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METHODOLOGY FOR MITIGATION OF SEISMIC HAZARDS IN EXISTING UNREINFORCED MASONRY BUILDINGS:

THE METHODOLOGY

ABK A Joint Venture

250 North Nash Street
El Segundo, California 90245

Topical Report 08
January 1984
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Prepared for
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This report presents a methodology for the mitigation of seismic hazards in existing unreinforced masonry (URM) buildings. Research undertaken to develop the methodology is described and limitations of the methodology are reported. Attention is focused on selection of the seismic risk zone, ~~on the degree of seismic hazard reduction,~~ and on design spectra for seismic hazard reduction. A procedure for field surveys of existing URM buildings is recommended and the response of existing structural systems to earthquakes is considered. Methodologies for mitigating seismic hazard in areas of design ground motion of effective peak acceleration of .1G, .2G, and .4G are presented. The philosophy used to develop recommended capacities of existing materials is discussed and the methodology is compared to current seismic reduction requirements. The methodology is shown to describe recommendations for seismic hazard mitigation by hazard zone rather than by means of a seismic zone reduction factor and the methodology does not recommend static force analysis methods for diaphragms.

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FOREWORD

This topical report is one of several reports prepared by ABK, A Joint Venture, for the National Science Foundation under Contract No. NSF-C-PFR-78-19200 and Grant No. CEE-8100532. The overall objective of the contract is to derive a methodology for the mitigation of seismic hazards in existing unreinforced masonry buildings. This research supports the objective of the Disaster and Natural Hazard Research being conducted under the Applied Science and Research Applications program of the National Science Foundation.

The Joint Venture ABK consists of the three firms, Agbabian Associates (AA), S.B. Barnes & Associates (SBB&A), and Kariotis & Associates (K&A), all in the Los Angeles area. The principal investigators for the three firms are R.D. Ewing for AA, A.W. Johnson for SBB&A, and J.C. Kariotis for K&A. The editor for the reports is J. Athey of AA.

This report presents a methodology for the mitigation of seismic hazards in existing unreinforced masonry (URM) buildings. This methodology is based on research that includes:

- Categorization of URM buildings
- Seismic input
- Dynamic testing of full-scale URM walls, out-of-plane
- Static and dynamic testing of full-scale diaphragms, in-plane
- Static and dynamic testing of URM walls, in-plane
- The performance of URM buildings in past earthquakes (e.g., Coalinga, Imperial Valley, Eureka, San Fernando)
- Analysis methods that have been correlated with the tests

Principal contributors to this report are J.C. Kariotis from K&A, R.D. Ewing from AA, and A.W. Johnson from SBB&A.

Dr. J.B. Scalzi served as Technical Director of this project for the National Science Foundation and maintained scientific and technical liaison with the joint venture throughout all phases of the research program. His contributions and support are greatly appreciated.

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EXECUTIVE SUMMARY

A multiyear investigation of unreinforced masonry (URM) construction was undertaken in order to provide a methodology for analysis/retrofit that mitigates seismic hazards for these structures. This research included: (1) categorization of URM buildings (ABK, 1981a), (2) seismic input (ABK, 1981b), (3) dynamic testing of full-scale URM walls, out-of-plane (ABK, 1981d, 1982b), (4) static and dynamic testing of full-scale diaphragms, in-plane (ABK, 1981c, 1982a), (5) static and dynamic testing of URM, in-plane (App. C, D), (6) performance of URM buildings in past earthquakes (e.g., Coalinga, Imperial Valley, Eureka, and San Fernando), and (7) analysis methods that have been correlated with the tests. This research has been detailed in seven previous reports. In this report, the methodology for mitigating seismic hazards in URM buildings is presented, and is adjusted for three seismic hazard zones as described by the ATC 3-06 provisional design guidelines (ATC, 1978 and ABK, 1981b). These seismic hazard zones are defined by Effective Peak Accelerations (EPA) of 0.1 g, 0.2 g, and 0.4 g respectively. A synopsis of the methodology for these three seismic hazard zones is presented in the remainder of this summary.

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SEISMIC HAZARD ZONE OF EPA = 0.1G
PROCEDURE FOR USE OF THE METHODOLOGY

1. FIELD SURVEY: SECTION 4

a. PREPARE PRELIMINARY FRAMING PLANS FOR ALL ROOFS AND FLOORS (Section 4.1).

SPECIAL INVESTIGATION:
Note all beams, trusses or major lintels that bear on URM piers or pilasters.

b. PREPARE PRELIMINARY ELEVATIONS OF ALL URM WALLS (Section 4.2).

SPECIAL INVESTIGATIONS:
Note all parts of the vertical load-carrying system that may act as ties to lateral load-resisting elements.

Sketch support systems for URM walls that are discontinuous to base of building.

Note on floor plans all walls that are continuous between floors or between floors and roof.

Sketch relationship of roof framing and ceiling framing.

Sketch on floor plans the description and extent of flooring materials. Note any opening through floor adjacent to URM wall. Verify continuity of flooring materials over entire floor.

Sketch on roof plan the extent of roof sheathing and roofing materials. Note whether roofing materials are directly laid on sheathing. Note any discontinuities in sheathing or roofing materials adjacent to URM walls.

c. INVESTIGATE ANCHORAGE OF URM WALLS (Section 4.3). Note on URM wall elevations and floor plans all wall anchorages. Record full description of anchors, their spacing, and their connection system to floors and roof.

SPECIAL INVESTIGATION:
Determine the configuration of the end of the anchor embedded in the URM. Expose approximately 2% of the embedded ends unless embedded anchor configuration is typical of geographical area.

d. INVESTIGATION OF EXISTING URM WALLS (Section 4.4). Note on preliminary wall elevations:

- wall thickness at all levels
- coursing of exterior wythes of masonry
- bonding of masonry wythes and veneers
- masonry materials
- lintel materials
- heights of parapets and cornices above existing or possible anchorage levels
- anchorage or bonding of terra cotta, cast stone or stone facing.

SPECIAL INVESTIGATIONS:
Observe quality of mortar: See Section 4.4.

Note areas of eroded mortar.

Note areas of deteriorating brick or stone.

Note all cracks in URM walls. Sketch on wall elevations. Determine probable cause of observed cracks.

e. TESTING OF EXISTING MATERIALS (Section 4.5). Determine if qualification testing for existing anchorage system is cost-effective. If cost-effective, plan test procedure in accordance with Section 4.5.4.

SPECIAL INVESTIGATION:
If low quality mortar areas are noted on preliminary wall elevations, conduct URM qualification tests in accordance with Section 4.5.6 or specify repointing in conformance with Section 4.5.6.

2. ANALYSIS PROCEDURE FOR MITIGATION OF SEISMIC HAZARDS: SECTION 6

Identify all hazardous building elements on preliminary framing plans, floor plans, and URM wall elevations. See Section 6.1.

Calculate recommended anchorage force at each floor above the building base and at the roof level. See Section 6.2.

Verify capacity of existing wall anchors in accordance with strength capacities of Section 9. Verify capacity of embedded ends of existing wall anchors in accordance with Sections 4.5.1 and 4.5.4.

Design retrofitted wall anchorage system in accordance with Section 10.4. Establish quality control testing procedure in accordance with Section 4.5.5.

Design bracing system for URM parapets and appendages extending above the roof anchorage level in accordance with Section 10.

3. SPECIAL ANALYSIS CONSIDERATIONS

If URM wall height-thickness ratios in excess of usual standards are discovered, building height-plan dimension ratio exceeds 3, and the building is founded in soft soils, then verify dynamic stability of these walls in accordance with Section 6.3.

If diaphragm discontinuities exist adjacent to a URM wall, design a horizontal beam to provide support wall anchorages at the discontinuity. Design member in accordance with Section 10.

If the building survey has determined that parts or portions of the vertical-load carrying system may act as a tie to a probable shear wall, and horizontal displacement of that part of the vertical load-carrying system relative to the shear wall will cause loss of bearing capacity, then design a supplemental tie system in accordance with Sections 6.5 and 10.

If the building survey has determined that major elements of the vertical load-carrying system are supported on masonry piers, review system in accordance with Section 6.6. If probability of significant relative rotation of beam on bearing surface exists, provide supplemental support system designed in accordance with Section 10.

SEISMIC HAZARD ZONE OF EPA = 0.2G
PROCEDURE FOR USE OF THE METHODOLOGY

1. FIELD SURVEY: SECTION 4

   a. PREPARE PRELIMINARY FRAMING PLANS FOR ALL ROOFS AND FLOORS (Section 4.1).

      SPECIAL INVESTIGATION:
      Note all beams, trusses or major lintels that bear on URM piers or pilasters.

   b. PREPARE PRELIMINARY ELEVATIONS OF ALL URM WALLS (Section 4.2). Note all openings in walls on elevations.

      SPECIAL INVESTIGATIONS:
      Note all part of the vertical load-carrying system that may act as ties to lateral load-resisting elements.

      Sketch support systems for URM walls that are discontinuous to base of building. Note all construction materials in support system.

      Note on floor plans all walls that are continuous between floors or between floors and roof.

      Sketch relationship of roof framing and ceiling framing.

      Sketch on floor plans the description and extent of flooring materials. Note any opening through floor adjacent to URM wall. Verify continuity of flooring materials over entire floor.

Sketch on roof plan the extent of roof sheathing and roofing materials. Note whether roofing materials are directly laid on sheathing. Note any discontinuities in sheathing or roofing materials adjacent to URM walls.

c. INVESTIGATE ANCHORAGE OF URM WALLS (Section 4.3). Note on URM wall elevations and floor plans all wall anchorages. Record full description of anchors, their spacing, and their connection system to floors and roof.

SPECIAL INVESTIGATION:
Determine the configuration of the end of the anchor embedded in the URM. Expose approximately 2% of the embedded ends unless embedded anchor configuration is typical of geographical area.

d. INVESTIGATION OF EXISTING URM WALLS (Section 4.4). Note on preliminary wall elevations:

- wall thickness at all levels
- coursing of exterior wythes of masonry
- bonding of masonry wythes and veneers
- masonry materials
- lintel materials
- heights of parapets and cornices above existing or possible anchorage levels.
- anchorage or bonding of terra cotta, cast stone or stone facing.

SPECIAL INVESTIGATIONS:
Observe quality of mortar: See Section 4.4.

Note areas of eroded mortar.

Note areas of deteriorating brick or stone.

Note all cracks in URM walls. Sketch on wall elevations. Determine probable cause of observed cracks.

e. TESTING OF EXISTING MATERIALS (Section 4.5). Determine if qualification testing for existing anchorage system is cost-effective. If cost-effective, plan test procedure in accordance with Section 4.5.4.

SPECIAL INVESTIGATION:
If low quality mortar areas are noted on preliminary wall elevations, conduct URM qualification tests in accordance with Section 4.5.6 or specify repointing in conformance with Section 4.5.6.

2. ANALYSIS PROCEDURE FOR MITIGATION OF SEISMIC HAZARDS: SECTION 7

Identify all hazardous building elements on preliminary framing plans, floor plans, and URM wall elevations. See Sections 6.1 and 7.1.

Calculate recommended anchorage force at each floor above the building base and at the roof level. See Section 7.2.

Verify capacity of existing wall anchors in accordance with strength capacities of Section 9. Verify capacity of embedded ends of existing wall anchors in accordance with Sections 4.5.1 and 4.5.4.

Design retrofitted wall anchorage system in accordance with Section 10.4. Establish quality control testing procedure in accordance with Section 4.5.5.

Design bracing system for URM parapets and appendages extending above the roof anchorage level in accordance with Section 10.

3. SPECIAL ANALYSIS CONSIDERATIONS

a. STABILITY OF ANCHORED URM WALL ELEMENTS (Section 7.3). If URM wall height-thickness ratios in excess of usual standards are discovered, building height-plan dimension ratio exceeds 3, and the building is founded in soft soils, then verify dynamic stability of these walls in accordance with Section 7.3. Verify stability of URM walls for out-of-plane motions by use of Figure 8-1.

b. ANALYSIS OF HORIZONTAL DISPLACEMENT CONTROL ELEMENTS (Section 7.4). Design diaphragm-shear wall connection at all diaphragm edges parallel to URM walls not continuous to base of building. Calculate shear transfer at diaphragm edge by recommendations of Section 7.4, using C_p (from Table 7-1) times tributary building weight. Maximum design shear is given in Table 9-1.

If diaphragm discontinuities exist adjacent to a URM wall, design a horizontal beam to provide support to wall anchorages at the discontinuity. Design member in accordance with Section 10.

c. ANALYSIS OF VERTICAL DISPLACEMENT CONTROL ELEMENTS (Section 7.5). If a single URM shear wall, with many openings, provides displacement control on an axis of analysis, calculate the resistance capacity of the wall in accordance with Section 8.7. Compare calculated capacity with analysis forces as recommended in Section 7.5. If additional resistance capacity is required, design retrofitted elements in accordance with Section 10.

d. INTERCONNECTION OF BUILDING ELEMENTS (Section 7.6). If the building survey has determined that parts or portions of the vertical-load carrying system may act as a tie to a probable shear wall, and horizontal displacement of that part of the vertical load-carrying system relative to the shear wall will cause loss of bearing capacity, then design a supplemental tie system in accordance with Sections 7.6 and 10.

e. REVIEW OF VERTICAL LOAD-CARRYING ELEMENTS (Section 7.7). If the building survey has determined that major elements of the vertical load-carrying system are supported on masonry piers, review system in accordance with Section 6.6. If probability of significant relative rotation of beam on bearing surface exists, provide supplemental support system designed in accordance with Section 10.

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SEISMIC HAZARD ZONE OF EPA = 0.4G

PROCEDURE FOR USE OF THE METHODOLOGY

1. FIELD SURVEY: SECTION 4

a. PREPARE PRELIMINARY FRAMING PLANS FOR ALL ROOFS AND FLOORS (Section 4.1).

SPECIAL INVESTIGATION:
Note all beams, trusses or major lintels that bear on URM piers or pilasters.

b. PREPARE PRELIMINARY ELEVATIONS OF ALL URM WALLS (Section 4.2). Note all openings in walls on elevations.

SPECIAL INVESTIGATIONS:
Note all parts of the vertical load-carrying system that may act as ties to lateral load-resisting elements.

Sketch support systems for URM walls that are discontinuous to base of building. Note all construction materials in support system.

Note on floor plans all walls that are continuous between floors or between floors and roof.

Sketch relationship of roof framing and ceiling framing.

Sketch on floor plans the description and extent of flooring materials. Note any opening through floor adjacent to URM wall. Verify continuity of flooring materials over entire floor.

Sketch on roof plan the extent of roof sheathing and roofing materials. Note whether roofing materials are directly laid on sheathing. Note any discontinuities in sheathing or roofing materials adjacent to URM walls.

c. INVESTIGATE ANCHORAGE OF URM WALLS (Section 4.3). Note on URM wall elevations and floor plans all wall anchorages. Record full description of anchors, their spacing, and their connection system to floors and roof.

SPECIAL INVESTIGATION:
Determine the configuration of the end of the anchor embedded in the URM. Expose approximately 2% of the embedded ends unless embedded anchor configuration is typical of geographical area.

d. INVESTIGATION OF EXISTING URM WALLS (Section 4.4). Note on preliminary wall elevations:

- wall thickness at all levels
- coursing of exterior wythes of masonry
- bonding of masonry wythes and veneers
- masonry materials
- lintel materials
- heights of parapets and cornices above existing or possible anchorage levels
- anchorage or bonding of terra cotta, cast stone or stone facing.

e. TESTING OF EXISTING MATERIALS (Section 4.5). Determine if qualification testing for existing anchorage system is cost-effective. If cost-effective, plan test procedure in accordance with Section 4.5.4.

SPECIAL INVESTIGATION:
Note areas of eroded mortar and repoint before qualification testing. See Section 4.5.6.

Test existing URM for qualification as shear wall and to determine v_a. See Section 4.5.6.

Note areas of deteriorating brick or stone.

Note all cracks in URM walls. Sketch on wall elevations. Determine probable cause of observed cracks.

2. ANALYSIS PROCEDURE FOR MITIGATION OF SEISMIC HAZARDS: SECTION 8

Identify all hazardous building elements on preliminary framing plans, floor plans, and URM wall elevations. See Sections 6.1, 7.1, and 8.1.

a. ANCHORAGE OF URM WALL ELEMENTS (Section 8.2). Calculate recommended anchorage force at each floor above the building base and at the roof level. Anchorage force = 1.0 times URM wall weight tributary. See Section 8.2.

If existing wall anchors are to be used as part of the wall anchorage system, verify capacity of the embedded ends of the existing wall anchors in accordance with Section 4.5.4. Determine allowable capacity of embedded ends of existing wall anchors in accordance with Section 4.5.1. Determine capacity of the existing anchors in accordance with Section 9.4.

Design retrofitted wall anchorage system in accordance with Section 10.4. Establish quality control testing procedure in accordance with Section 4.5.5.

Design bracing system for URM parapets and appendages extending above the roof anchorage level in accordance with Section 10.

b. STABILITY OF ANCHORED URM WALL ELEMENTS (Section 8.3). Determine height-thickness ratio of all URM walls. Compare calculated ratios with acceptable ratios given in Table 8-1. If height-thickness ratios are greater than specified for "all other buildings" but less than specified for "buildings with crosswalls..." verify that existing crosswalls occur in all stories and that spacing and capacity of crosswalls conform to the recommendations of Table 8-3. Buildings with diaphragms conforming to the requirements of Table 8-2 qualify as "buildings with crosswalls." Crosswalls conforming to the minimum requirements of Table 8-3 may be introduced into the building to increase the acceptable height-thickness ratio of URM walls, or URM walls that exceed the recommended height-thickness ratio may be braced by supplemental members designed in conformance with the requirements of Section 8.3. Design supplemental members in conformance with the requirements of Section 10.

c. COMPUTATION OF EARTHQUAKE RESPONSE FORCE (Section 8.4). Calculate weight of building as a lumped weight at each floor, mezzanine or roof level. Tabulate building weight as W_w and W_D.

