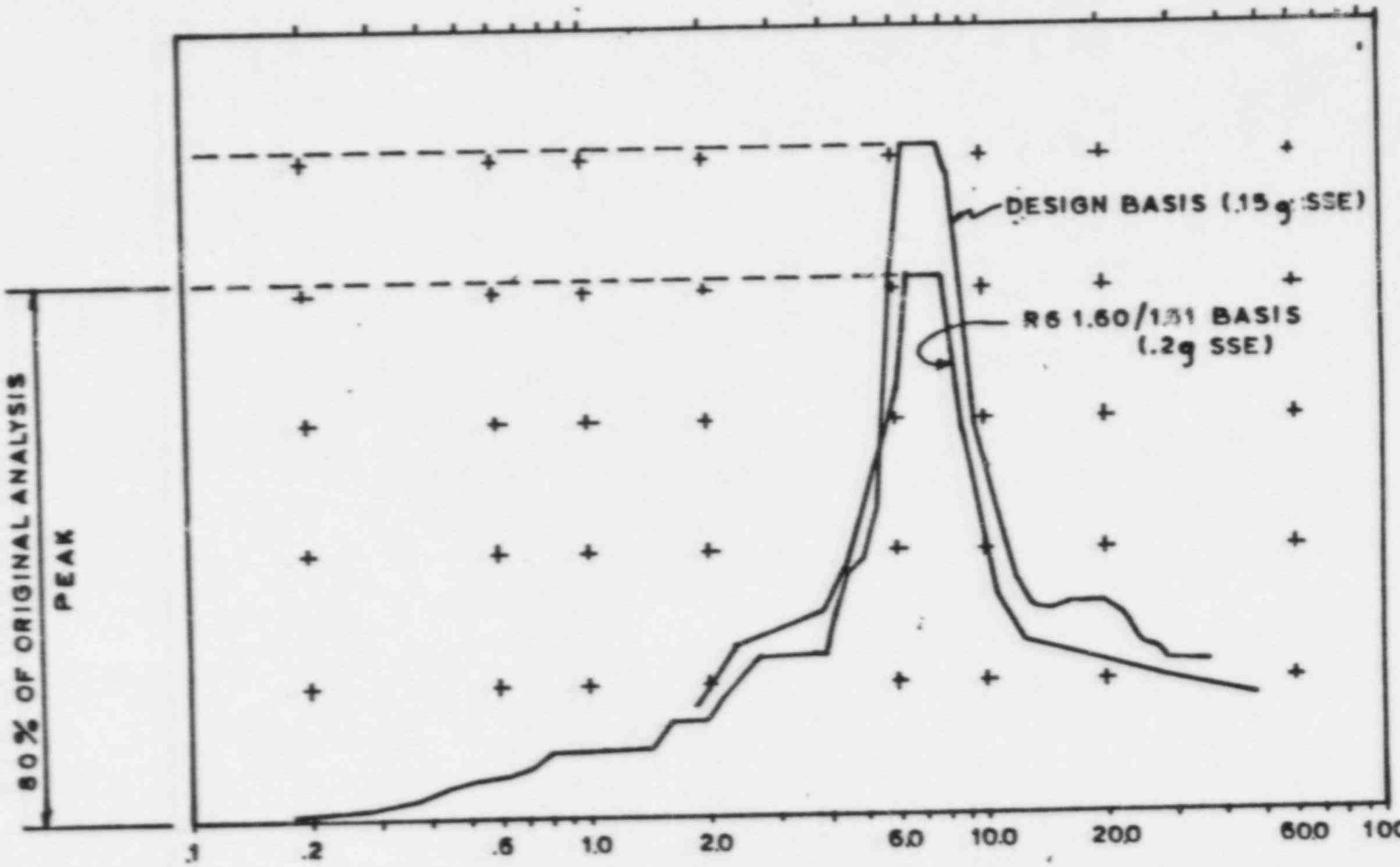


FIGURE 6

DAVIS-BESSE UNIT 1 AUX BLDG AREA 7 NS FLOOR RESPONSE SPECTRA

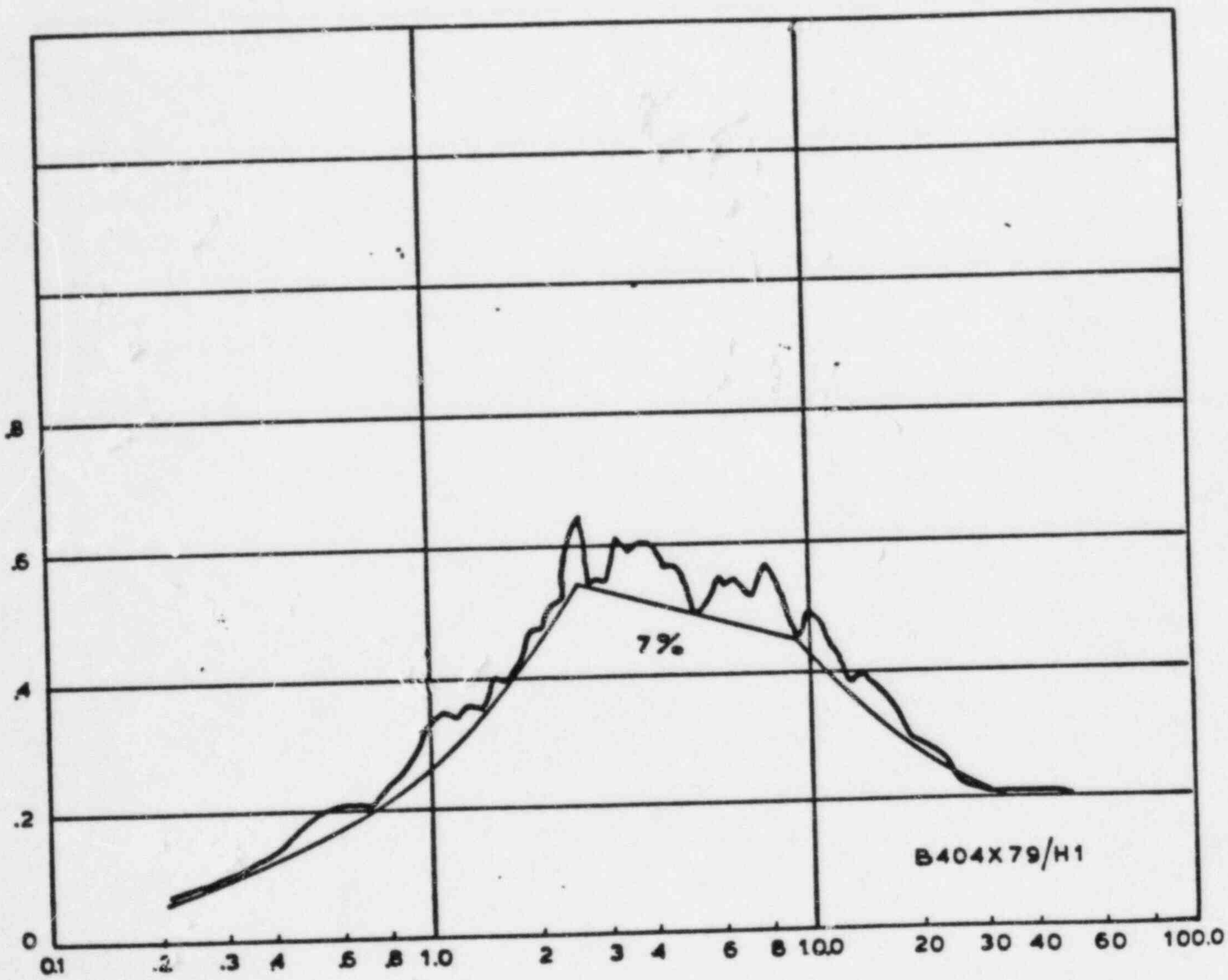
ACCELERATION SPECTRUM | ELEV = 623.00 DMPG = 7%



FREQUENCY (HZ)