Design diaphragm-shear wall connection. Calculate recommended shear capacity as W_D times C_p given in Table 8-4. However, shear connection capacity need not exceed $v_u \cdot D$. Recommended yield capacities of diaphragms are given in Table 9-1.

Calculate response shear as: $V = 0.4W_w + \sum_1^n 0.4V_D$, however $0.4V_D$ at any level need not exceed $v_u \cdot D$ of the diaphragm at that level.

Calculate restoring shear capacity as: $V_R = 0.2W_w + \sum_1^n 0.2V_D$, however $0.2V_D$ at any level need not exceed $v_u \cdot D$ of the diaphragm at that level.

d. DISTRIBUTION OF RESPONSE FORCES (Section 8.5). For buildings with URM shear walls that exceed the height/length ratios used for response studies and that are founded on soft soils (Appendix A), redistribute the response forces in accordance with the recommendations of the reference, ATC 1978, using formulas (4-6) and (4-6a):

$$F_x = C_{vx} V \qquad (4\text{-}6)$$

$$C = \frac{w_x h_x}{\sum_{i=1}^{n} w_i h_i} \qquad (4\text{-}6a)$$

w_i, w_x = the portion of W_D or W_w located at or assigned to level i or x

h_i, h_x = the height above the base to level i or x.

e. ANALYSIS OF HORIZONTAL DISPLACEMENT CONTROL ELEMENTS (Section 8.6). For diaphragms without crosswalls, calculate diaphragm demand/capacity ratio as:

$$\frac{W_D}{2v_u \cdot D}$$

W_D = total weight tributary to diaphragm

v_u = yield capacity of diaphragm given in Table 9-1

D = diaphragm depth. Use D_1 if opening in diaphragm occurs within depth of diaphragm as measured from the shear wall.

Check diaphragm span limitation as recommended by Figure 8-2. If existing diaphragm span exceeds the recommendation of Figure 8-2, retrofit the diaphragm to increase v_u or design crosswalls to limit relative diaphragm displacement. Calculate diaphragm demand/capacity ratio as:

$$\frac{W_D}{2v_u D + \sum V_c}$$

Recheck diaphragm span limitation as recommended by Figure 8-2.

For diaphragm analysis for "open-front" buildings, calculate an equivalent diaphragm span L_1 as follows:

$$L_1 = 2 \left(\frac{W_w \cdot L}{W_D} + L \right)$$

L = length of diaphragm measured from the front of the building to the nearest shear wall

W_w = total weight of the wall above the open front tributary to the diaphragm

W_D = total weight tributary to the diaphragm

Calculate diaphragm demand/capacity ratio as:

$$\frac{W_D + W_w}{v_u \cdot D} \quad \text{or} \quad \frac{W_D + W_w}{v_u \cdot D + \sum V_c}$$

Check diaphragm span limitations as recommended by Figure 8-2. Acceptable span length as determined by diaphragm demand/capacity ratio must exceed L_1.

f. ANALYSIS OF VERTICAL DISPLACEMENT CONTROL ELEMENTS (Section 8.7). For all URM shear walls that are divided into piers by door and window openings, calculate restoring shear capacity of the pier systems as:

$$V_R = \sum_1^n 0.9 \frac{P_x D_x}{H_x}$$

P_x = axial load on pier
D_x = in-plane depth of pier
H_x = least height of pier if opening height on sides of pier varies

Compare calculated restoring shear capacity with minimum recommended restoring shear: $V_R = 0.2W_w + \sum_1^n 0.2W_D$

Compare calculated V_R on each pier with in-plane shear capacity V_a. Calculate V_a as:

$$V_a = \frac{v_a A}{1.5}$$

v_a = allowable shear
A = gross area of pier

The allowable URM shear v_a is calculated as:

$$v_a = 3/4\ (3/4 v_t + P/A)$$

v_t = 20th percentile of in-plane test shear values reduced to equivalent shear at zero axial stress
P = axial load on pier
A = gross area of pier

If $V_R < 0.2W_w + \sum_1^n 0.2V_D$, and for all piers $V_R < V_a$, supplement restoring shear by any materials designed in accordance with Section 10.

If for any pier, $V_R > V_a$, in-plane shear failure is probable and piers must be analyzed for shear capacity.

- Distribute response shear V to pier system using stiffness as D/H.
- Calculate $v = 1.5\ V/A$ for stiffest pier.
- If $v > v_a$, increase shear capacity of URM wall with consideration of relative stiffness of existing and new materials.
- For walls without openings and with height/length ratio of 0.5 or less, calculate $v = V/A$. See Appendix D, Section D-3.1.

g. INTERCONNECTION OF BUILDING ELEMENTS (Section 8.8). A continuous load path for all calculated response forces should be provided. However, interconnection capacity of existing materials described in Section 9 need not be analyzed.

- Design tie system parallel to shear wall for distribution of calculated response forces.
- Design diaphragm distribution tie system for retrofitted crosswalls or shear walls.

h. REVIEW OF VERTICAL LOAD-CARRYING ELEMENTS (Section 8.9). If the building survey has determined that major elements of the vertical load-carrying system are supported on masonry piers, provide independent structural steel columns or equivalent at the face of the masonry pier. An independent foundation system is not required.

EXCEPTION: If a shear wall is retrofitted into the line of bearing masonry piers, such as in the plane of an open front or a URM wall not continuous to the building base, independent support columns are not required.

SPECIAL INVESTIGATION:
If a concrete frame without ductile detailing provides support for URM walls not continuous to the base of the building, an in-plane shear wall should be designed in accordance with the recommendations of Section 10. A special analysis of the retrofitted shear wall should be made to insure that the non-ductile system remains elastic during yield excursions of the shear wall.

SECTION 1

INTRODUCTION

1.1 BACKGROUND

Building construction using unreinforced masonry (URM) predates the development of seismic criteria that guide the design and construction of present-day buildings. A substantial number of these URM buildings are still being used in areas considered seismically active, even though investigations of earthquake damage have confirmed that this type of building has been a major contributor to personal injury and loss of life during relatively high intensity earthquakes. Yet the cost of rehabilitating existing URM buildings to standards required for new construction is usually unacceptable. Also, URM construction is still being used in some parts of the United States that have experienced lesser intensity earthquakes.

Public agencies and the private sector are becoming more concerned about the potential for personal injury or death resulting from failure of these buildings. However, political jurisdictions struggling with limited budgets can rarely afford the extensive research programs required to develop rehabilitation standards. It is apparent that a system of analysis methods and procedures - a methodology - is needed for determining realistic hazard-mitigation requirements and cost-effective methods of retrofit to fill such requirements. Research that can provide usable tools to meet these goals will have a major impact on cities squeezed between economic constraints and threats to life safety. The developed methodology and standards could reduce the enormous investment now required to make existing buildings conform to standards for new construction, and eliminate the economic loss that would result from demolition of these buildings.

In 1977, the National Science Foundation (NSF) initiated a multiphased program for the mitigation of seismic hazards, which resulted in a study to develop a methodology for the mitigation of seismic hazards in existing URM buildings. A program plan for this study was based on existing research, observed damage in past earthquakes, and an assessment of the response of typical URM buildings. A review of existing research work on masonry, available at that time, showed that most of the effort had been directed toward determining the response of reinforced masonry components to in-plane forces;

and little or no effort had been devoted to typical URM building response. Reports of observed damage in past earthquakes indicated degrees of damage that varied from minor cracking of URM walls to separation of the walls from the diaphragm and, in some instances, the subsequent collapse of the URM walls. A key observation taken from these damage reports is that some structures sustained more damage than others, and the researchers were led to the assumption that the interaction among the building components was a vital issue in explaining and predicting URM building damage. Accordingly, a study of typical URM building seismic response was conducted and three related component responses and their interactions were identified for further study; namely:

- Horizontal diaphragms subjected to in-plane motions
- URM walls subjected to out-of-plane motions
- Anchorage between the URM walls and diaphragm

Moreover, a review of the existing research work showed that the first two items, the response of diaphragms to in-plane motions and the response of URM walls subjected to out-of-plane motions, have received little or no attention. As part of the overall study to develop the methodology, analytical and experimental investigations were conducted on these two items. From data obtained by the research into the response and dynamic behavior of the diaphragms and URM walls, additional studies and experimental work were planned. The topics of these studies were:

- Seismic Response Model of a Rigid Block on Soils
- Seismic Response Model of Diaphragms with Crosswalls
- In-Plane Testing of Existing URM Masonry
- Finite Element Studies of URM Wall Piers

The results of these studies are reported in the appendix of this methodology. Topical reports are presented in the following documents (see "Reference" section) and form the basis of this methodology:

- ABK-TR-01 Categorization of Buildings
- ABK-TR-02 Seismic Input
- ABK-TR-03 Diaphragm Testing
- ABK-TR-04 Wall Testing, Out-of-Plane

- ABK-TR-05 Interpretation of Diaphragm Tests
- ABK-TR-06 Interpretation of Wall Tests, Out-of-Plane
- ABK-TR-07 Anchorage

The scope of the methodology encompasses the range of seismic hazard (ABK, 1981b) that exists in the United States and the range of URM buildings reported in Categorization of Buildings (ABK, 1981a).

1.2 PHILOSOPHY

Life-safety in the event of ground shaking is the paramount consideration of this methodology. Mitigation of life-safety threats in existing URM buildings is provided by minimizing the probability of the separation of the URM walls and parapets from the roof and floors and collapse of the gravity load-carrying system.

The first goal can be attained by retrofitting anchorage systems; the second goal is attained by analysis of the existing structural systems to determine the need for retrofit systems.

Mitigation of life-safety threats caused by seismic ground motions is generally related to the limitation of property damage. Use of this methodology provides that benefit, but it is not a primary consideration. The methodology uses the concept of a design earthquake as an entry to analysis methods. It is recognized, however, that because of the random and unpredictable nature of earthquake motions, the uncertainties of the response of URM buildings to earthquake motions, and the determination of undesigned material resistance capacities, even a relatively complete methodology cannot ensure that there will be no loss of life.

1.3 OBJECTIVES

To understand the project goals of this methodology, the objectives are listed below:

- Evaluate the past performance of URM buildings in earthquakes from knowledge gained by on-site observations.
- Categorize URM buildings to correlate observed earthquake damage and predicted behavior of building elements.

- Provide recommendations for determining the resistance capacity of existing building materials.
- Provide guidance to governmental bodies charged with setting public policy for reduction of risk caused by seismic hazards.
- Provide seismic hazard mitigation methodologies applicable to all probable earthquake areas of the United States.
- Provide a methodology usable by a knowledgeable design professional to evaluate the magnitude of risk.
- Provide a methodology to enable a design professional to retrofit an existing URM building to obtain the desired hazard reduction.
- Provide a commentary in conjunction with the methodology to assist the user in understanding the intent of the methodology.

1.4 NEW CONCEPTS

The methodology incorporates several new concepts that are significant departures from existing seismic design recommendations and provisions. These new concepts were introduced into the methodology in recognition that this document is primarily describing an analysis procedure. Hazard reduction evaluation of an existing building must recognize the reasonable means available and the economic impact of retrofit decisions based on the evaluation. Cost-effectiveness of seismic hazard mitigation recommendations must be enhanced by refining the seismic response model and determining the dynamic behavior of building elements. Consequently, the methodology incorporates the new concepts listed below:

- Input ground motions for earthquake hazard zones are taken from the updated - although still tentative - seismic design provisions of the Applied Technology Council (ATC, 1978).
- The seismic response model for the URM buildings is modeled as a rigid block on flexible soils. This basic response model is modified for URM walls with a limited story shear capacity.
- Determination of response and relative displacement control capacities of horizontal diaphragms is based on tested dynamic behavior in lieu of static analysis criteria.

- Dynamic stability concepts for URM wall elements are utilized in lieu of requirements for an elastic resistance capacity that is based on a prescribed static horizontal force.
- The recommendations of the methodology are described separately for seismic hazard zones in lieu of the use of a factored coefficient for correlation with seismic hazard zones.
- Materials resistance capacities are based on displacements equivalent to limits of elastic behavior. Inelastic behavior or equivalent behavior of materials is utilized in these recommendations.
- All existing materials and elements in the URM building that are distorted by relative horizontal or interstory displacement are considered in the response model and the structural resistance model.

1.5 LIMITATIONS ON APPLICATION OF THE METHODOLOGY

The methodology has been developed for seismic hazard mitigation for a broad range of URM buildings. The majority of these URM buildings use the URM walls as bearing and enclosure walls at the building perimeter. The URM buildings categorized in a nationwide survey (ABK, 1981a) have this general characteristic. Many buildings use URM as enclosure walls at the perimeter either infilled in the structural frames or attached to the exterior of the building framing.

If the URM is solidly infilled in a structural frame, the recommendations of the methodology are not applicable. Dynamic stability of this wall is provided by structural restraint at the top and bottom of the wall, not by dynamic stability principles. The in-plane strength of the solidly infilled wall is similar to a URM pier that has shear oriented failure modes, but post-cracking displacement along the crack is limited by the structural framing. Application of facets of the methodology is possible, but each analysis procedure must be modified for these special considerations.

If the URM walls are attached to the perimeter of a framed building, then application of the wall anchorage requirements of the methodology should be considered. A reanalysis of transmission of ground motions to the diaphragm should be made. When the modified ground motion to the edge of the diaphragm is determined, appropriate recommendations for wall-anchorage can be selected. Use of this methodology is not appropriate for analysis of a building framed by a system that does not use URM except as enclosure walls.

The recommendations for dynamic stability for URM walls when shaken out-of-plane are not appropriate for free standing walls or walls that are not either anchored to framing at both ends or continuous to the building base.

SECTION 2

GUIDE TO USE OF THE METHODOLOGY

2.1 SELECTION OF SEISMIC RISK ZONE

The methodology utilizes a description of intensity of ground shaking termed Effective Peak Acceleration (EPA) as input to the response model. EPA is utilized as a descriptor of a spectral shape (ABK, 1981b). This procedure follows the direction of the Applied Technology Council (ATC). In a document titled "Tentative Provisions for the Development of Seismic Regulations for Buildings, ATC 3-06," contour maps of EPA were prepared (ATC, 1978). Use of these maps (Figs. 2-1 and 2-2) is recommended.

For determination of the intensity of sites located between the contours, interpolation between the contours is recommended. Reference sources described in the commentary to the provisions of the ATC document may also be used. Determination of the probability of the intensity of ground shaking by special studies is an alternate method. The procedure for these studies should parallel the ATC methodology to provide a uniform method of establishing seismic hazard.

Seismic zone maps such as those published in the Uniform Building Code, 1982 or earlier editions, or American National Standards Institute (ANSI) A58.1-1982 were prepared with the goal of defining a design parameter for new construction. The goals of design requirements for new construction are not identical to the objectives of seismic hazard mitigation, as previously described. Therefore, it is recommended that the ATC maps be used when using this methodology.

The development of the methodology included estimates of the probability of building and element response, both as-existing and as-modified, in the preparation of the specific hazard mitigation recommendations. The representative ground motions that were selected for use in the research were based on fitting scaled spectra from recorded data to smoothed spectral shapes (ABK, 1981b). This procedure averages, rather than bounds, the selected input motions. From this average input, the median response of URM building elements was determined. Probability of the prediction of response and resistance capacity of each element was considered for each URM building element (ABK, 1982a; 1982b). The in-plane URM wall shakes the ends of the diaphragm.

The diaphragm shakes the out-of-plane walls and generates the wall anchorage forces. A chain of response is developed by element analysis and has an uncertainty introduced at each step. Median response and median member capacity must be considered to avoid compounding of uncertainty multipliers. The methodology introduces a single uncertainty factor in the recommendations. This uncertainty factor was developed utilizing the judgment of the authors.

References EERL 77-06 and EERI 1982 are suggested for additional commentary. These documents describe the relationship of ground motion data, its probability and recurrence interval, to the total body of data that is required for development of seismic hazard mitigation recommendations.

2.2 SELECTION OF DEGREE OF SEISMIC HAZARD REDUCTION DESIRED

The methodology provides recommendations for seismic hazard mitigation based on a probable intensity of ground motion called EPA. Section 2.1 provides guidance in determining EPA for all areas within the United States. It is recommended that this procedure be used unless a general risk-benefit analysis is made.

For guidance in planning a risk-benefit analysis, it should be recognized that a substantial number of existing URM buildings have survived moderate to strong ground shaking with moderate to minimal property damage. These URM buildings shaken by recorded, or estimated, ground motions have survived without implementation of a planned seismic hazard mitigation program. However, when ground shaking is described as strong, the URM survivors generally have a common structural element. This structural element is the traditional or supplemental anchors which attach the URM walls to the roof and floor framing.

References on earthquake design criteria (see ATC, 1978; EERI, 1982; and EERL, 1977) describe the probability of occurrence of ground shaking intensity of less than design level. For correlation with the recommendations of this methodology, design intensity ground shaking (EPA) is selected by use of the contour lines of Figures 2-1 and 2-2. Figure 2-3 from ATC 3-06 (ATC, 1978) indicates the annual risk of EPA ground shaking occurring at locations on the indicated contours of the recommended maps (Figs. 2-1 and 2-2). For areas within the EPA 0.4 g contour, the annual risk of ground shaking of EPA = 0.2 g is on the order of ten times the annual risk of design level, 0.4 EPA, ground shaking.