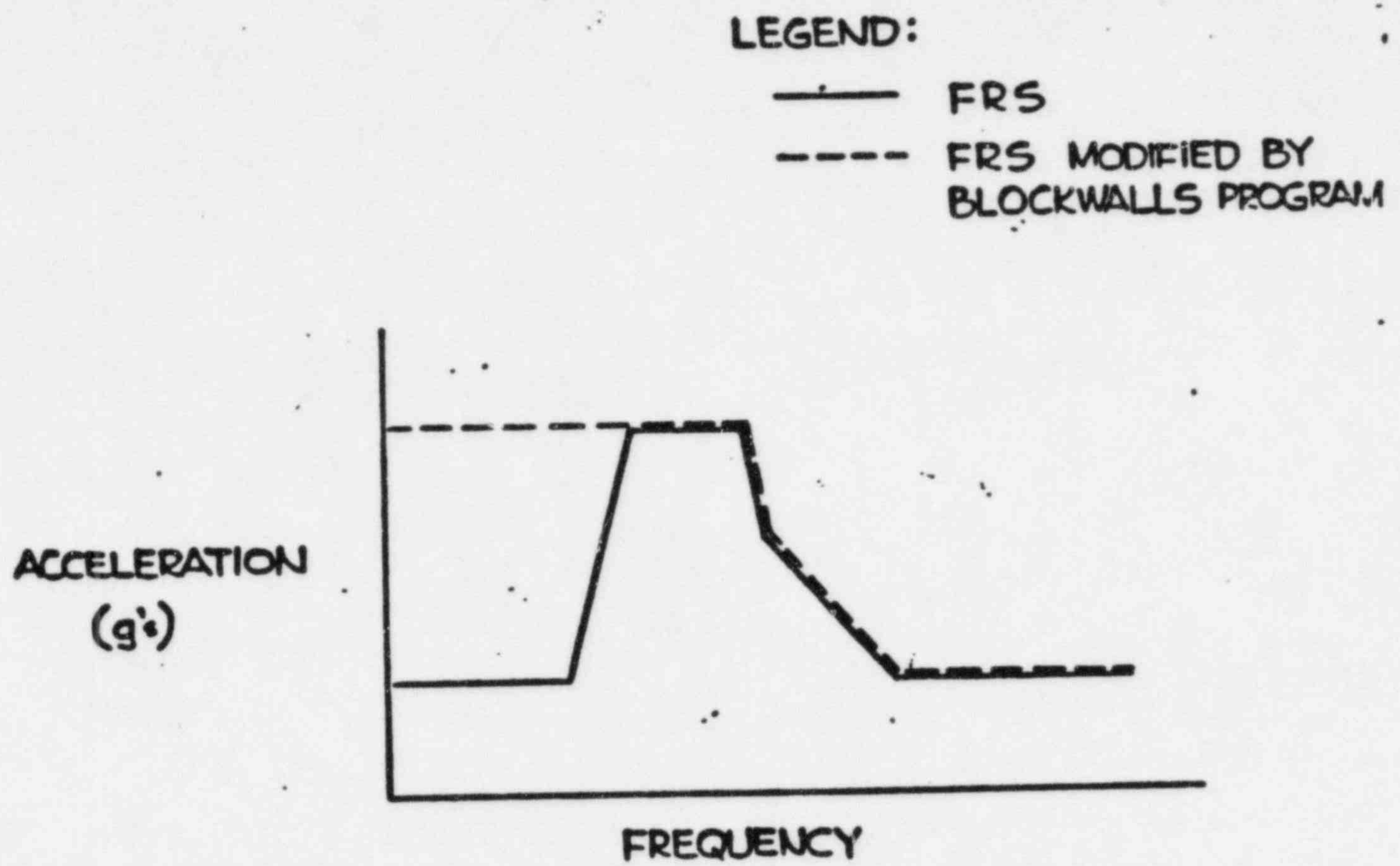
FIGURE 7

R. G. 1.60 TIME HISTORY



FREQUENCY (CPS)

FIGURE 8



REPRESENTATIVE FLOOR RESPONSE SPECTRA

FIGURE 9

WALL MODELING TECHNIQUES

BOUNDARY CONDITIONS

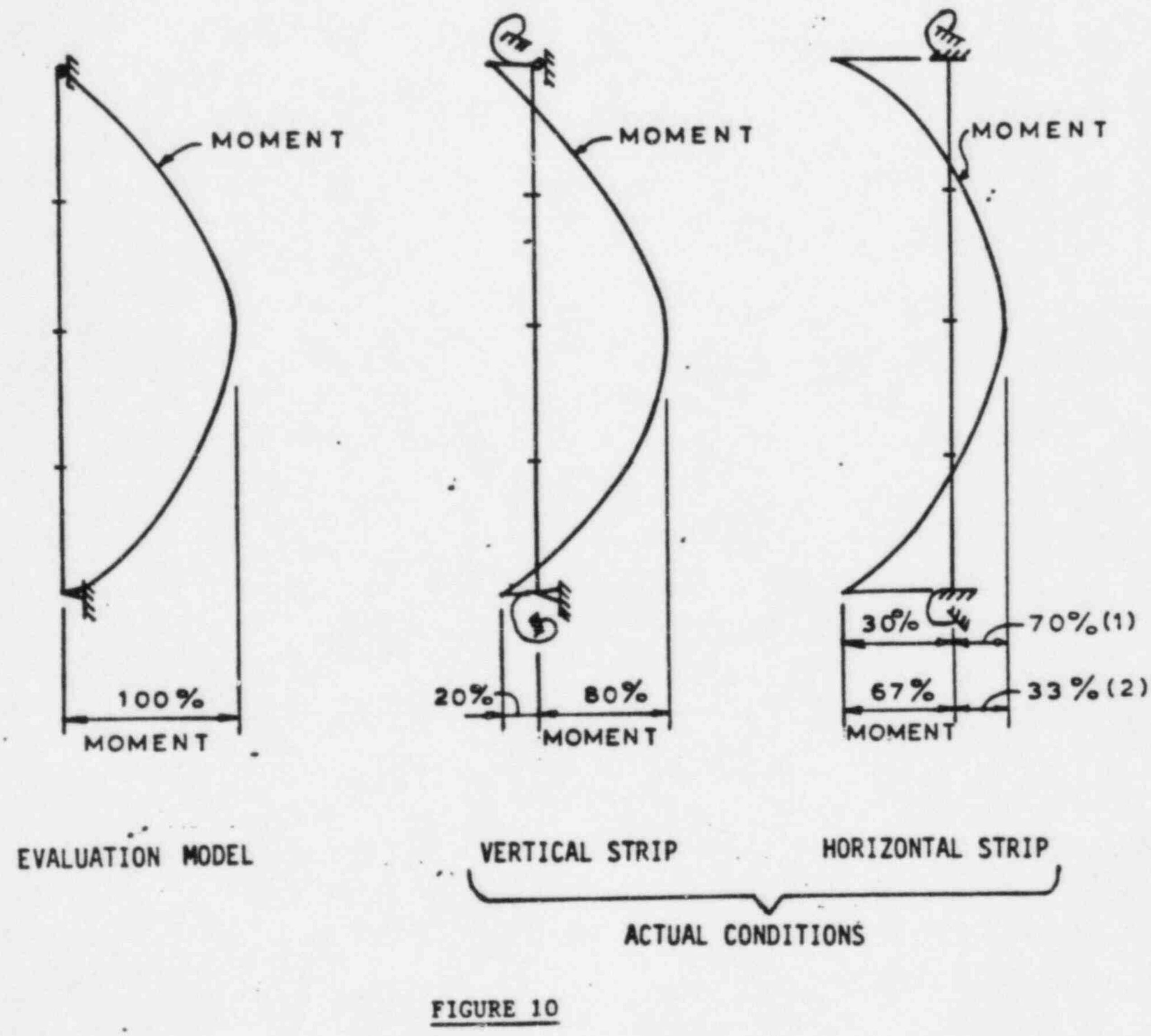
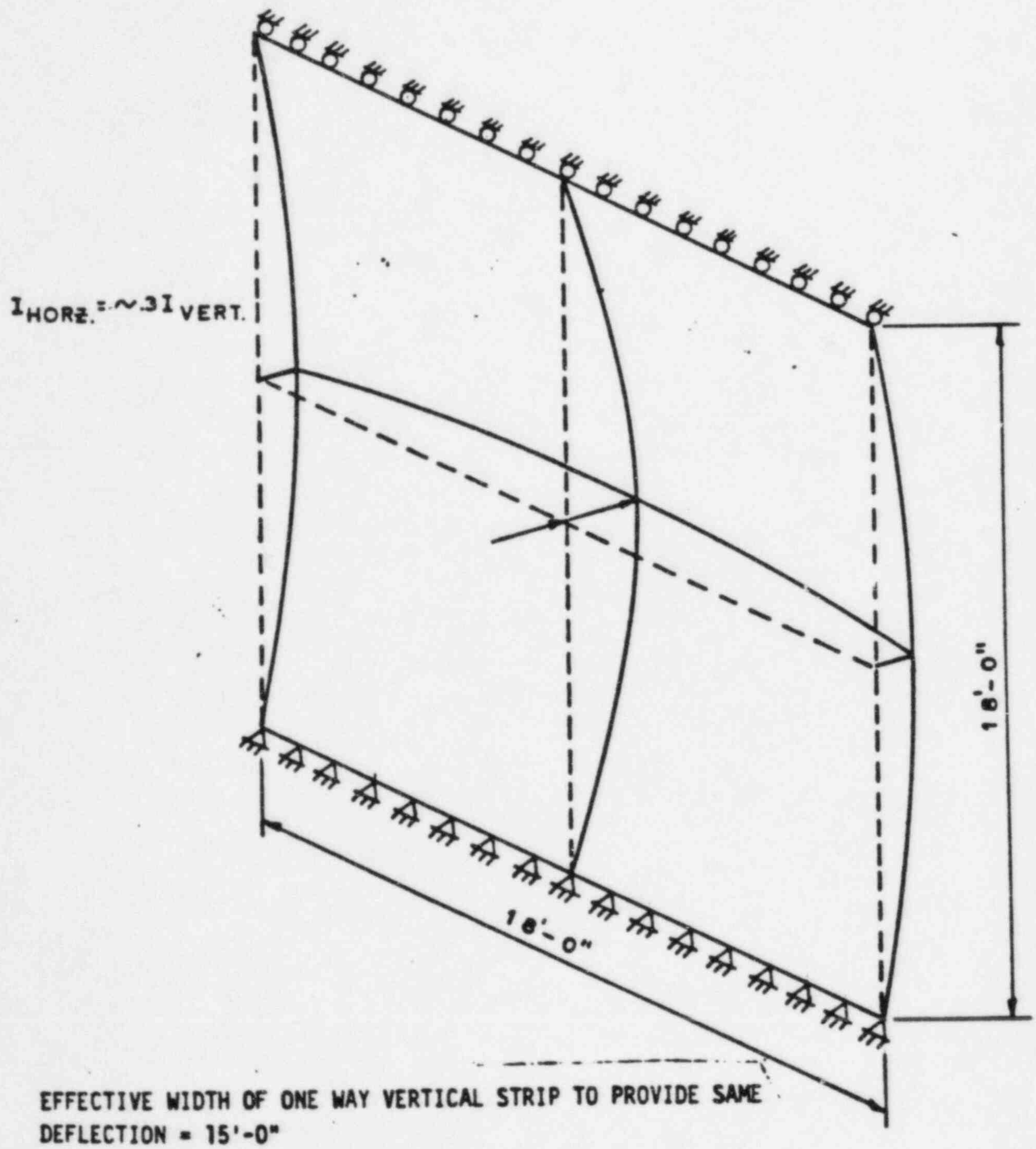