From this data (Fig. 2-3) and the observed performance of URM buildings shaken by earthquakes, the relationship of life-safety risks and hazard mitigation recommendations can be generally defined. Separation of parts of the URM walls has a moderate probability of occurrence for ground shaking intensity of less than design level. Ground motions of intensity less than design level have a significantly larger annual probability of occurrence. For this reason, an effective seismic hazard mitigation procedure will always include a wall-anchorage recommendation. In areas of design ground motion of EPA equal to 0.1 or 0.2 g, this wall-anchorage recommendation will comprise the major part of the seismic hazard mitigation program. The probabilities of the occurrence of significant damage to other elements of URM buildings is very small in these hazard zones. The methodology will address the special cases of URM construction that may alter this probability. Hazard mitigation recommendations, other than an effective wall-anchorage program, have a diminishing return. The cost-benefit ratio is highest for URM wall anchorage, and an immediate reduction in annual earthquake risk is obtained by a URM wall-anchorage program.

The risk-benefit analysis should examine the cost-benefit derived by application of the recommendations for EPA zones of less than design level. Mandatory earthquake hazard reduction ordinances, now in effect in California, allow the building owner to either fully comply with the ordinance within a limited time period, or to immediately retrofit anchorage devices, postponing the complete retrofit that is appropriate to the seismic hazard zone for a specified time period. A similar program or, if necessary, a mandatory program to obtain seismic hazard mitigation benefits that conform to the more significant annual risk, may be undertaken. Full compliance with the recommendations may then be required by the regulatory agency when adaptive reuse or occupancy change of the building is instigated by the building owner.

2.3 SELECTION OF DESIGN SPECTRA FOR SEISMIC HAZARD REDUCTION

The methodology recommends that an EPA for a hazard mitigation program be selected from Figures 2-1 and 2-2. The EPA selected establishes a standard spectra (ABK, 1981b) that defines an input ground velocity. These input velocities were used for large-scale dynamic testing (ABK, 1981c; 1981d). The spectral shapes used to define acceleration, velocity, and displacement are similar to those described in ATC 3-06 (ATC, 1978) for design spectras anticipated for firm soil sites. The methodology does not recommend that spectral shapes be altered in the velocity region by use of a soil factor. It is recognized that alteration of the region of the design spectra for soils influence is common in current seismic design requirements, but observed property damage of URM buildings on soft soils does not confirm a velocity amplification at the building base. This is due to the fact that URM buildings typically have a significantly larger weight than currently designed buildings. Computation of the weight of existing URM buildings indicates that these buildings weigh about 2-1/2 times the weight of currently constructed masonry buildings. Attenuation at the base of the building of recorded free-field ground motions by the softer and less competent soils is probable. The observed behavior in many earthquakes and consideration of a probable modification of a free-field motion base provide the substantiation for the recommended use of a standard spectral shape.

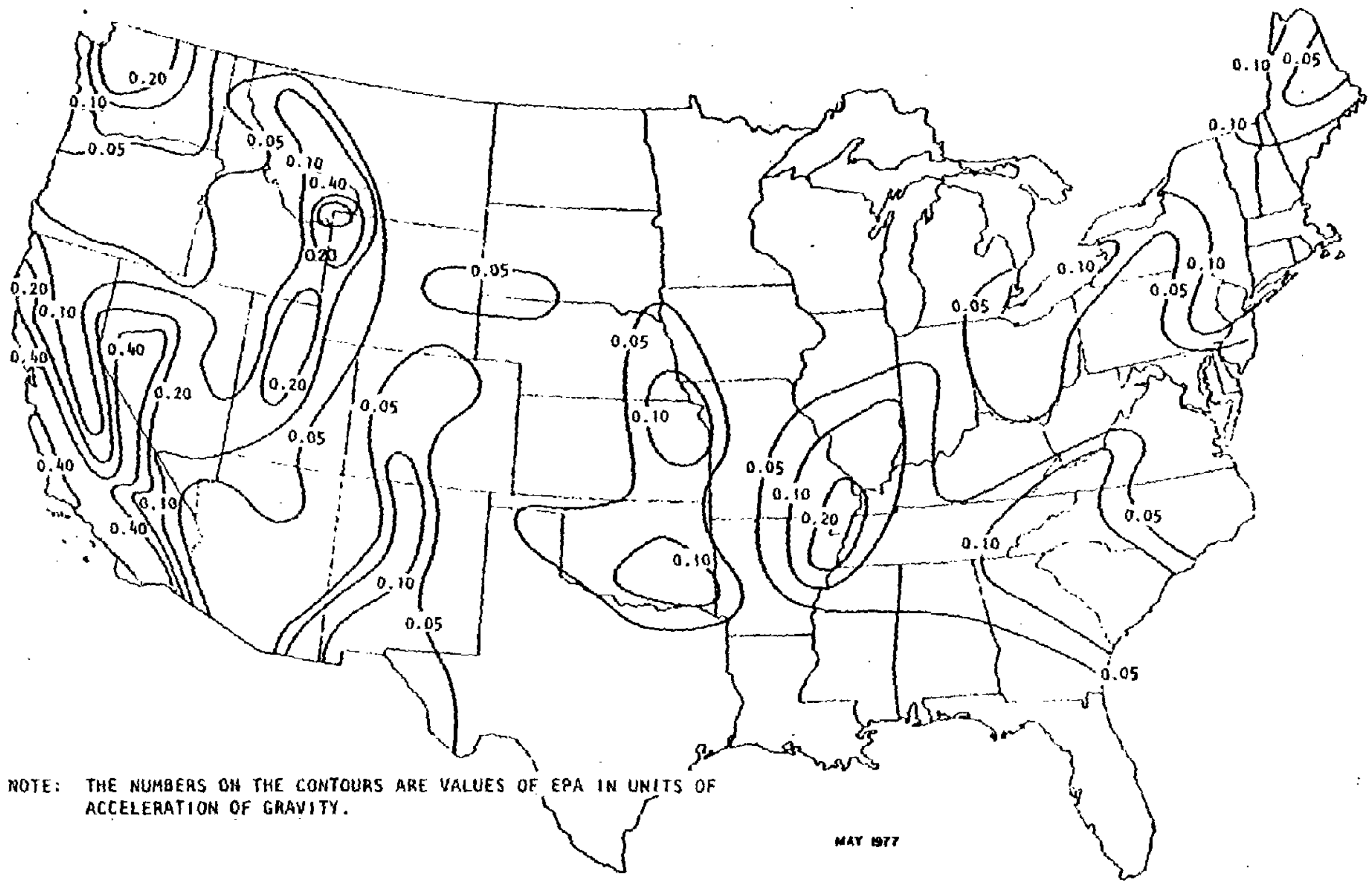


FIGURE 2-1. CONTOUR MAP FOR EFFECTIVE PEAK ACCELERATION

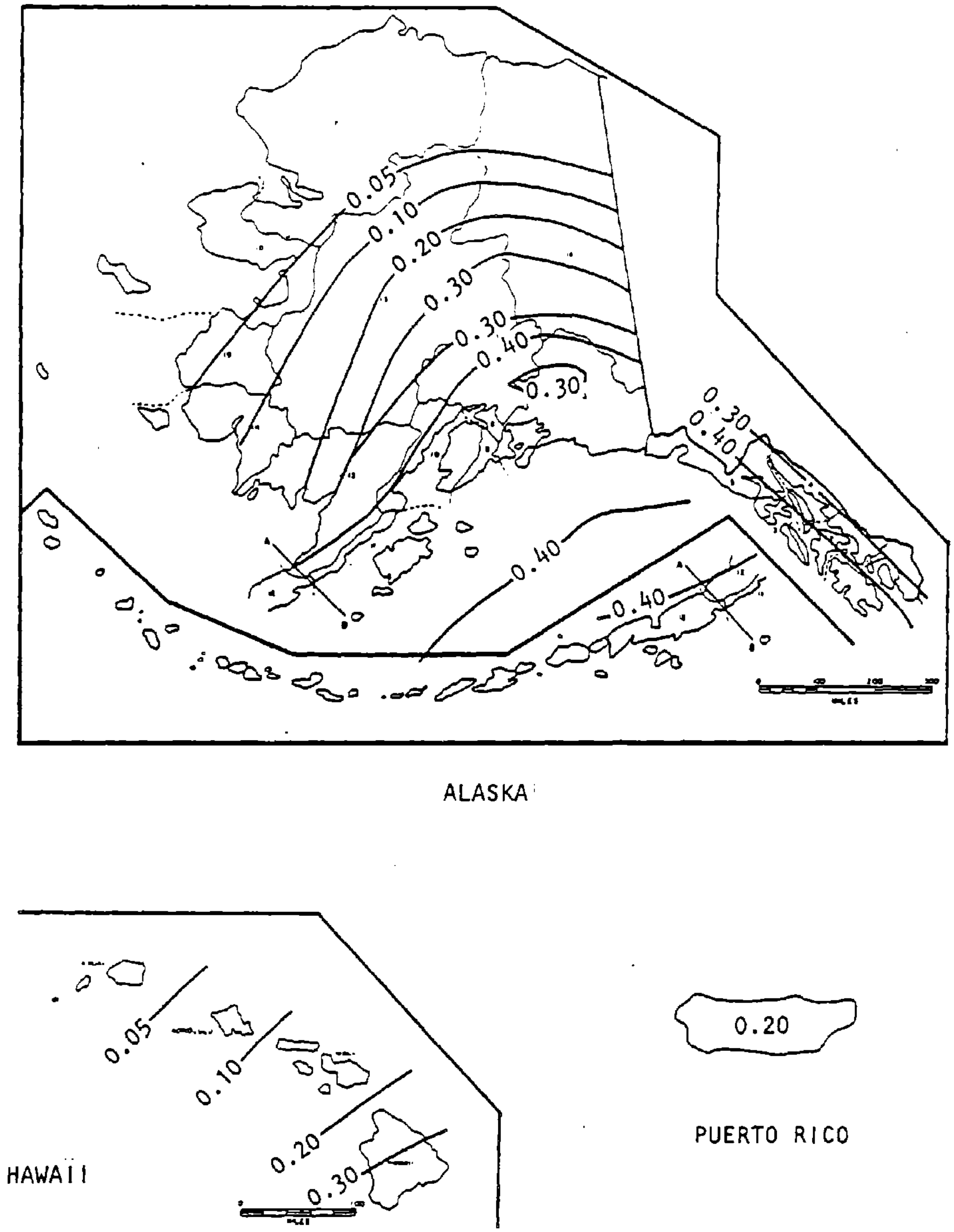


FIGURE 2-2. CONTOUR MAPS FOR EFFECTIVE PEAK ACCELERATION

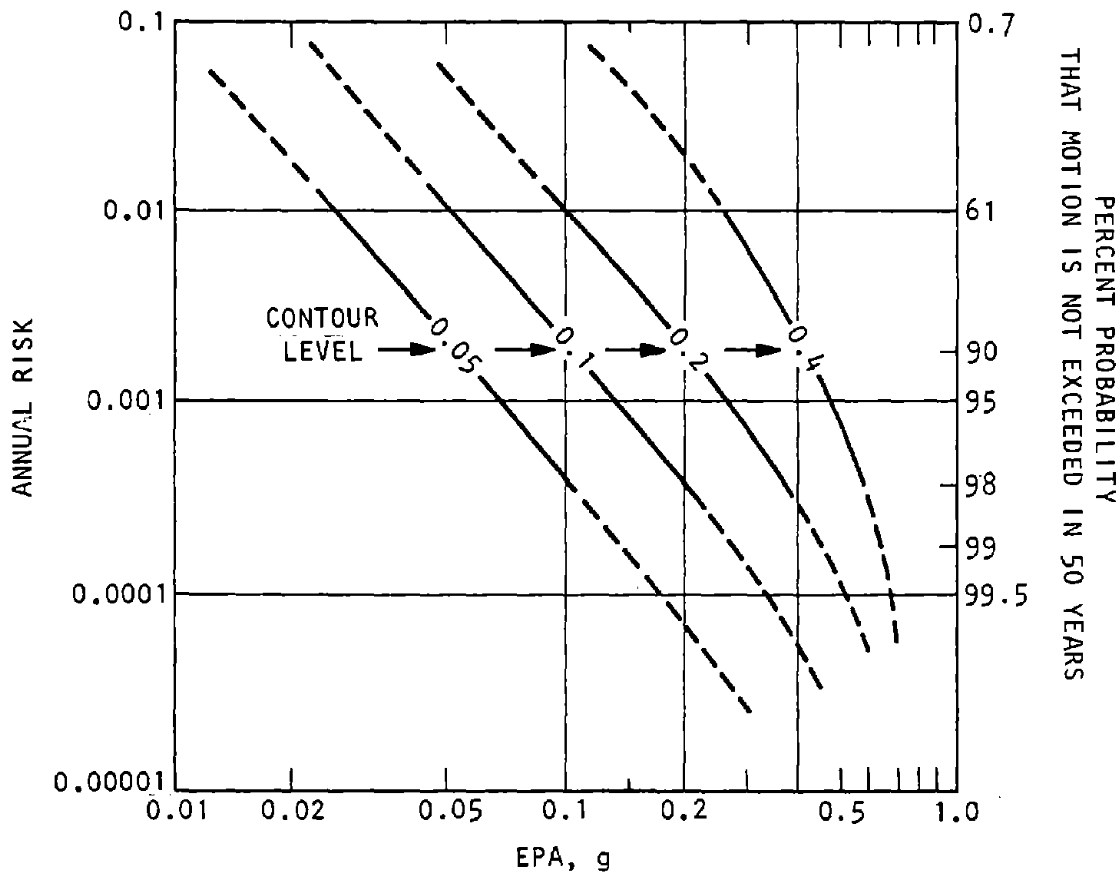


FIGURE 2-3. ANNUAL RISK OF EXCEEDING VARIOUS EFFECTIVE PEAK ACCELERATIONS FOR LOCATIONS ON THE INDICATED CONTOURS OF EPA IN FIGURES 2-1 AND 2-2

SECTION 3

DEFINITIONS AND SYMBOLS

The following definitions and symbols are used throughout this text. They are grouped here for convenience in four general categories: general masonry, joints, walls, and response and analysis terms.

GENERAL MASONRY

Coursing: Lapping of masonry units in each wythe with the masonry units above and below. Typically the units lap one-half of the unit length.

Wythe: The portion of a wall that is one masonry unit in thickness. A collar joint is not considered a wythe.

Header Coursing: A layer of masonry units extending between wythes. In multiwythe walls, header courses generally lap within the wall.

JOINTS

Bed Joint: The mortar unit that is horizontal at the time the masonry units are placed.

Collar Joint: The vertical space separating a wythe of masonry from another wythe or from another continuous material; may be filled with mortar.

Head Joint: The mortar unit between units in the same wythe, usually vertical.

WALLS

Bonded Wall: A wall in which two or more of its wythes of masonry are adequately bonded together to act as a structural unit, usually by header courses.

Cavity Wall: A wall containing continuous air space between wythes; the wythes are tied together with metal ties.

Crosswall: A wall that interconnects levels of horizontal diaphragms. Interconnection of existing crosswalls to diaphragms may be by nominal connections and by finish materials. The yield capacity of a crosswall is not related to the function of a shear wall.

Hollow Unit Masonry Wall: A type of construction made with hollow masonry units in which the units are laid and set in mortar.

Pier: A portion of a URM wall, generally defined as a wall section between door and window openings.

Shear Wall: A wall that has the capacity to couple the above grade inertial mass of the building with ground motion.

RESPONSE AND ANALYSIS TERMS

Building Base: The level that provides support for the URM shear walls. The base is generally considered the floor level at grade unless story height grade changes occur at the building perimeter.

Dynamic Stability: A characteristic of a URM wall that has stability against collapse when shaken by out-of-plane forces or a masonry pier that is cracked and displaced by in-plane inertial forces. Dynamic stability is predicted by consideration of dynamic properties and displacements, not strength requirements.

Horizontal Displacement Control Element: Any floor, mezzanine, or roof that has in-plane (horizontal) capacity to resist the relative displacement of the center of the element and the ends that are in contact with the URM walls or other vertical displacement control elements. In moderate to strong ground shaking, the edges of the floor and roof are excited by the in-plane capacity of the shear walls. The central part of the diaphragm is displaced relative to its ends by inertial forces. Control of dynamic displacements is a goal of the methodology.

Response Forces: The inertial forces that result from the acceleration of the building weight by ground motion. The ground motion is transmitted to the building diaphragms by shear walls and crosswalls.

Restoring Shear Capacity: The elastic capacity of a structural system to restore the deflected building to a no-stress condition after ground motions cease. For a masonry pier system, this restoring shear capacity is provided by the closing of horizontal cracks by gravity loads.

Vertical Displacement Control Element: Any wall that has an in-plane capacity to resist the interstory displacements caused by ground shaking and extends from floor to floor, or floor to roof. These walls may be URM or an internal partition covered with finish materials. However, for partitioning to be effective, it must contact each diaphragm level. Partitioning that extends to a ceiling below a roof should be considered as a displacement control element. The field survey should determine if the ceiling and roof framing are interconnected by framing. URM walls that are not continuous to the building base are displacement control elements in the stories where they are continuous.

Vertical Load Carrying System: A combination of structural elements that provide support for floors, roofs, and URM walls not continuous to the building base. The URM bearing walls are part of the vertical load carrying system.

SYMBOLS

A	Gross area of a URM wall or pier.
C_p	A coefficient used for calculation of inertial forces on an element of the building.
D	Depth of a pier that is measured in the wall plane. Depth of a diaphragm measured perpendicular to the span length "L."
D_1	Equivalent depth of a diaphragm having an opening through the diaphragm adjacent to the diaphragm end. See Section 8.6.
L	Horizontal span of a diaphragm measured between shear walls.
L_1	Equivalent span length of a diaphragm that is controlling the displacement of an "open front" building.
O	Weight of URM wall above the level of wall being analyzed for dynamic stability.
P	Axial load on a URM pier in a shear wall.
SRSS	Square root of the sum of the squares.
v	Bed-joint shear in a URM wall or pier calculated as 1.5 V/A.
v_a	Allowable bed-joint shear for tested URM walls and piers.
v_t	Basic bed-joint shear as determined by in-plane shear testing. Basic shear is reduced to zero axial stress normal to the bed joint.