FIGURE 10

(1) Anchorage is two 3/8" Ø inserts with two 3/8" Ø all-thread rods lapped with each layer of Dur-o-wall.

(2) Anchorage is one "Z" type rigid steel masonry anchor lapped with each layer of Dur-o-wall.

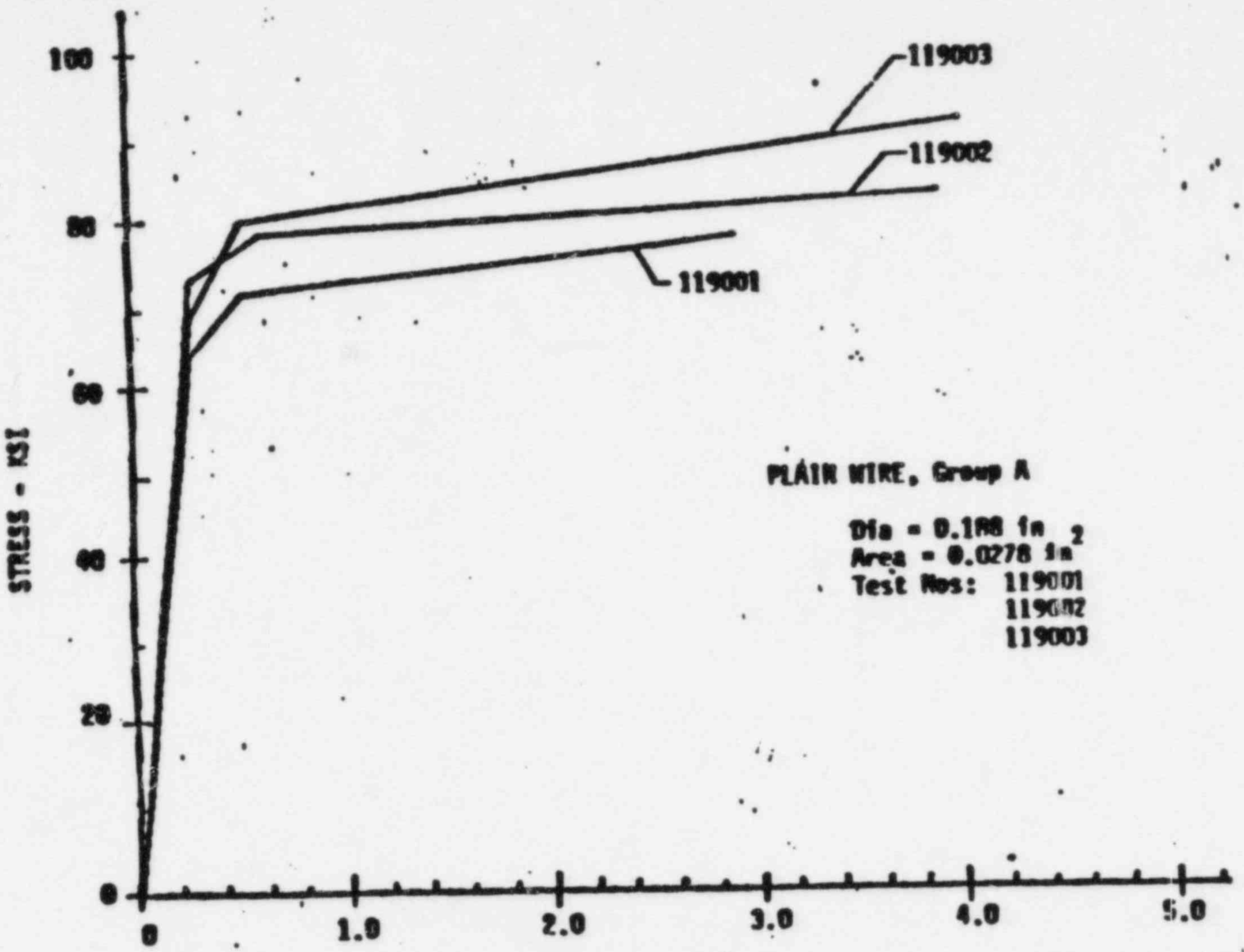