V Seismic response shear at any level of a shear wall or shear walls.

V_D Calculated shear at diaphragm edge.

V_c The yield capacity of a crosswall or crosswalls.

V_R Elastic restoring shear provided by the structural elements or by the piers in a URM shear wall.

V_u Yield capacity of an element that resists seismic shear.

V_{SRSS} Square root of the sum of the squares of seismic velocities that is imparted by the diaphragms to the ends of the wall being analyzed for dynamic stability.

W Weight of the wall between anchorage levels in that level of wall being analyzed for dynamic stability.

W_D Weight at any level of a diaphragm. For calculation of the in-plane response of a diaphragm it will include the weight of the out-of-plane walls tributary to the diaphragm.

W_w Weight of a URM wall. It may be the total weight of the wall above the base or the weight tributary to any diaphragm level.

SECTION 4

RECOMMENDED PROCEDURE FOR FIELD SURVEYS OF EXISTING URM BUILDINGS

4.1 DETERMINATION OF VERTICAL LOAD-CARRYING SYSTEMS

The analyst of a URM building should develop by a preliminary field investigation an as-built schematic structural framing plan for all levels of the building. This schematic plan will provide the information base for a preliminary hazard analysis. The preliminary hazard analysis will define specific interest areas for a more detailed investigation of the vertical load-carrying system. This detailed investigation may include removal of architectural finishes to determine the complete load path of the vertical load-carrying system. The specific areas defined by the preliminary hazard reduction analysis will be those parts of the vertical load path that will have a probability of joint rotation or displacement when subjected to design ground shaking.

The two major goals of seismic hazard mitigation recommendations are: (1) reduction of the probability of separation of parts of the URM walls from the building and (2) reduction of the probability of partial collapse of the building. Investigation of the vertical load-carrying system is directed toward maintenance of a capable load path for gravity loads when the elements of the building have relative inelastic displacements or joint rotations. Special field investigation should be directed toward determining if the bearing of concentrated loads is on URM wall or pier elements.

4.2 DETERMINATION OF ELEMENTS OR SYSTEMS RESISTING RELATIVE DISPLACEMENTS

The field investigation for determination of the elements or systems that may control relative displacements between the building's base, floors, and roof should include all wall elements that interconnect the floor and roof elements with each other and the building base. Usual concepts of structural and nonstructural walls should not be used to describe the wall system considered in hazard mitigation analysis. If the wall extends to the adjacent floor or roof and has a connection, even if only by finishes, it should be included in the schematic plan. Special investigation should be made at the ceiling-roof level of existing URM buildings with wood roof framing. Partitioning typically extends to the ceiling level, and the roof level may be interconnected to the ceiling system by undesigned trussing in one direction only.

All URM walls of the building should be noted on the schematic plan. Schematic wall elevations should be developed, and continuity of all walls to the building base should be determined. If the wall is noncontinuous to the building base, a special investigation of the wall support system should be made. Opening configurations in the URM wall systems shoud be noted on the schematic URM wall elevations. These schematic URM wall elevations may be developed into detailed elevations if the EPA hazard zone requires an analysis of vertical displacement control elements.

Horizontal relative displacement control elements and systems include all floors, partial floors such as mezzanines, and roofs. The schematic information gathered should include a description of sheathing and finish materials that are applied to wood-framed systems. Extent of floor systems that have a substantial variation in diaphragm stiffness should be noted on the schematic plan. Openings in floors and roofs adjacent to URM walls should be specially noted on the schematic framing plans. Levels of the diaphragms relative to a common datum should be noted. All discontinuities of sheathing materials, especially at the roof level, should be noted. Types of roofing systems, if applied directly to wood-framed roof systems, should be noted.

4.3 INVESTIGATION OF ANCHORAGE OF URM WALLS

The schematic framing plans and URM wall elevations will provide the as-built building plans for defining the areas of investigation for URM wall anchorage. The schematic hazard mitigation analysis, based on the data gathered as described in Sections 4.1 and 4.2, will define wall height/thickness (h/t) ratios that conform to the recommended limit for dynamic stability. The investigation and categorization of the horizontal displacement control elements will define the recommended dynamic force levels for analysis of wall-diaphragm interconnection.

Anchorage of URM walls to the building framing is the most critical and effective part of seismic hazard mitigation. Separation of parts of URM walls is probable in moderate intensity ground shaking if the anchorage is nonexistent or inadequate due to spacing or resistance capacity. Exposure of existing anchorage systems in representative areas of the building is necessary to determine the type and condition of the anchors. Special investigative care should be directed to anchorage details of URM walls that would have been considered nonbearing in the original construction.

Detailed sketches of existing anchorage systems should be prepared during the detailed field investigation. Reference to TR-07 "Anchorage" (ABK, 1983) can provide information on common anchorage systems. The preliminary hazard mitigation analysis will assume that all URM walls are anchored as necessary to minimize the probability of separation. The detailed field investigation must confirm this assumption and discover any deficiency. Unless past experience can reasonably establish the configuration of the embedded parts of the wall anchors, exposure of the embedded portions of a limited number of anchors, approximately 2% of the total number of the existing anchors, should be done in areas where replacement of anchors can be accomplished.

4.4 INVESTIGATION OF EXISTING URM WALLS

The field survey of the URM walls should gather the following information:

- Thickness of URM walls at all levels
- Coursing of exterior wythes of masonry
- Bonding of wythes of masonry, including veneer wythes
- Masonry materials used in each wythe
- Location of thickness changes in walls
- Materials utilized for lintels and/or masonry arch construction
- Materials utilized for columns or piers supporting lintel beams at open fronts
- Height of parapets and cornices above the uppermost existing anchorages or the highest possible retrofitted anchorage system
- Height of gable ends of URM walls
- Anchorage and/or bonding of terra cotta, cast-stone, or stone facing to back-up wythes of brickwork at cornices and similar architectural features

The information obtained from this survey will be utilized for dynamic stability analysis of the walls, computation of anchorage capacity requirements, and design of parapet and appendage bracing systems.

Quality of mortar should be observed. In seismic hazard zones of EPA equal to 0.2 g or less, testing of mortar as specified in Section 4.5 is not recommended, except for severely weathered or eroded areas. Severely eroded mortar joints can be defined as a joint in which the mortar can be removed to a depth of 1-1/2 times the mortar joint width by scraping with two to five passes of a metal tool. Areas of lightly burned brick, generally termed salmon brick, should be noted on the URM wall elevations.

Quality of mortar is a judgmental classification. Density or strength of mortar samples does not necessarily represent a bonding of the mortar to the masonry unit. The methodology analysis methods are not dependent on the tensile capacity of the masonry assemblage.

Dynamic stability of URM walls is determined by assuming that tensile cracking exists in the URM walls. These cracks could have been caused by foundation settlement, temperature changes, or prior seismic shaking. Detailed examination of the URM walls, piers, and columns of buildings shaken by strong ground motions cannot always discover cracks in zones that would have high tensile stresses if the seismic response of the URM walls was elastic. In URM test specimens that have been fully cracked during test sequence, a detailed examination of the cracked area after removal of test loading is often necessary to locate the crack. Gravity loads imposed on the URM wall system close the crack that was opened in dynamic motions. The roughness of the mortar-masonry interface conceals the tensile separation.

Cracks caused by foundation settlement or other effects should be noted on the URM wall elevations. The field investigation should be adequate to reasonably define the cause of the visible crack.

4.5 TESTING OF EXISTING MATERIALS

Application of the methodology for seismic hazard mitigation in existing URM buildings is not dependent on determination of the elastic limit of a stress-strain relationship for undesigned materials. Response characteristics and the material resistance of undesigned assemblages that control relative displacement in URM buildings were determined by static and dynamic testing (ABK, 1981c; 1981d). Resistance capacities of undesigned assemblages such as typical interior partitions are obtained from reported tests (FPL, 1958; APA, 1976). Standard racking tests of nailed systems performed in accordance with ASTM Standard E72 have been used to determine capacity and stiffness properties. Qualification of systems not described in this methodology can be made by the standard racking test.

In general, use of existing materials to control relative displacements in URM buildings subjected to seismic ground shaking does not require testing of the existing materials, except as noted in this section. Displacement control during dynamic displacement is obtained by the capacity of materials that have a stable hysteretic load-displacement characteristic. This characteristic is best described by the commonly known behavior of structural steel. A plotted load-displacement relationship exhibits a near-constant load capacity at displacements in excess of yield displacement. The displacement can be cyclic, and subsequent load-displacement plots will closely approximate the initial force-displacement plotting. Nailed systems exhibit a decreasing stiffness to subsequent cyclic loadings (ABK, 1981c; 1982a), but load capacity increases slightly for increasing postyield displacements. This behavior is termed a stable hysteretic load-displacement characteristic in this methodology.

4.5.1 CONNECTIONS TO URM WALLS

Testing of retrofitted connections of URM walls to floors and roofs is recommended to determine if the connector has a stable load-displacement relationship prior to a failure that is caused by disruption of a brittle material. The brittle material that typically fails is the URM in the embedment zone. For this reason, extensive testing is recommended for connector parts embedded in URM walls. For each type of connector, a test procedure will be described to enable a regulatory body to write a definitive testing requirement.

The connector to a URM wall may be designed to transfer shear, tie the URM wall to the floors and roof (wall anchor), or perform both functions. The installation of the connector must consider a randomness of connector placement in relationship to the mortar joints and adjacent joist pockets (ABK, 1983).

Load and deflection must be recorded. Cyclic loading of less than quasi-yield displacement is appropriate. Permanent offsets at each cyclic load removal should be recorded. Qualification testing of shear or anchorage devices must include a substantial number of test specimens loaded to their ultimate capacity, i.e., failure of the URM assemblage containing the embedment.

Interpretation of the test data is dependent on the purpose of the connector. If the connector is one of many connectors and the plotted cyclic load-displacement is stable as earlier defined, the resistance capacity can be defined as the least of three criteria:

- Mean of the load capacity minus two standard deviations of the data when capacity is defined by failure of the URM materials
- Mean of the load capacity minus one standard deviation when capacity is defined by a plateau on the load-displacement plot
- Mean of the load capacity when resistance is determined by an acceptable permanent offset remaining upon removal of the load, or by a displacement taken from the plotted data

4.5.2 SHEAR CONNECTORS

Specimens for testing of shear connectors to URM walls should be installed in typical configurations. Installation should be in accordance with a specification developed for quality control. Applied test loading should be in the direction of the nearest edge of the URM. The limiting distance to this edge is as prescribed by the specifications. Representative testing is described in ABK-TR-07, "Anchorage" (ABK, 1983).

4.5.3 WALL ANCHORS

Wall anchors to URM walls should be installed in usual configurations and in accordance with a quality control specification. Tensile loads are applied to the anchor extending from the URM wall and reacted against the URM wall. The reaction points on the URM wall should be at a minimum distance from the anchor being tested that is equal to the URM wall thickness.

If a combined wall anchor-shear connector device is to be tested, a combined tension and shear loading in accordance with the prior recommendations should be applied. Test loads in shear and tension should be alternately increased to determine the increase in displacement caused by the subsequent application of shear or tension load.

Testing of wall anchors that are dependent on bonding within the wall thickness should be embedded in a crack width that is appropriate to the probable dynamic out-of-plane displacement of the URM wall. ABK-TR-06 "Interpretation of Wall Tests, Out-of-Plane" (ABK, 1982b) can provide guidance for the test program.

Wall anchorage devices that extend upward or downward from the probable crack in the URM wall may be tested without the requirement of an open crack in the URM wall specimen as previously described. These devices should be tested with the anchor inclined toward the free edge of the wall specimen, unless an adjacent free edge is prohibited by the installation specifications.

Anchor devices that depend upon the expansion of the device on the opposite face of the wall from the floor or roof should be tested with bond eliminated between the enlargement and the interior wall face. The decrease in anchor capacity due to an open crack in the enlargement zone should be determined.

4.5.4 EXISTING WALL ANCHORS

Existing wall anchors that depend on a bar or plate embedded in the URM wall should be tested by nondestructive methods that have been qualified by destructive tests. This qualification procedure can be generally applied to common existing wall anchorage systems that are typical of a broad geographic area. ABK-TR-07 "Anchorage" describes an anchor observed throughout the United States which is called a "government anchor." This 3/4 in. (19 mm) round rod depends on a 3/4 in. x 9 in. (19 x 230 mm) pin embedded in the interior wythes of clay brick walls. An existing anchorage testing program now in effect has the following requirements (SEASC, 1981): "5% of the existing rod anchors utilized as all or part of the required wall anchors shall be tested in pullout by an approved testing laboratory. The minimum number tested shall be four per floor, with two tests at walls with joists framing into the wall and two tests at walls with joists parallel to the wall. The test apparatus shall be supported on the masonry wall at a minimum distance of the wall thickness from the anchor tested. The rod anchor shall be given a preload of 300 lb prior to establishing a datum for recording elongation. The tension test load reported shall be recorded at 1/8 in. (3 mm) relative movement of the anchor and the adjacent masonry surface. Results of all tests shall be reported."

This nondestructive test procedure was based on a limited number of destructive tests. The destructive tests indicated that 1/8 in. elastic displacement can be correlated with a percentage of the failure load capacity. The failure load capacity was controlled by a disruption of the brickwork. An acceptable displacement associated with regional construction quality should be determined by destructive testing.

4.5.5 QUALITY CONTROL TESTING FOR EMBEDDED ANCHORS

Resistance capacities for embedded anchors in URM walls are dependent on the quality of workmanship. Quality control testing is recommended for all items that depend on installation of materials that cannot be visually inspected after installation. Through-wall anchors are excepted, as all elements that provide pullout resistance are visible on the wall surfaces.

The following procedure is recommended for testing embedded shear and tension bolts: 25% of all new shear bolts and dowels embedded in unreinforced masonry walls should be tested by an inspector using a torque calibrated wrench to the following minimum torques:

1/2 in. diameter bolts or dowels = 40 ft-lb
5/8 in. diameter bolts or dowels = 50 ft-lb
3/4 in. diameter bolts or dowels = 60 ft-lb

No bolts exceeding 3/4 in. diameter should be used.

The test torque is a qualification test only and is not implied to represent a loading condition. Current quality control testing indicates that the bolt can be withdrawn from the mortar or grout used for embedment by the specified torque if the material does not fill the full depth of the drilled hole.

4.5.6 QUALIFICATION TESTING FOR URM

Testing to quantify specific elastic properties of URM is not intended for the full range of masonry assemblages used in the United States. Testing of URM is recommended only for buildings in the seismic hazard zone with EPA 0.4 g or for buildings with severely eroded mortar. If the erosion of mortar indicates repointing of the joints is necessary, this repointing should be done in accordance with recommendations such as those of the U.S. Department of Interior, Preservation Briefs (USDI, 1980) prior to in-plane testing.

Nondestructive testing (Noland, 1982) has indicated that the relative quality of masonry can be determined by the several test methods utilized in that research. It is suggested that nondestructive testing be combined with some destructive tests that provide a strength reference. However, the research on nondestructive testing suggests that these test methods are more applicable to recently built masonry than to the older masonry construction. In addition, none of the test procedures, destructive or nondestructive, were found to have moderately good correlation with joint shear strength.

For the older multiwythe brick masonry, typically constructed with low-strength mortar, the methodology recommends use of the following qualification test:

> In-Place Shear Tests. The bed joints of the outer wythe of the masonry should be tested in shear by laterally displacing a single brick relative to the adjacent bricks in that wythe. The opposite head joint of the brick to be tested shall be removed and cleaned prior to testing. Steel bearing plates of the full dimension of the brick shall be inserted at each end of the test jack. The bearing plates shall not contact the mortar joint. See Fig. 4-1. To qualify, the minimum quality mortar in 80 percent of the shear tests should not be less than the total of 30 psi when reduced to an equivalent zero axial stress. The shear stress should be based on the gross area of both bed joints and should be that at which movement of the brick is first observed. See Sec. 9.6 and Fig. 9-1.
>
> Number and Locations of Tests. The minimum number of tests should be two per wall or line of wall elements resisting a common force, or 1 per 1500 square feet of wall surface, with a minimum of ten tests in any case. The exact test or core location should be determined at the building site by the licensed engineer or architect responsible for the seismic analysis of the subject building. In single-story buildings, the masonry above the lintel beam at an open front need not be tested. However, the number of required tests should not be reduced.

Retrieval of cores from masonry walls for testing is not recommended. Analyses of 8-in. (200 mm) cores indicate that the state of shear stress in cored shear specimens is not uniform across the cross section (Noland, 1982).

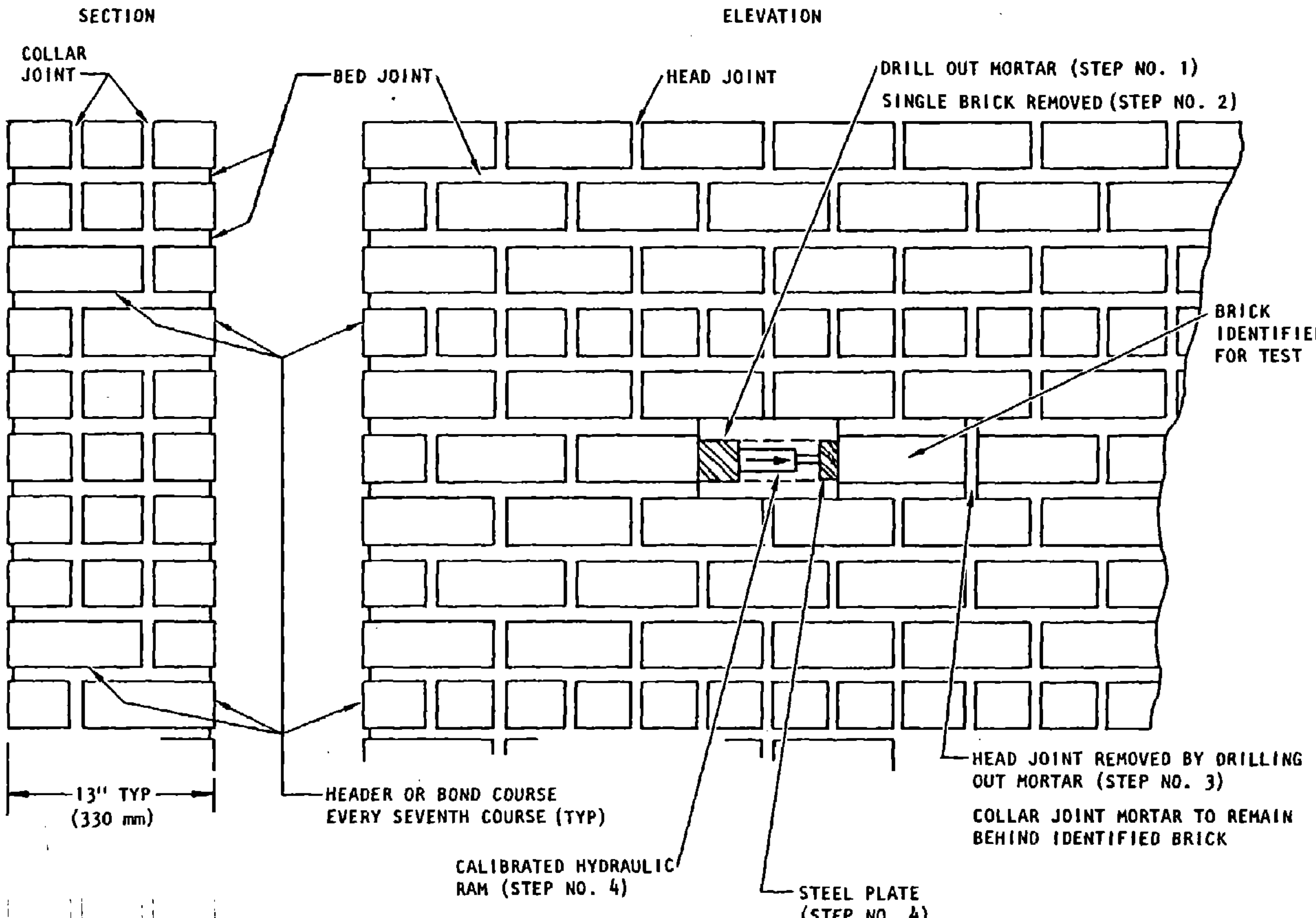


FIGURE 4-1. IN-PLACE SHEAR TESTS

SECTION 5

RESPONSE OF EXISTING STRUCTURAL SYSTEMS

5.1 URM WALLS, IN-PLANE GROUND MOTION

During an earthquake, the ground motion is transmitted from the building/foundation interface through the end walls (in-plane response) to the floor and/or roof diaphragms, and the diaphragms drive the URM walls in the out-of-plane direction. Accordingly, the in-plane response of the URM end walls directly influences the kinematic environment delivered to the walls and the ends of the diaphragms.

It is clear that a rigid URM end wall will deliver larger motions to the diaphragms than flexible and ductile end walls. An analytical study of the response of rigid end walls of varying aspect ratios resting on a wide range of soil foundations was conducted; the results are reported in Appendix A. The results show that over a realistic range of building and soil characteristics the ground motion is transmitted through the end walls with little amplification. Accordingly, in actual URM buildings it is sufficient to assume that the ground motion is transmitted unmodified through end walls (in-plane) and is directly transmitted to the diaphragms.

5.2 HORIZONTAL ELEMENTS, IN-PLANE MOTIONS

All floors, mezzanines, and roofs are shaken by ground motions carried upward in the building by the URM shear walls. The typical materials used for floor and roof construction are amplifiers of the input motions for a broad range of shape factors and dynamic masses. If the shape factor (span/depth ratio) is more than the critical shape factor or if the available attached mass forces the horizontal element into significant inelastic displacements, the amplification of input motion is minimized. However, in this case, limitation of relative horizontal displacement becomes a significant problem.

Amplification of input acceleration has a major influence on the prediction of wall anchor forces. A diaphragm with a span/depth ratio of 3 to 4 may have significant relative dynamic displacement in the direction of the long span. But the shape factor of 1/3 to 1/4 on the other axis of excitation can cause high amplification of input motions and increase wall anchorage forces in the ends of the diaphragm. Amplification of the input velocity by the diaphragm is the major parameter affecting stability of the URM walls. The amplification ratio again is strongly related to diaphragm shape factor (span/depth ratio) and the coupled mass.

5.3 MODIFIERS OF RESPONSE OF HORIZONTAL ELEMENT

The parameters that affect diaphragm response are:

- Initial stiffness at small displacements
- Yield capacity of diaphragms
- Available inertial mass coupled with the diaphragm
- Materials of construction
- Shape factor (span/depth ratio)

A description of the materials of diaphragm construction is used in the methodology to define diaphragm initial stiffness and yield capacity. When these properties are defined for any construction assemblage, modification of the shape factor has a linear relationship with initial stiffness and yield capacity. If a diaphragm is 60 ft deep by 120 ft span (span/depth ratio = 2), its response is not significantly altered by doubling the available response mass (ABK, 1982a). However, if its depth is doubled, the coupling of the mass to the shear wall may be more than doubled.

In studying the parameters that affect diaphragm response, it was recognized that an inelastic damper placed in the center of the diaphragm span had the most significant influence on diaphragm amplification and relative displacement (App. B). In the typical URM building these elements are the crosswalls that interconnect diaphragms with each other and with the ground. The methodology considers dynamic response that is modified by the presence of crosswalls, their spacing along the diaphragm span, and their yield capacity.

5.4 RESPONSE OF WALLS, OTHER THAN URM, FOR IN-PLANE GROUND MOTION

Crosswalls are recognized in the methodology as a critical modifier of diaphragm response. The methodology procedures, in many cases, will encourage the analyst to introduce crosswalls of the minimum capacity at the maximum spacing into the URM building. The methodology recognizes the benefit of existing crosswalls. The total yield capacity of existing crosswalls may greatly exceed the minimum recommended capacity. In this case the crosswalls may be the path for excitations of the diaphragm rather than the URM shear walls at distant ends of the diaphragm. Also, the crosswalls can perform either as near rigid bodies carrying unamplified ground motions upward to the upper levels of the building or as a yielding element exciting the diaphragm levels above the base. These cases have been considered in preparation of the recommendations of the methodology. The density of the crosswalls that may cause them to act as shear walls limits diaphragm amplification. This limitation on diaphragm amplifications reduces anchorage forces, minimizes the probability of URM wall instability, and reduces the coupling of the diaphragm mass with the URM shear wall. Therefore, the recommendations of the methodology are based on the use of minimum capacity crosswalls. An upper bound of crosswall capacity need not be recommended.

SECTION 6

METHODOLOGY FOR MITIGATION OF SEISMIC HAZARD IN AREAS OF DESIGN GROUND MOTION OF EPA = 0.1 G

6.1 CATEGORIZATION OF HAZARDOUS BUILDING ELEMENTS

Hazardous elements of existing buildings in this seismic hazard zone consist of parts of URM walls that may separate from the building under design ground shaking. Separation of URM at the building exterior constitutes a life-safety hazard to persons adjacent to the building. Also, URM elements within the exterior building perimeter may fall through the roof and threaten building occupants.

The hazardous building elements are categorized as follows:

- URM cornices, parapets, and appendages extending above the uppermost anchorage level.
- URM walls adjacent to roof elements not continuous with the major plane of the roof sheathing. Mansard roofs, roof edges pitched for roof drainage, and end walls of northlighted roof framing are examples of these hazardous building elements.
- URM walls adjacent to skylights or other openings through the roof and/or floors.
- URM walls with unbonded veneer courses.
- URM walls without anchors to roofs and floors above ground.
- Gable ends of URM walls.
- Masonry ornamentation cantilevering from the URM wall face.

The hazard posed by each of the elements increases in a direct ratio to its height above the building base. A minimum seismic hazard reduction program should consist of anchorage of URM walls at the roof level and bracing of URM parapets and other projections above the roof.

6.2 ANCHORAGE OF URM WALL ELEMENTS

Recommendations for calculation of anchorage forces for design of wall anchors or analysis of existing anchorage systems are as follows:

- Assume the URM wall contributing response to the anchorage system is 1/2 of the wall height measured to the adjacent level(s) of wall-anchors or to the base of the building.
- Calculate the anchorage force at each roof or floor level as 0.4 times the URM wall weight tributary to the anchorage level.
- Determine the capacity of existing wall anchorage systems in accordance with the recommendations of Section 9.4. Existing wall-anchor systems should be tested in accordance with Section 4.5.4. Design capacities for existing tested wall anchorage systems should be determined in accordance with Section 4.5.1.
- Design retrofitted wall anchorage systems in accordance with the recommendations of Section 10.4. Design capacities for retrofitted wall anchorage systems qualified by testing should be established in accordance with Section 4.5.1. Qualification tests for wall anchorage systems should be conducted in accordance with Section 4.5.3. Quality control testing for retrofitted wall anchorage systems should be done in accordance with Section 4.5.5.
- Design bracing systems for URM parapets and appendages extending above the roof anchorage level. Use recommendations for wall anchorage systems to calculate bracing forces. See Section 10 for recommendations for member design criteria.

Design recommendations for wall anchorage systems are based on a series of simplifying assumptions. Each of these assumptions is discussed to enable the analyst to assess the probability of compounding of assumptions (EERL, 1977).

Input ground motions for the geographic areas of the United States with a design intensity of ground shaking of 0.1 EPA were selected to represent a mean intensity (ABK, 1981b). Section 2.1 discusses the rationale for this recommended procedure. These unamplified ground input motions are assumed to be imparted to the edges of the horizontal elements by the URM walls. Amplification of ground motion may occur in buildings with URM walls having a height/length ratio in excess of 1-1/2 when founded on moderately soft soils. This probability is discussed in Sections 2.3 and 5.1.

The input ground motions imparted to the edges of the horizontal diaphragms are assumed to be amplified by the diaphragms. The recommended upper bound of amplification of acceleration pulses of the input ground motions in this seismic hazard zone is 4. This is because dynamic testing of diaphragms (ABK, 1981c, 1982a) recorded amplifications of 3, and amplifications of 4 times the input motions have been recorded in instrumented diaphragms shaken by ground motion appropriate to this seismic hazard zone (OSA, 1978, 1980; CDMG, 1981). This recommendation of the upper bound of amplification is based on the authors' judgment. Other factors influencing this judgmental recommendation were:

- Probable separation of URM walls from the building constitutes a significant life-safety threat.
- Capacity of anchorage systems is strongly influenced by seismic response in the acceleration range. Peak accelerations, rather than effective peak accelerations (EPA), can be amplified by near-elastic diaphragms and initiate failure of the ends of wall anchors embedded in URM walls.
- When retrofitted anchorage systems are required due to the absence of any anchors, the cost of the retrofitted system is not sensitive to the recommended design force.

The inertial weight of the URM wall assumed tributary to the wall anchorage level is taken as one-half the distance to the adjacent levels of anchorage. This is a reasonable assumption, based on data obtained from dynamic testing of diaphragm specimens (ABK, 1981c, 1982a).

Recommendations for establishing capacity of existing or retrofitted wall anchorage systems (Sec. 4.5.1) are based on the authors' judgment. Analogies to current seismic design requirements were used to develop these recommendations. If the test failure mode is within a brittle material (URM) and the data have a significant scatter, the design capacity is significantly reduced from the mean of the data obtained from testing. Other ultimate capacities that are determined by testing are reduced less. These recommendations are intended to penalize brittle systems with erratic test values and to encourage use of systems with low-scatter test values and quasi-yielding behavior.

6.3 STABILITY OF ANCHORED URM WALLS

Dynamic testing of anchored URM walls (ABK, 1981d) indicates that URM walls constructed in accordance with height/thickness limitations of past practice and ordinances have a very high probability of dynamic stability. Cracking of URM walls at the upper bound of height/thickness limits that were prescribed by past regulations may occur. The predictions of dynamic stability for URM walls are not dependent on the tensile capacity of URM (ABK, 1982b).

If unusual wall height/thickness ratios in tall buildings that are founded on moderately soft soils are discovered in the building survey, the analyst may use the analysis procedures described in Section 8.3. An input ground velocity of 3 in./s (7.5 mm/s) is recommended as appropriate for this seismic hazard zone. Probable amplification of this ground motion by the in-plane URM walls can be taken from Section 5.1. For the determination of the SRSS design velocities, a maximum amplification by the diaphragm of three times the modified velocities may be assumed.

6.4 ANALYSIS OF HORIZONTAL DISPLACEMENT CONTROL ELEMENTS

Observation of URM buildings shaken by ground motions appropriate to this seismic hazard zone indicates that common undesigned diaphragms have acceptable displacement control capacities. The recommended survey of diaphragms by the procedures described in Section 4.2 is intended to discover discontinuities adjacent to the required anchorages of the URM walls. If a discontinuity in the horizontal diaphragm exists, a tie system to carry anchorage loads through the discontinuity should be designed. If an opening in the horizontal diaphragm adjacent to the URM wall exists, a horizontal beam at the level of the diaphragm should be designed. In both cases, the forces recommended for design of anchorage systems should be used for beam or tie design. See Section 10 for recommendations for member design criteria.

6.5 INTERCONNECTION OF BUILDING ELEMENTS

The development of the methodology was based on a seismic response model that assumed interconnection of all of the parts of the building. The anchorage of URM walls to floors and roofs is part of this interconnection and is required. Interconnection of the edges of diaphragms to the URM walls for shear transfer is generally adequate even in typical undesigned connections. Observation of buildings shaken by earthquake intensities appropriate for this hazard zone indicates that a very small risk can be assigned to the lack of designed interconnections other than wall anchors.

The field survey of existing URM buildings described in Section 2 is intended to discover unusual as-built conditions. A seismic hazard mitigation analysis should consider the effect of small relative displacements on parts of the vertical load-carrying system. If the review indicates that damaging relative displacements have a probability of occurrence, a tie system should be designed. The design of the interconnection system, other than wall anchors, should be based on a response force of 0.1 times the weight of the element to be connected. For shear transfer at the ends of diaphragms having an unusual configuration, the design shear force need not exceed the diaphragm shear capacity given in Section 9.5.

6.6 REVIEW OF VERTICAL LOAD-CARRYING ELEMENTS

An analysis of the lateral-load capacity of the vertical load-carrying system is not recommended for this seismic hazard zone. The as-built system should be reviewed to determine if substantial vertical loads are transferred from stiff elements, such as major steel lintels, to URM piers. The review of the vertical load-carrying system should consider that small relative displacements will occur at the vertical discontinuities in the URM walls. The "open-front" URM building typifies this discontinuity. A similar condition may occur at the ground floor level supports for URM walls forming light and ventilation courts in the upper levels of buildings.

If the review of the vertical load-carrying system discovers that the bearing surfaces of URM columns and piers are sensitive to rotation caused by interstory displacement and that the bearing stress distribution induced by the displacement has a probability of causing a brittle bearing failure of the

URM, a supplemental load-carrying system can be added adjacent to the URM bearing surface. Logically, this supplemental system would be a pin-ended steel column that is relatively insensitive to beam-column joint rotation.

SECTION 7

METHODOLOGY FOR MITIGATION OF SEISMIC HAZARD IN AREAS OF DESIGN GROUND MOTION OF EPA = 0.2 G

7.1 CATEGORIZATION OF HAZARDOUS BUILDING ELEMENTS

Hazardous building elements in this seismic hazard zone include all those building elements described in Section 6.1 plus the following:

- Vertical load-carrying systems consisting of nonductile concrete beams and columns that provide vertical support for a URM wall that is not continuous to the base of the building.
- A vertical load-carrying system consisting of steel beams supported on masonry piers or columns that provides support for a URM wall that is not continuous to the base building.

The recommended analysis for determination of the hazard posed by these parts of a URM building is described in this section.

7.2 ANCHORAGE OF URM WALL ELEMENTS

Recommendations for calculation of anchorage forces for design of wall anchors or analysis of existing anchorage systems are as follows:

- Assume the URM wall mass contributing response mass to the anchorage system is 1/2 of the wall height measured to the adjacent level(s) of wall anchors or to the base of the building.
- Calculate the anchorage forces at each roof or floor level as 0.6 times the URM wall weight tributary to the anchorage level.
- Determine the capacity of existing wall anchorage systems in accordance with the recommendations of Section 9.4. Existing wall anchor systems should be tested in accordance with Section 4.5.4. Design capacities for existing wall anchorage systems should be determined in accordance with Section 4.5.1.
- Design retrofitted wall anchorage systems in accordance with the recommendations of Section 10.4. Design capacities of wall-anchorage systems qualified by testing should be established in

accordance with Section 4.5.1. Qualification tests for wall-anchorage should be in accordance with Section 4.5.3. Quality control testing for retrofitted wall anchors should be in accordance with Section 4.5.5.

- Design bracing systems for URM parapets and appendages extending above the roof anchorage level. Use recommendations for wall anchorage systems to calculate bracing forces. See Section 10 for recommendations for member design criteria.

Design recommendations for wall anchorage systems are based on a series of simplifying assumptions. The discussion of Section 6.2 is applicable for this seismic hazard zone with the following exception:

The recommended upper bound of diaphragm amplification of acceleration pulses of the input ground motion is 3. This amplification factor is adequate to define the diaphragm response to high-energy ground motion. The displacement time histories selected for dynamic testing were matched to smoothed response spectra (ABK, 1981b). This selection method uses families of ground motions for each seismic hazard zone. One of the selected time-displacement histories appropriate for this hazard zone contains single peak accelerations of nearly twice the seismic zone EPA. Response of the tested diaphragms to this acceleration pulse fits within the recommended acceleration amplification factor times the zonal EPA. The recommended factor of 0.6 times the tributary wall weight for wall anchorage analysis accounts for the amplified response of the URM wall.