EFFECTIVE WIDTH OF VERTICAL STRIP



EFFECTIVE WIDTH OF ONE WAY VERTICAL STRIP TO PROVIDE SAME DEFLECTION = 15'-0"

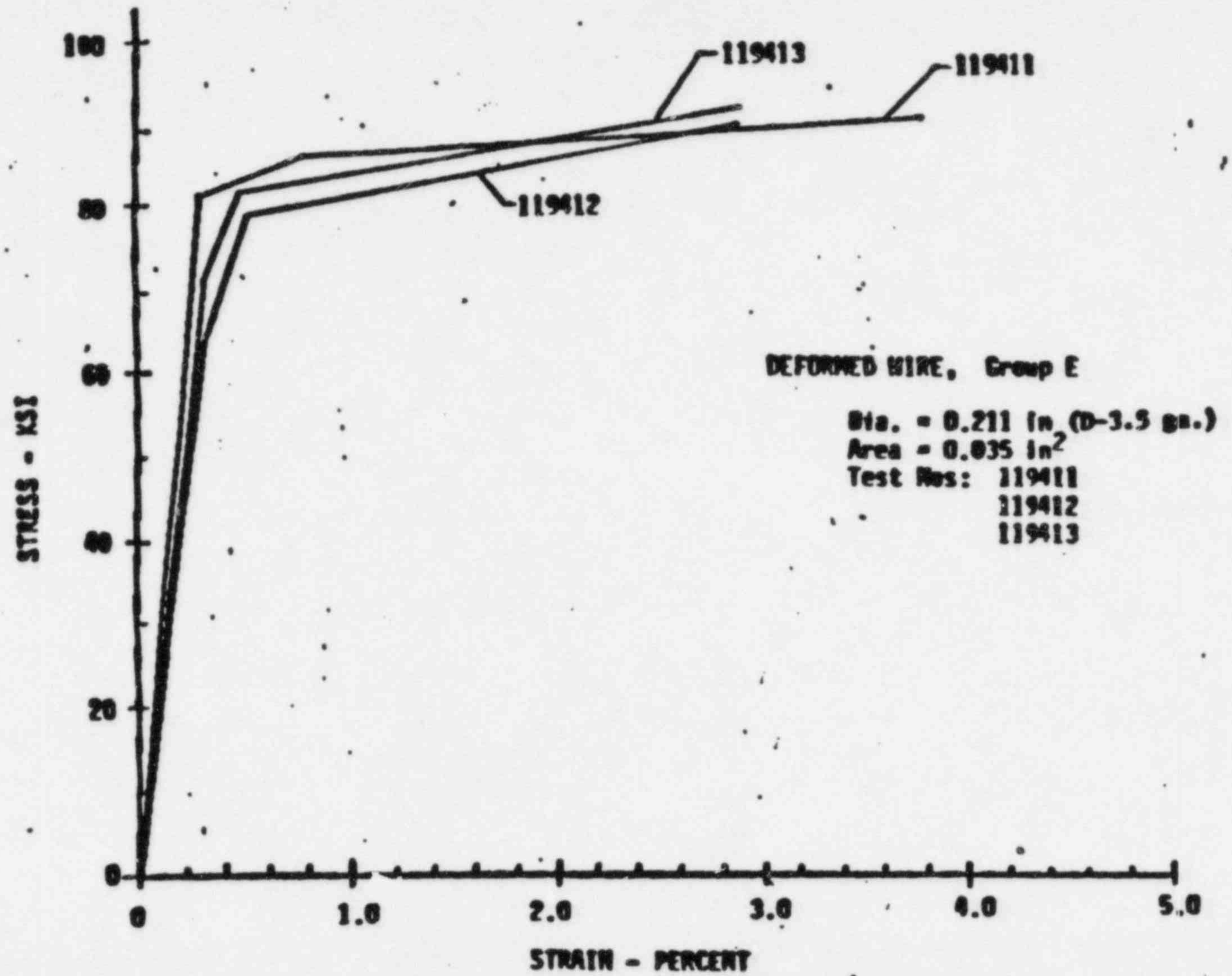
EFFECTIVE WIDTH OF ONE WAY VERTICAL STRIP TO PROVIDE SAME MOMENT = 12'-0"