7.3 STABILITY OF ANCHORED URM WALL ELEMENTS

Dynamic testing of anchored URM walls (ABK, 1981d) indicates that URM walls constructed in accordance with height/thickness limitations of past practice and ordinances have a very high probability of dynamic stability. Cracking of URM walls at the upper height/thickness limits prescribed by past requirements may occur. The predictions of dynamic stability for URM walls are not dependent on the tensile capacity of URM (ABK, 1982b).

If unusual wall height/thickness ratios in tall buildings founded on moderately soft soils are discovered in the building survey, the analyst may use the analysis procedures described in Section 8.3. An input ground velocity of 6 in./s (150 mm/s) is recommended as appropriate for this seismic hazard zone. Amplification of this ground motion by the in-plane URM walls can be taken from Section 5.1. For the determination of the SRSS diaphragm velocities, a maximum amplification by the diaphragm of 2-1/2 times the modified velocities may be assumed.

7.4 ANALYSIS OF HORIZONTAL DISPLACEMENT CONTROL ELEMENTS

The recommendations and discussion of Section 6.4 are applicable to this seismic hazard zone. In addition, the anticipated seismic intensity in this hazard zone has a probability of causing a relative horizontal shear displacement at the diaphragm-URM wall juncture in special cases.

The recommended field survey will develop elevations of all URM walls in the building. When URM walls in the upper levels are not continuous to the base of the building, the horizontal diaphragms must control the relative displacement of the wall and the ground. In general, analysis of the displacement control capacity of diaphragms is not recommended for this seismic hazard zone. If the analyst discovers special conditions in the field survey, an analysis as described in Section 8.6 can be made by factoring the demand/capacities ratio, $W_D/2v_u$, by the ratio of seismic zone EPA's.

The recommendations of this methodology for shear transfer at the diaphragm edges parallel to URM walls that are not continuous to the base of the building are:

- Design the shear transfer at the edge of the diaphragm for the <u>lesser</u> of the forces calculated as follows:

  a. The shear force, v_u, as given in Table 9-1.

  b. C_p, from Table 7-1, times the building weight tributary to the diaphragm. The recommended method for computing tributary weight is similar to current seismic design procedures.

For multistory buildings, with existing internal crosswalls, constructed of the materials described in Section 9.6, in all levels above the level with the discontinuity in the URM walls, the total weight tributary to all of the diaphragms should be redistributed to each diaphragm in proportion to the v_u recommended in Section 9.5. However, the designed shear connection need not exceed v_u.

TABLE 7-1. RESPONSE FACTORS FOR EXISTING DIAPHRAGMS

Description of Existing Materials	Value of C_p
Single layer of boards or plywood with applied roofing	0.35
Double or multiple layer of boards or blocked plywood sheathing	0.5
Steel decking, detailed for lateral load capacity	0.45
Steel decking, without detailed lateral load capacity	0.4
Concrete filled steel decking or concrete floor systems with span/depth ratio of 3 or less	0.3
Concrete filled steel decking or concrete floor systems with span/depth ratio of less than 2	0.2

7.5 ANALYSIS OF VERTICAL DISPLACEMENT CONTROL ELEMENTS

For the purpose of this section, vertical displacement control elements are defined as URM walls extending above the building base. URM walls not continuous to the building base are not considered to have a significant influence on vertical displacement control for this seismic hazard zone.

In general, analysis of URM walls for in-plane forces is not recommended for this seismic hazard zone. URM buildings of usual configurations shaken by earthquakes appropriate to this seismic hazard zone have not sustained life-threatening in-plane damage (ABK, 1981a). For special cases, such as when a single URM wall with many openings for doors and windows is the only existing vertical displacement control element on one axis of the building, an analysis of this URM wall in conformance with the recommendations of Section 8.7, for half of those recommended story shear capacities, will provide a comparable seismic hazard reduction. Capacity of all existing vertical elements, as described in Section 9.6, should be included in the analysis. If retrofitted structural elements are required by the analysis, the elements should be designed in conformance with the recommendations of Section 10.

7.6 INTERCONNECTION OF BUILDING ELEMENTS

The recommendations and discussion of Section 6.5 are applicable to this seismic hazard zone. However, interconnection of the edges of diaphragms for special cases, as described in Section 7.4, is recommended. Displacement of the diaphragm, relative to in-plane URM walls, has been an observed cause of damage to corners of URM walls.

Design forces for wall anchors should be based on the anchorage forces given in Section 7.2. Design forces for other interconnections should be based on a response force of 0.2 times the weight of the element to be connected.

7.7 REVIEW OF VERTICAL LOAD-CARRYING ELEMENTS

The recommendations and discussion of Section 6.6 are applicable to this seismic hazard zone.

SECTION 8

METHODOLOGY FOR MITIGATION OF SEISMIC HAZARDS IN AREAS OF DESIGN GROUND MOTION OF EPA = 0.4 G

8.1 CATEGORIZATION OF HAZARDOUS BUILDING ELEMENTS

Hazardous building elements in this seismic hazard zone include all those building elements described in Sections 6.1 and 7.1 plus the following:

- All URM walls, with or without openings for doors and windows, that extend upward from the base of the building.
- All URM walls, with or without openings for doors and windows, that are not continuous with the base of the building.

The recommended analysis for determination of the hazard posed by these parts of the building is described in this section.

8.2 ANCHORAGE OF URM WALL ELEMENTS

The recommendations and discussion of Section 7.2 are appropriate for this seismic hazard zone, except that the anchorage forces should be calculated as 1.0 times the URM wall weight tributary to the anchorage level.

8.3 STABILITY OF ANCHORED URM WALL ELEMENTS

Recommendations for analysis of dynamic stability of URM walls in this seismic hazard zone are given by prescribing acceptable height/thickness ratios. The selection of height/thickness ratios is determined by the absence or presence of crosswalls as defined in this section, or by diaphragm demand/capacity ratio and span length (L). Acceptable height/thickness ratios are given in Table 8-1.

Crosswalls are defined as existing walls constructed of materials other than URM or retrofitted similar structural elements. Crosswalls should extend between all diaphragms at all levels of the building. Spacing of crosswalls should not exceed the spacing specified in Table 8-3. Capacity of crosswalls should not be less than specified in Table 8-3.

TABLE 8-1. ALLOWABLE HEIGHT/THICKNESS RATIO OF URM WALLS WITH MINIMUM QUALITY MORTAR

	Buildings with Crosswalls or Diaphragms with Demand/Capacity Ratio and Span as Specified in Table 8-2	All Other Buildings
Walls of one-story buildings	20	14
First story walls of multistory buildings	20	20
Walls in top story of multistory buildings	14	9
All other walls	20	15

TABLE 8-2. MINIMUM DEMAND/CAPACITY RATIO AND SPAN OF DIAPHRAGMS BETWEEN URM SHEAR WALLS FOR QUALIFICATION AS "WITH CROSSWALLS"*

Horizontal Span Between URM Shear Walls	Minimum Demand/Capacity Ratio
60 ft or less	2.5
180 ft maximum	3.0

*Minimum demand/capacity ratios may be interpolated for diaphragms with spans between 60 and 180 ft

TABLE 8-3. MINIMUM CAPACITY OF CROSSWALLS AND SPANS (L) OF DIAPHRAGM[1 2 3]

Diaphragm Span (L) in Feet	Demand/Capacity of Diaphragm	Minimum Capacity of Crosswalls as a Percentage of Diaphragm Capacity ($v_u \cdot D$)
300 or more	1.0 or less	30
180 minimum	2.0 or more	30

[1]Minimum demand/capacity ratios may be interpolated for diaphragms with spans between 180 and 300 feet.

[2]Maximum spacing of crosswalls is 40 feet measured in the direction of the span.

[3]Not applicable for steel decking detailed for lateral load resistance, concrete filled steel decks, and concrete framed floors.

Dynamic stability of URM walls is highly dependent on the response of the horizontal diaphragms. All diaphragms will amplify the input motions applied to the edges of the diaphragm in some span/depth configurations and within a range of demand/capacity ratios. Dynamic stability of URM walls was developed from full-size testing (ABK, 1981d), and stability criteria were developed by analysis of collected data (ABK, 1982b). The methodology uses the criteria of predicted dynamic stability shown in Figure 8-1. The parameters that affect stability are:

- SRSS of the input velocities imparted by the diaphragms to the ends of the URM wall.
- The ratio of weight of wall in the stories above the story under consideration, "O," to the weight of the URM wall in the story under consideration, "W."
- The height/thickness ratio of the wall in the story under consideration.

The parameters O/W and H/T are calculated by the analyst. The SRSS of the amplified input velocities used for preparation of Table 8-3 were calculated by the following procedures.

For buildings with crosswalls as defined in this section, or with diaphragms having minimum demand/capacity ratios and maximum spans specified by Table 8-2, the prescribed amplification of input velocities is taken as 1-3/4. The mean ground input velocity for this seismic hazard zone is taken as 12 in./s (0.3 m/sec) (ABK, 1981b). For walls in single-story buildings or in the first story of multistory buildings, the basic input velocity at the base of the wall is the ground motion, 12 in./s. For walls above the first story, the input velocity to the wall end is the amplified velocity, 21 in./s (.53 m/s). This input velocity is assumed at all levels including the roof. Crosswalls, especially existing walls, commonly will have enough yield capacity to cause a common displacement of the upper story floors and roof.

The ratio O/W is taken as 0 for the walls of single-story buildings and for walls in the top story of multistory buildings. The ratio O/W is taken as 0.5 for all other walls. H/T ratios of greater than 20 are not recommended in this seismic hazard zone.

The recommendations for height/thickness ratios for dynamic stability of URM walls in all other buildings, those without crosswalls, are based on the following velocity amplification ratios:

- For typical wood-framed roof construction: 2
- For typical wood-framed floor construction: 2-1/4

Input ground motion is taken as 12 in./s (0.3 m/s). Ratios for O/W are as used for walls in buildings with crosswalls. The recommended maximum H/t for this seismic hazard zone is 20.

Table 8-2 provides recommendations for the minimum demand/capacity ratios and span length (L) of diaphragms that limit amplification of input velocities to 1-3/4. These ratios were developed from dynamic testing (ABK, 1981c) and analysis of data extended by computer modeling (ABK, 1982a).

Table 8-3 provides recommendations for maximum spacing of crosswalls. This recommendation is keyed to the dynamic response of diaphragms. If the diaphragm has an appropriate demand/capacity ratio and span, as defined by Table 8-2, the diaphragm response has a limited amplification of input motions. As the diaphragm demand/capacity ratio decreases, the diaphragm response moves into an amplification region that is unacceptable for prediction of the stability of URM walls using Table 8-1, "with crosswalls."

The purpose of the crosswall is not to provide a seismic shear wall designed to input ground motions into the diaphragm. The crosswall, if retrofitted into the building, should be specified at near the minimum recommended capacity since its function is as an inelastic damper that is introduced into the system. To fulfill this requirement, the crosswalls must exist in all levels of the building above the base.

If there are already crosswalls in the building, their capacity may exceed the minimum capacity recommended by Table 8-3. The recommended crosswall capacity (Table 9-1) was determined by static testing, and this capacity is maintained at large inelastic displacements. The crosswalls need not have the capacity to transmit response ground motions to the diaphragm. If URM walls or other masonry or concrete walls exist in the building, they should not be considered as crosswalls. These shear walls are the elements that excite the diaphragms with ground motions. This methodology recommends that the crosswalls be constructed of sheathed wood framed systems. Other materials, such as moment frames, must have a yield-deflection compatibility analysis and must be detailed in full conformance with code requirements for ductile moment frames. This ductile capacity requirement is applicable to retrofitted elements that are used for crosswalls, but not all supplemental elements that are designed in accord with Section 10.

As discussed, the crosswall must have the minimum capacity recommended in Table 8-3. This capacity is prescribed to provide a minimum energy absorption and is keyed to a percentage of diaphragm capacity, v_u. The initial stiffness of the diaphragm or crosswall does not have a significant response effect (ABK, 1982a). However, use of crosswalls to minimize diaphragm response is not applicable to certain framing systems described in Table 8-3. These framing systems have both a high initial stiffness and ultimate capacity. Prediction of a response reduction by use of an inelastic damper for these materials is not within the scope of this methodology.

If existing crosswalls do not meet the requirements of Table 8-3 and introduction of crosswalls into the building is not feasible, the stability of the URM walls may be increased by use of supplemental vertical bracing members.

Recommendations for design and installation of the supplemental vertical bracing members are as follows:

- Design bracing members for 0.4 times the tributary URM wall weight.
- Deflection of the bracing member, calculated using recommended forces, should not exceed 0.15 times the wall thickness.
- Horizontal spacing of the vertical bracing members should not exceed one-half the unsupported height of the wall or 10 ft maximum.
- The vertical bracing members should be anchored to the floor or roof framing independently of the recommended wall anchorage system.

8.4 COMPUTATION OF EARTHQUAKE RESPONSE FORCE

The methodology utilizes the concept of building element response. The response of each element excites the next element in the response chain. The summation of the responses of the elements will be given as a base shear. In many cases, the element response shear is given as an upper bound. The upper bound given is the yield shear capacity of the elements.

The recommendations for computation of earthquake response forces are as follows:

- Calculate weight of building as a lumped weight at each floor, mezzanine, and roof level. However, for convenience of computational procedure, tabulate the weight computations as in-plane URM wall weight (W_W) and weight tributary to diaphragms (W_D), at each level, for each axis of analysis of the building.
- For analysis of the connection of the ends of diaphragms to the URM walls, select C_p from Table 8-4. However, the design shear at the end of the diaphragm need not exceed $v_u \cdot D$. Yield capacities of diaphragms, v_u, are given in Table 9-1.
- For analysis of wall-anchorage capacity, use $C_p = 1.0$. See Section 8.2.
- For analysis of shear in each URM shear wall, use $V = 0.4\ W_W + \Sigma_1^n\ 0.4\ V_D$. However, the diaphragm shear at the shear wall at any level need not exceed the yield capacity, $v_u \cdot D$, of the diaphragm at that level.

- The restoring shear capacity, V_R, of any URM shear wall need not exceed $0.2\ W_W + \Sigma_1^n\ 0.2\ V_D$, and the diaphragm shear at the shear wall at any level need not exceed the yield capacity, $v_u \cdot D$, of the diaphragm at that level.

The recommendations of the methodology are based on the seismic response model described in Section 5.1. The URM walls acting as shear walls, in the plane of the wall, excite the ends of the diaphragms. The seismic response of the diaphragms is expressed by the factor, C_p, as noted in Table 8-4. The C_p factors given in that table equal or exceed the seismic zone EPA to account for amplification of input motions that are applied to the ends of the diaphragms. However, the upper bound of response shear that can be coupled with the URM shear walls is the yield capacity of the diaphragm.

TABLE 8-4. RESPONSE FACTOR, C_p, FOR SHEAR CONNECTION OF HORIZONTAL DIAPHRAGM

	C_p
Single layer of boards or plywood with applied roofing	0.45
Double layer of boards or blocked plywood	0.8
Steel decking not detailed for lateral load resistance	0.6
Concrete filled steel decks or concrete framed system with span/depth ratio of 2 or less*	0.4

*Not applicable for span/depth ratio greater than 2

For analysis of the anchorage of URM walls to the diaphragm, a C_p is recommended that is an upper bound of acceleration amplification. This upper bound of amplification is appropriate for diaphragms that have near-elastic response. This near-elastic response can be due to diaphragm stiffness and strength or to a small span/depth ratio.

The sum, V, of element response recommended for the total building is equal to the seismic hazard zone EPA. The response at a URM shear wall is calculated as the hazard zone EPA times the weight of the shear wall and the weight that can be dynamically coupled with the URM shear wall. The effective coupling of the weight tributary to a diaphragm is calculated as EPA times the calculated weight, but the coupling response is limited to the yield capacity of the diaphragm(s) at any level. This procedure is not intended to give a summation of peak element response, but follows usual seismic design procedures for new buildings.

8.5 DISTRIBUTION OF RESPONSE FORCES

The methodology does not recommend a redistribution of the calculated base shear as required by current seismic design provisions. URM shear walls have been modeled as rigid blocks rocking in soils (App. A). This model generally indicates insignificant amplification at the upper stories. If the URM shear walls exceed the common height/length ratios used for the response studies and are founded on soft soils, redistribution computations can be made in accordance with current seismic design recommendations for new buildings (ATC, 1978). See Section 2.3 for additional commentary.

8.6 ANALYSIS OF HORIZONTAL DISPLACEMENT CONTROL ELEMENTS

The methodology recommends analysis procedures for horizontal displacement control elements that are based on dynamic testing and modeling. The procedure is as follows:

- For diaphragms without crosswalls:

  Calculate demand/capacity ratio

$$\frac{W_D}{2v_u \cdot D}$$

W_D = Total weight tributary to diaphragm.

v_u = Yield capacity of diaphragm (see Table 9-1.)

D = Diaphragm depth as defined in this section.

From Figure 8-2, using the appropriate diaphragm yield capacity and span length, determine adequacy of existing diaphragm.

If the existing diaphragm does not meet the span limitations, the diaphragm must be retrofitted to increase v_u or crosswalls may be added to limit relative horizontal displacement.

- For diaphragms with minimum capacity crosswalls:

  Calculate demand/capacity ratio:

$$\frac{W_D}{2v_u \cdot D + \Sigma V_c}$$

  ΣV_c total yield capacity of crosswalls that are spaced not to exceed that specified in Table 8-3.

- If the spacing of existing crosswalls is less than specified in Table 8-3 and the capacity (ΣV_c) exceeds 20% of W_D, the span of the diaphragm is unlimited.

- The interconnection of the diaphragm to the URM shear walls should be calculated in accordance with Section 8.4.

- For multistory buildings, V_c utilized for diaphragm analysis at any upper story should be added to the W_D of the story below for analysis of that story.

This section recommends a departure from existing seismic design procedures. Use of current static analysis procedures for diaphragms does not address the two functions of diaphragms. Horizontal diaphragms couple the weight of the out-of-plane URM walls and the diaphragm weight with the URM shear walls. In addition, the stiffness properties of the horizontal diaphragm control the relative displacement of the center of the diaphragm span with the shear walls (Fig. 8-3). When crosswalls do not exist, the displacement of the point a vs. b is controlled by the dynamic properties of the yielding diaphragm. Static analysis methods cannot predict the relative dynamic displacement of a and b or a and c. Figure 8-2 was developed from data obtained by dynamic testing (ABK, 1981c) and computer modeling (ABK, 1982a). The effective coupling of the tributary diaphragm weight with the shear wall can be expressed only by the response factor given in Table 8-4 for near-elastic response and is limited by the yield capacity v_u. If appropriate coupling yield capacity is given as a static analysis parameter, displacement control capacity is misstated.

If crosswalls as shown in Figure 8-3 are introduced into the displacement control system ⓐ vs. ⓑ and ⓐ vs. ⓒ, crosswall capacity has a significant influence on relative displacements. The methodology recommends a simple technique to estimate an acceptable span for the diaphragm with crosswalls. The crosswalls should be reasonably symmetrically distributed along the diaphragm length. The spacing of crosswalls is limited to that specified in Table 8-3 to achieve a relatively uniform spacing.

If crosswalls are used in a multistory building (Fig. 8-4), a crosswall capacity must be maintained vertically in the building. For analysis, V_c used in Level 3-4 must be added to W_D at Level 3 for the diaphragm analysis at that level.

Diaphragm depth, D, is the depth of the building perpendicular to the analysis direction (Fig. 8-5). Diaphragm span, L, is the diaphragm length between shear walls. If openings occur in the diaphragms, within the depth of the diaphragm as measured from each shear wall (e.g., opening a in Fig. 8-5), a revised depth (D_1 in Fig. 8-5) must be used in the recommended analysis procedure. If the opening occurs in the remainder of the diaphragm (opening b in Fig. 8-5), the full depth D is used in the analysis.

For the special case of horizontal displacement control of an "open-front" building (Fig. 8-6), the recommendation for diaphragms with shear walls at the diaphragm ends may be used (Fig. 8-2). To utilize the table, an equivalent L_1 is calculated. The wall weight, W_W, at the open end is used to calculate L_1:

$$L_1 = 2\left(\frac{W_W}{W_D} \cdot L + L\right)$$

Calculate

$$\frac{W_D + W_W}{v_u \cdot D}$$

Compare demand/capacity ratio of diaphragm with an acceptable span calculated as L_1/D. If acceptable crosswalls exist, calculate:

$$\frac{W_D + W_W}{v_u \cdot D + \Sigma V_c}$$

for entry to Figure 8-2.

This procedure recognizes that shear deforming diaphragms can control the relative displacement of the open fronts of URM buildings. Acceptable displacement control of open fronts by existing diaphragms was observed in a survey of damage caused by a recent earthquake with a ground shaking intensity of 0.6 EPA (EERI, 1983). Similar acceptable performance was also reported (EERI, 1979) from a field investigation of open-front buildings in California that were subjected to moderate to strong intensity ground motions.

8.7 ANALYSIS OF VERTICAL DISPLACEMENT CONTROL ELEMENTS

Vertical displacement control elements include both crosswalls and shear walls. Crosswalls may be existing or retrofitted. If existing crosswalls conform to Table 8-3 for spacing and capacity, no further analysis is required. If crosswalls are retrofitted, the required capacity of the crosswalls is designed in conformance with usual seismic design procedures. Materials capacity for use in design of retrofitted crosswalls is given in Section 10.

Shear walls are existing URM walls or retrofitted structural systems designed to supplement the interstory shear capacity of the existing URM walls. The analysis of URM shear walls uses a double criterion to determine:

- A shear capacity of the masonry walls and piers and to compare this shear capacity with the recommended base shear
- The restoring shear capacity of piers that have flexural cracks at their top and bottom (Fig. 8-7)

The recommended procedure for determination of the shear capacity of a URM wall or pier system is:

- Calculate the total restoring shear capacity of the pier system as

$$V_R = \sum_1^n 0.9 \frac{P_x \cdot D_x}{H_x}$$

where

P_x = Axial load on a pier

D_x = In-plane depth of the pier

H_x = Least height of the pier if the opening height on sides of pier varies.

- Compare the calculated value of V_R with the minimum restoring shear capacity as recommended in Section 8.4.
- Compare the V_R on each pier with the in-plane shear capacity V.
- If the calculated restoring shear capacity of the pier system is less than the recommended V_R, but V_a of each pier exceeds V_R, supplement the restoring shear capacity with structural elements designed in accordance with Section 10.
- If V_R of any pier exceeds the pier shear capacity, distribute the shear wall response shear, V, calculated as recommended in Section 8.4, to the piers, using D/H as the individual pier stiffness.
- Calculate the bed-joint shear v for the stiffest pier as v = 1.5 V/A. See Appendix D for calculation of shear in walls.
- Compare calculated shear with v_a as determined by the URM testing. See Section 9.6 for procedure.
- If the URM wall capacity is less than the recommended V, strengthen the URM wall in accordance with the recommendations of Section 10.

This section recommends that distribution of calculated response shear be made by using D/H as pier stiffness rather than using stiffness computations that are based on moment and shear deflections. Testing of URM piers indicates that shear deformation of the masonry assemblages is a large part of the total elastic deformation. The methodology recommendation is an approximate procedure.

For computation of restoring shear, the stability moment of a fully cracked pier system is used. The restoring shear is computed from the weight on the pier, P, times 0.9 of the in-plane pier depth D. The recommended 0.9 factor is based on in-plane testing of URM piers (App. C).

The recommendations for a minimum restoring shear follow current seismic design requirements. Current seismic design requires response base shears of 0.14 x 1.33. The required resistance capacity is calculated using increased working stress levels. The required base shear resistance calculated at yield is 0.14 x 1.33 x 1.7 ÷ 1.33 or about 0.24 times the weight of the building above the level of calculation. Buildings designed by prior codes have used base shears as low as 0.10 of the building weight and have had adequate inelastic displacement control.

8.8 INTERCONNECTION OF BUILDING ELEMENTS

Interconnection of all elements of the building is recommended. A continuous load path for all calculated forces should be provided. The exception to the recommendation is that the interconnection capacity of existing materials described in Section 9 need not be analyzed. See Section 9.7 for a complete discussion of this exception.

All wall-anchorages must be attached to the existing diaphragms by connections designed in accordance with Section 10. Development of anchorage forces within the existing diaphragm construction is not required. If cross-walls are retrofitted into an existing structure, distribution ties to the diaphragm are recommended. Supplemental resistance or restoring shear capacity that is retrofitted to a URM shear wall should have a designed tie system to the remainder of the wall. Design of the tie system should use assumptions consistent with the recommended analysis procedure.

8.9 REVIEW OF VERTICAL LOAD-CARRYING ELEMENTS

The methodology assumes that many elements of the URM building will have inelastic displacements. This is similar to current seismic design provisions. However, the vertical load-carrying elements in existing URM buildings have generally not been designed and constructed with adequate consideration of probable inelastic displacements. The discussion of Section 6.5 is applicable to buildings in this seismic hazard zone and should be expanded to include a special review of reinforced concrete columns that support discontinuous URM walls. Displacement control of these elements by nailed wood diaphragms should not be used. In-plane vertical displacement control systems should be designed in accordance with the requirements of Section 10 and a special analysis should be made to ascertain that the nonductile columns remain elastic when displaced by yielding of the retrofitted system.

If the support of discontinuous URM walls is provided by structural steel beams and columns, special analyses need not be made. If masonry piers provide support for steel beams at an open front or discontinuous interior URM wall, and an in-plane displacement control system is not retrofitted into the plane of the URM wall, steel columns should be added at the masonry pier face. These columns act as shoring in the event of a partial support failure. Design of an independent foundation system is not required. Similar independent supports should be considered for steel beams framing into a URM wall when diaphragm displacement control is critical for the diaphragm level containing the steel beams.

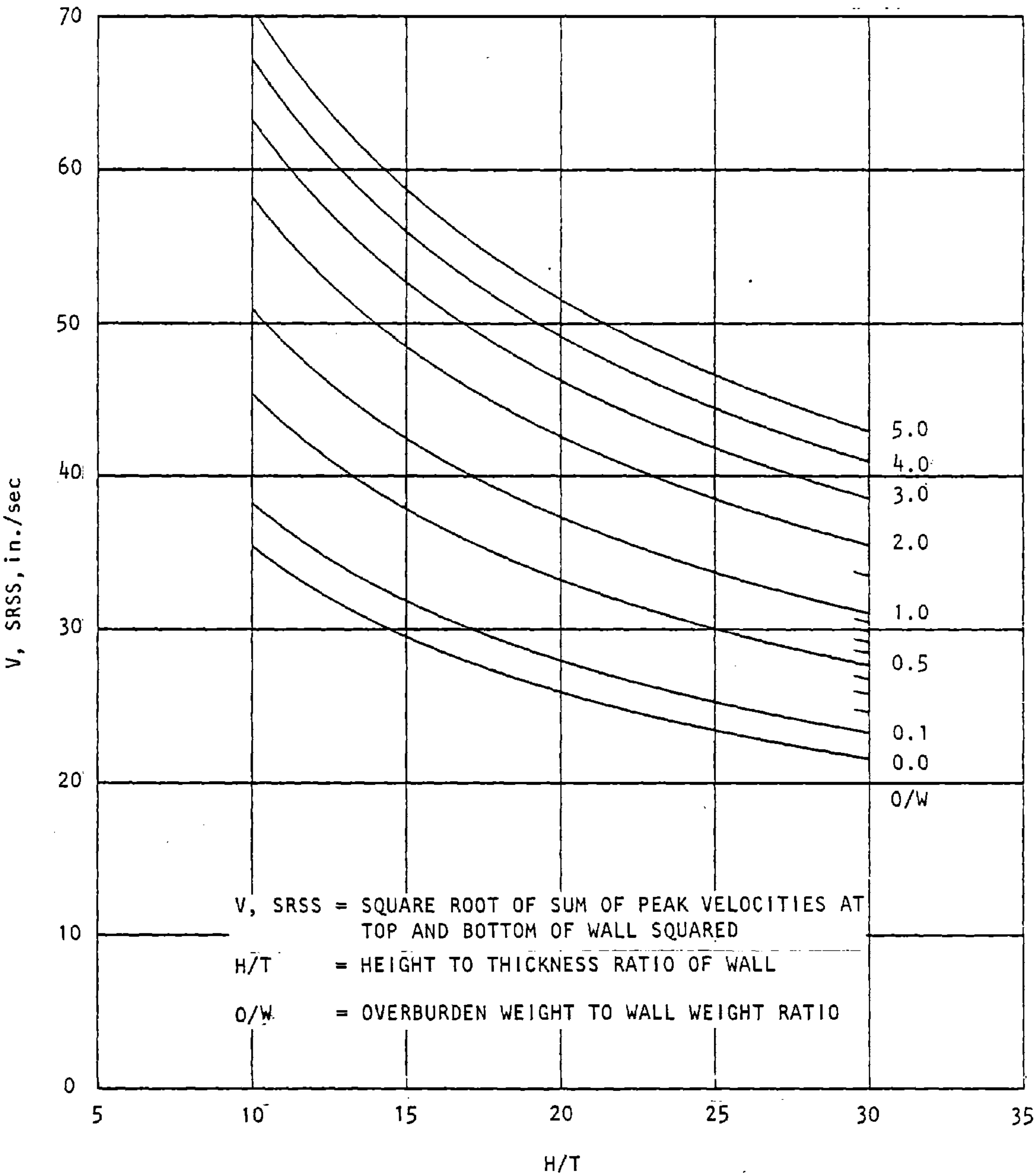


FIGURE 8-1. UNREINFORCED MASONRY WALL STABILITY CRITERIA, 98% PROBABILITY OF SURVIVAL

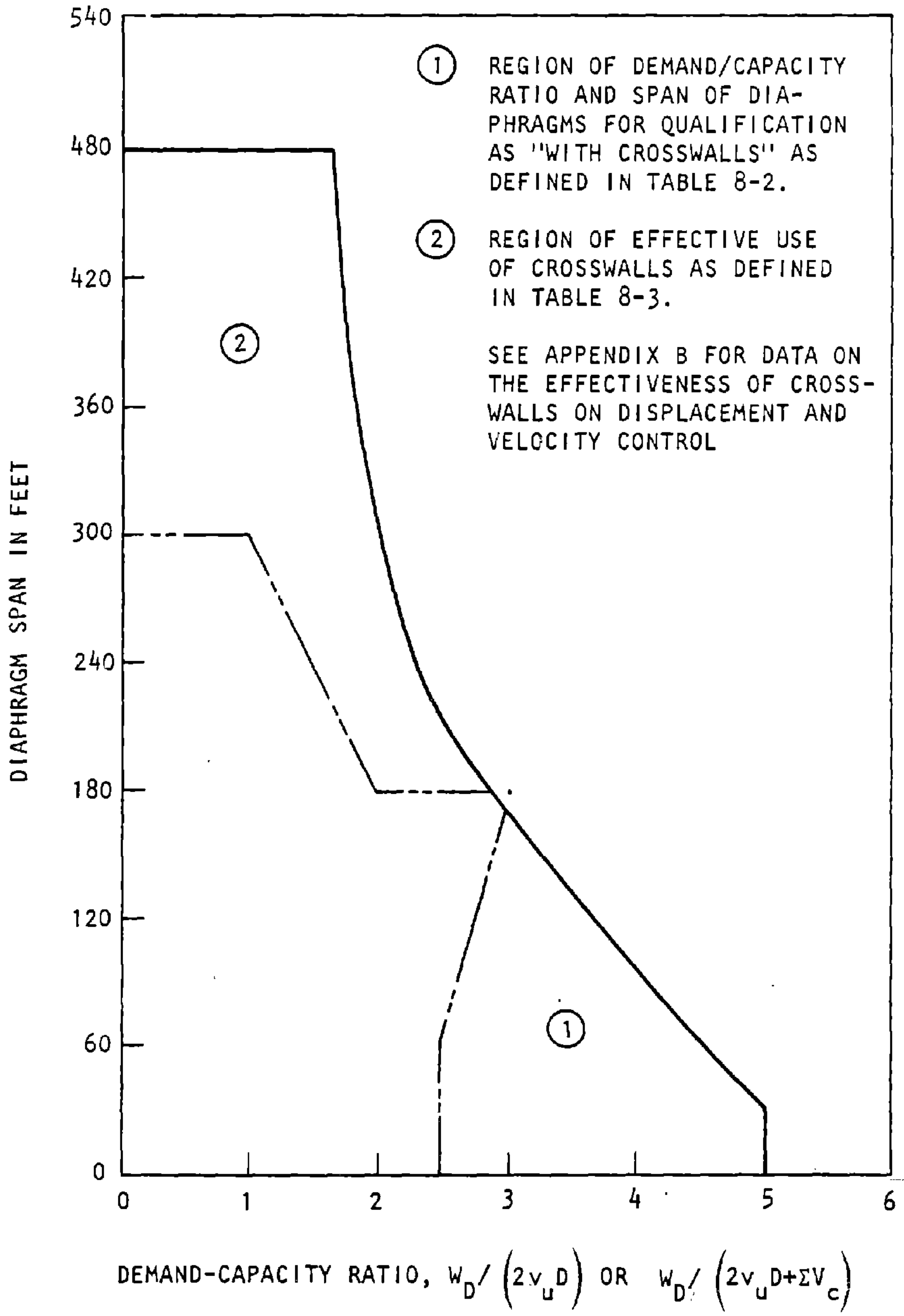


FIGURE 8-2. ACCEPTABLE SPAN FOR DIAPHRAGMS
(BASED ON DISPLACEMENT CONTROL CONCEPTS)