FIGURE 11



Typical Stress - Strain Curves for Plain Wire,
(Reference 1)

FIGURE 12



Typical Stress - Strain Curves for Deformed Wires
(Reference 2)

FIGURE 13

CONDUIT SUPPORT LOADS

AVERAGE FILL PERCENTAGES

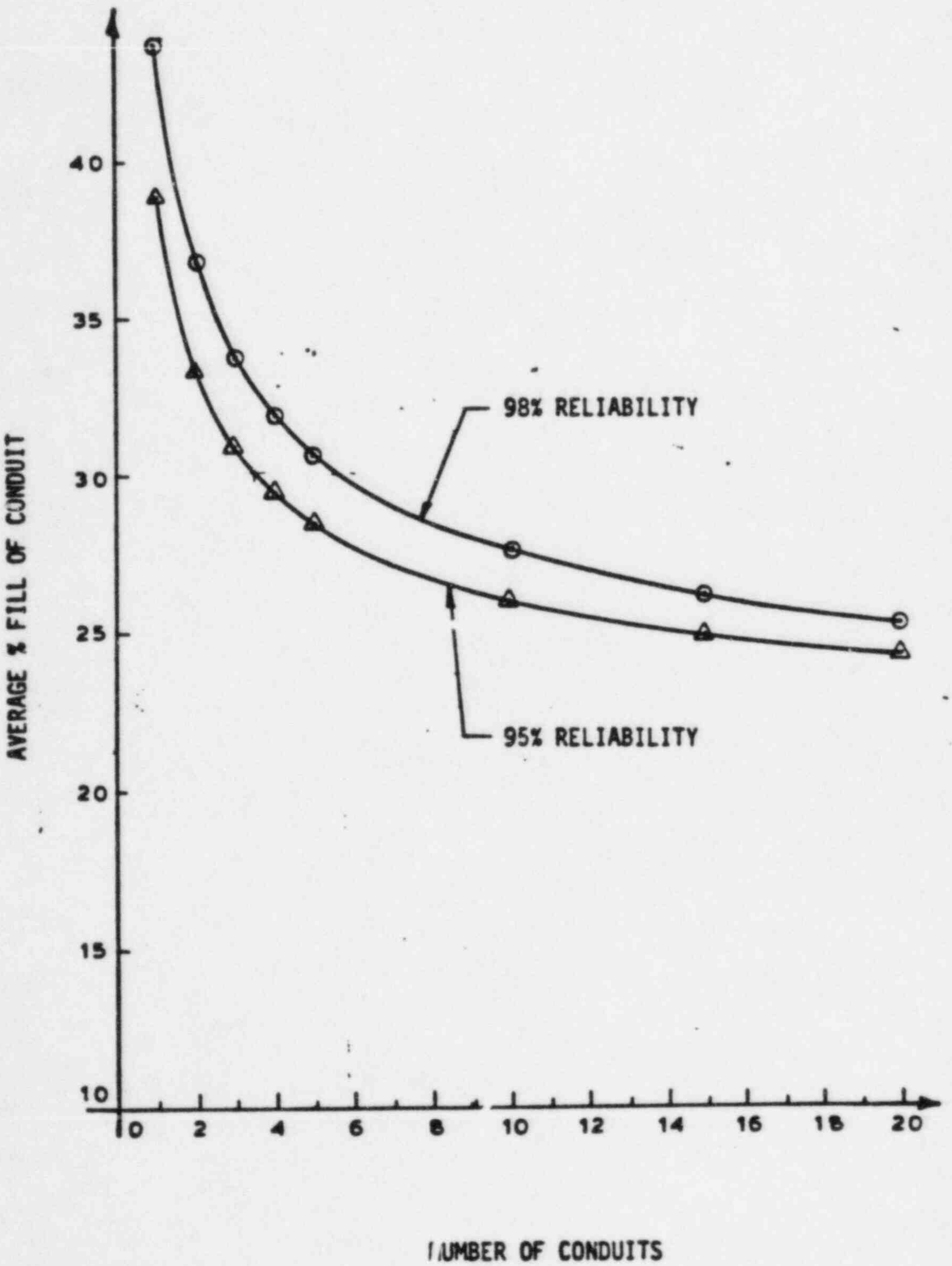
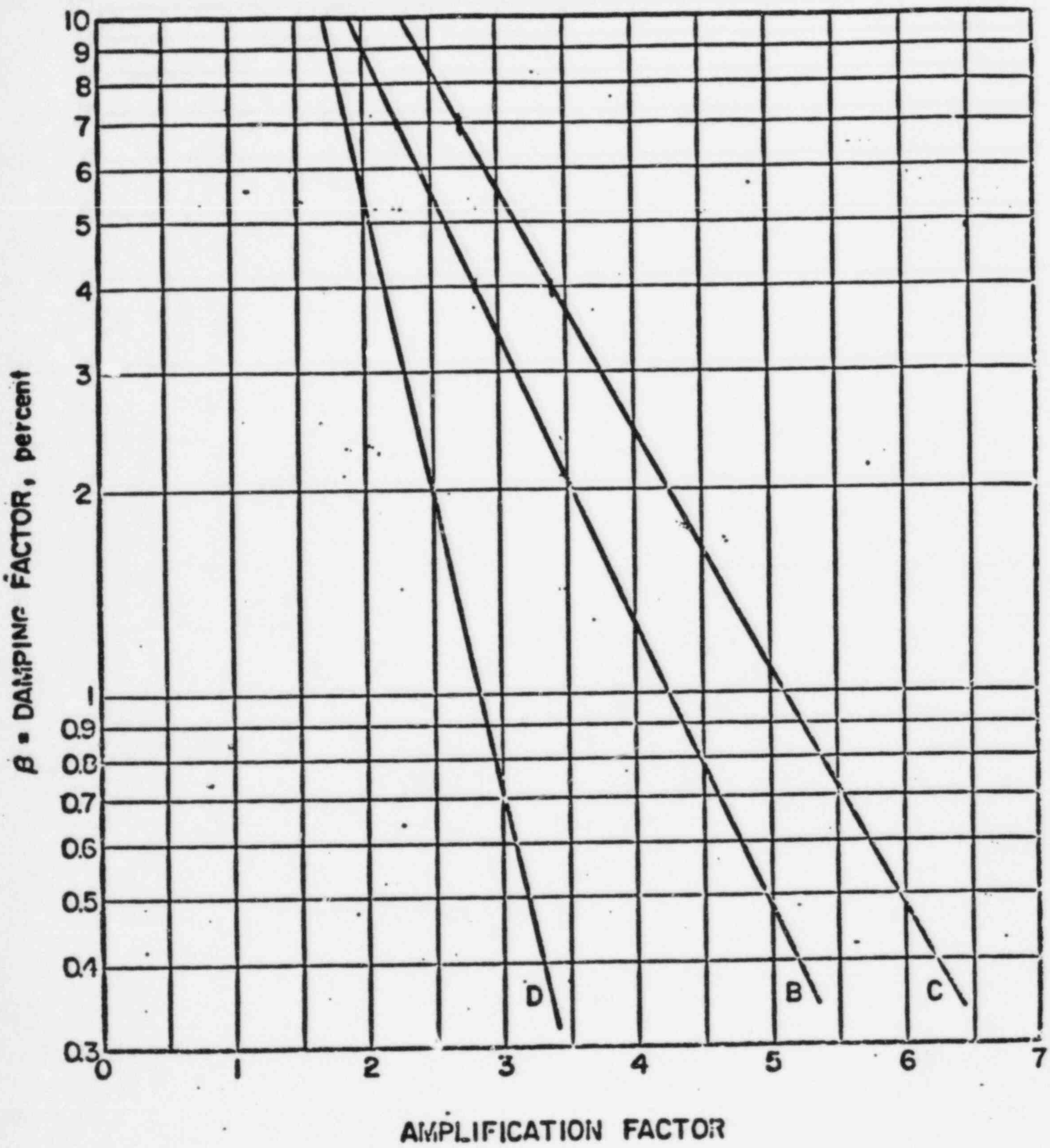


FIGURE 14



AMPLIFICATION FACTORS AT CONTROL FREQUENCIES FOR DESIGN SPECTRUM

FIGURE 15