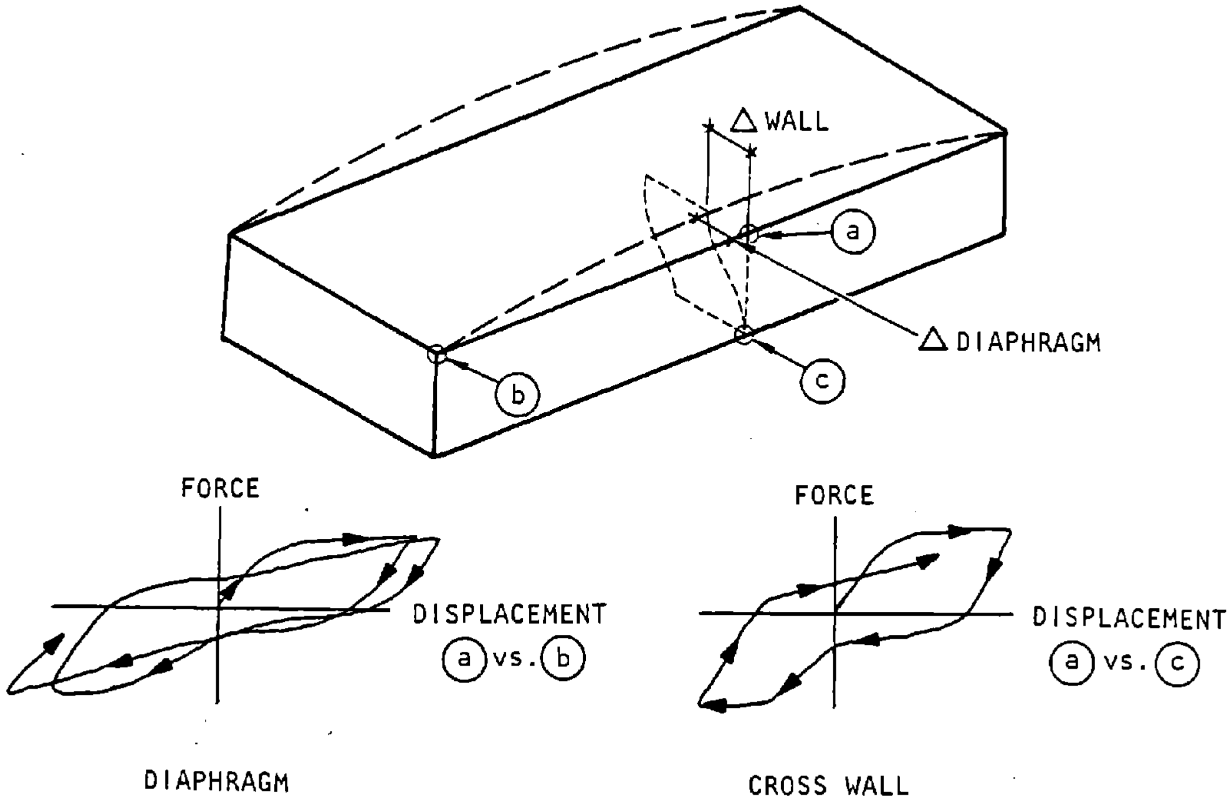


FIGURE 8-3. SINGLE STORY RESPONSE MODEL

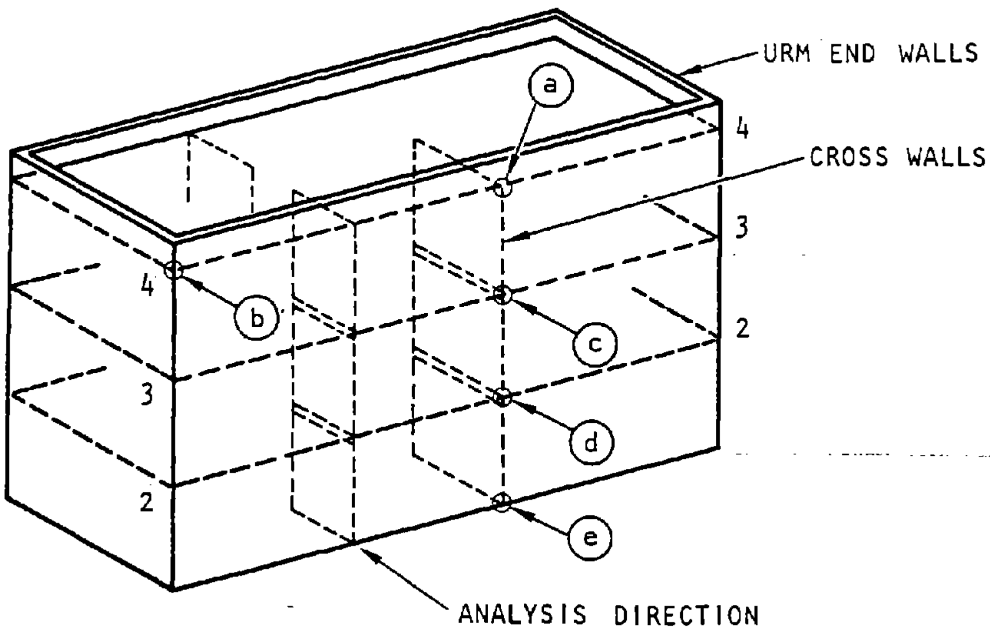


FIGURE 8-4. MULTISTORY RESPONSE MODEL

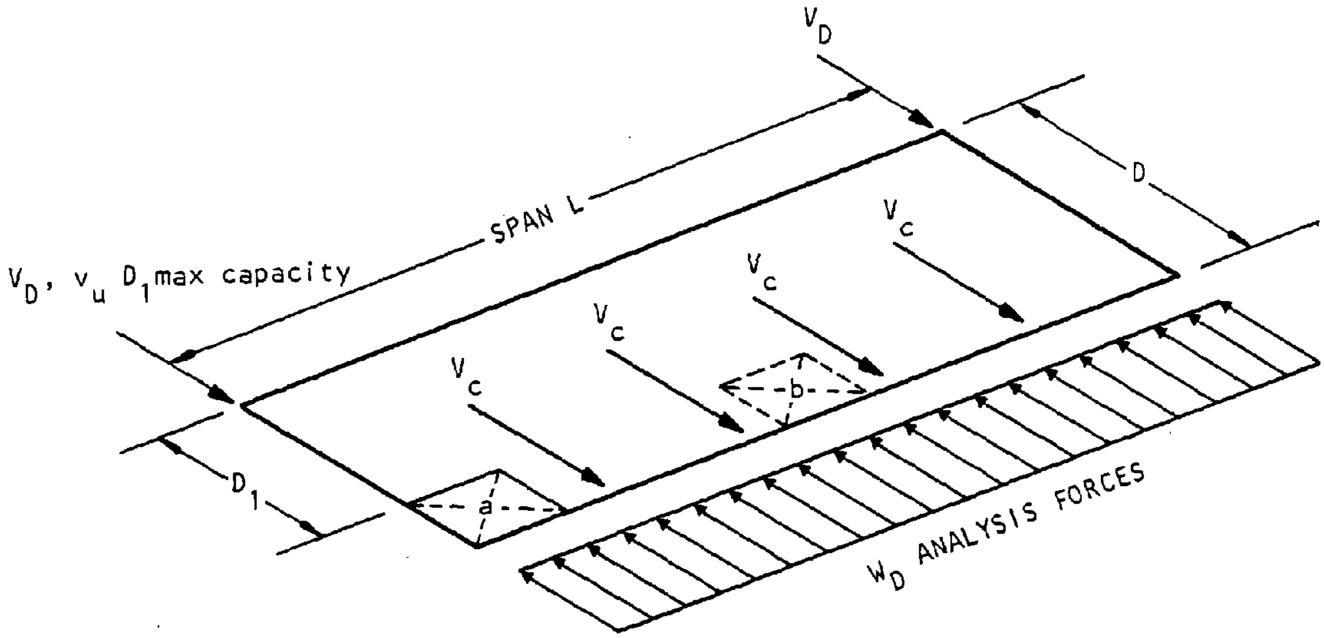


FIGURE 8-5. MODEL OF DIAPHRAGM WITH OPENINGS

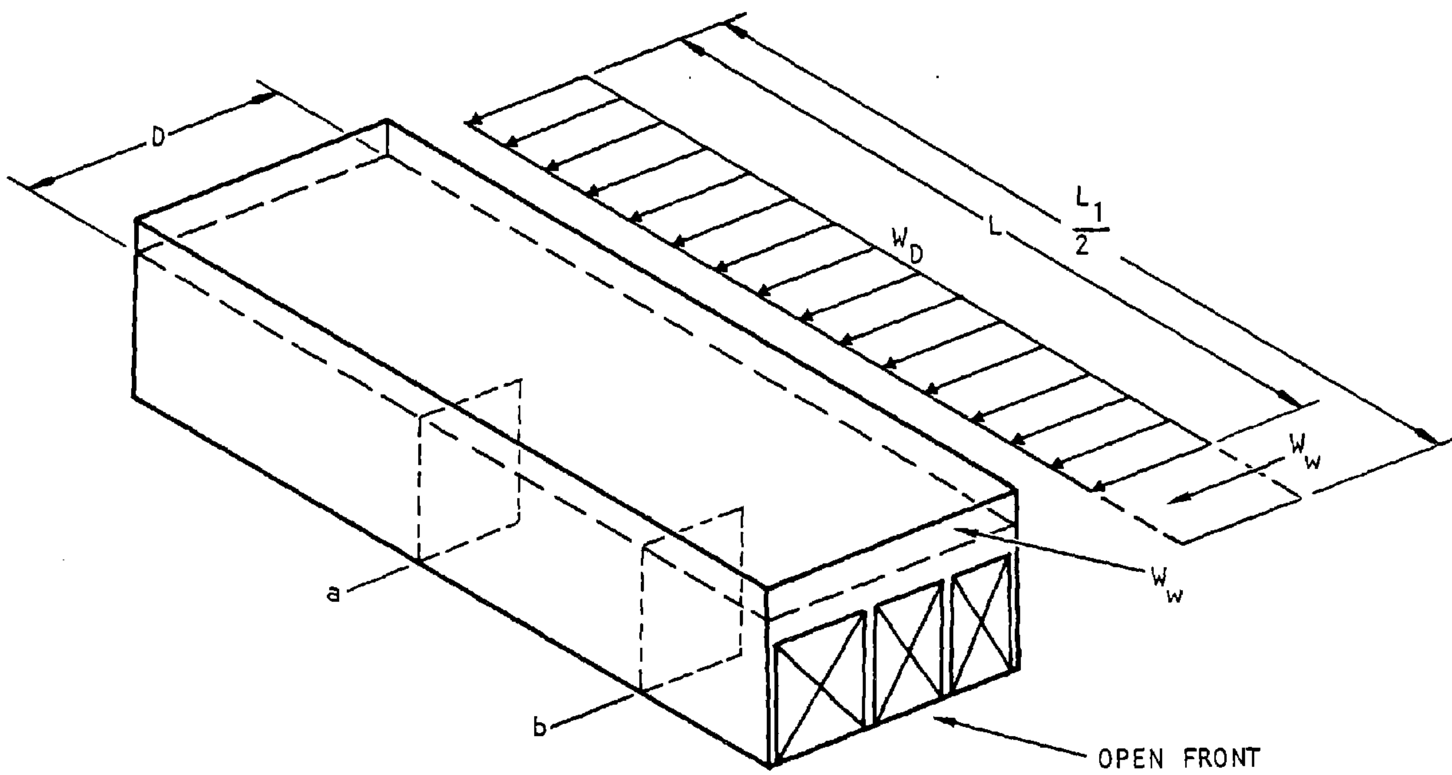
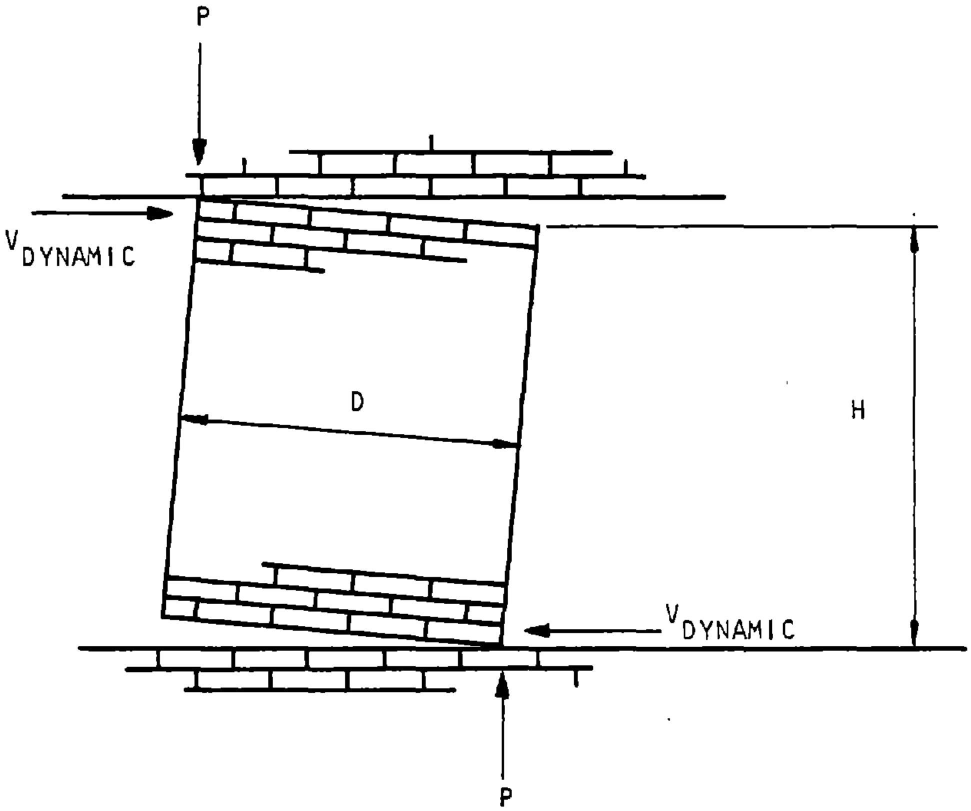


FIGURE 8-6. RESPONSE MODEL OF OPEN FRONT BUILDING



$$P \cdot 0.9D = V \cdot H$$

$$V_{DYNAMIC} \sim \frac{0.9P}{H/D}$$

FIGURE 8-7. RESPONSE MODEL FOR A FLEXURALLY CRACKED PIER

SECTION 9

CAPACITY OF EXISTING URM BUILDING ELEMENTS

9.1 PHILOSOPHY

The seismic resistance capacity of existing materials is given as the yield capacity that is maintained at large inelastic displacements. An analogy is the yield capacity of ductile systems. Recommended capacities of elements that have a deteriorating load capacity after attaining a peak capacity are reduced as described in Section 4.5.1.

The resistance capacities given in this section have been determined by static testing. Development of the methodology used static testing to validate a computer model that can adequately replicate dynamic testing (ABK, 1982a). If capacity of existing materials other than those generically described here is required, a static test program can provide the required data. The resistance capacities listed in this section are recommended for use in all seismic hazard zones.

9.2 OBJECTIVES

The objective of the methodology for mitigation of seismic hazard is reduction of life-safety risks. This objective is attained by minimizing property damage and the risk of separation of parts of the URM walls. Displacement of interior partitioning, floors, and ceiling into the inelastic range will cause cracking of finish materials. This methodology recognizes this behavior and is not intended to prevent building damage during moderate to strong ground shaking.

9.3 EXISTING FOUNDATIONS

Existing foundations are considered to have adequate capacity to resist seismic loadings if foundation settlement is not observed in the building survey. Corrective work to stop discovered foundation settlement should be designed without consideration of the probability of seismic loading.

If supplemental structural materials need to be added to URM walls, the calculated foundation loading may be increased by 25% for added dead loads.

If supplemental structural systems are retrofitted into the building to increase restoring shear capacity or to function as crosswalls, a designed foundation system should be provided. See Section 10.3 for design recommendations.

9.4 EXISTING WALL ANCHORAGES

Capacity of ends of existing wall anchorages embedded in URM walls should be determined by testing as described in Section 4.5.4. Criteria for determination of design capacity are related to the failure mode of destructive testing and are described in Section 4.5.1. Determine capacity of the anchor attachment to existing wood framing in accordance with Section 10.8. Determine capacity of the anchor in accordance with Table 9-3.

9.5 EXISTING HORIZONTAL ELEMENTS

All existing floors, roofs, and mezzanines are used in the application of this methodology as horizontal displacement control elements. The capacity of these elements is the yield shear capacity as determined by static and dynamic testing (ABK, 1981c; 1982a).

The capacities given in this methodology for assemblages not tested by ABK are taken from static testing by others. Recommendations for yield capacity of common existing assemblages are shown in Table 9-1.

9.6 EXISTING VERTICAL ELEMENTS

Existing vertical elements that control interstory displacement are crosswalls, URM shear walls, or other existing materials such as reinforced concrete and structural steel framing. Capacities of common construction of sheathed walls are given in Table 9-2. Procedures for calculation of capacities of structural systems are described in this section. The procedure for determination of shear capacity of URM walls is given in this section.

TABLE 9-1. YIELD CAPACITIES, v_u, OF EXISTING ROOF AND FLOOR CONSTRUCTION

Description of Existing Construction	Yield Capacity of Materials v_u in lb/ft, shear
Straight sheathing with roofing applied on the sheathing or a single layer of tongue and groove sheathing without roofing	300
Straight sheathing with plywood overlay	650
Unblocked plywood sheathing with roofing applied on the sheathing	400
Diagonal sheathing with roofing applied on the sheathing	750
Plywood sheathed floors or roofs with blocking at panel edges	2-1/2 x shear values listed in design codes such as UBC Table 25-J or SBCC Supplement to Chapter XVII
Double board systems with finish flooring laid over diagonal sheathing or multiple board systems with board edges offset	1800
Metal roof deck systems designed for minimal lateral load capacity	1800
Metal roof deck systems designed for lateral load capacity	3000
Concrete filled steel decks	As determined by static yield capacity testing
Concrete framed floors	Concept of v_u is not applicable

TABLE 9-2. YIELD CAPACITIES OF EXISTING VERTICAL ELEMENTS USED AS CROSSWALLS

Description of Existing Construction	Yield Capacity of Materials in lb/ft shear
Plaster on wood or metal lath	900
Plaster on gypsum lath	550
Gypsum wall board, unblocked edges	200
Gypsum wall board, blocked edges	400
Plywood sheathing applied to studding	4 x shear values listed in design codes

Capacity of all materials on crosswalls may be combined, except that the total combined capacity used for any existing crosswall should be limited to 1300 lb/ft shear.

TABLE 9-3. YIELD CAPACITIES OF EXISTING STRUCTURAL SYSTEMS

Description of Existing Construction	Yield Capacity of Materials
Structural steel framing	Yield strength of 33,000 psi unless tested
Reinforced concrete walls	Yield capacity calculated in accordance with ACI 318 using load factor of 1.0
Reinforced concrete frame without ductile detailing in accordance with Appendix A, ACI 318	Yield concept is not applicable
URM shear walls	$0.9 \frac{P \cdot D}{H}$ (see Sec. 8.7)

Shear capacity of URM walls and piers shall be determined as follows. Unreinforced masonry walls shall be tested in accordance with Section 4.5.6. Tested shear values shall be reduced to a common shear value by deducting the existing axial dead load stress from the tested shear value. The basic shear, v_t, shall be determined from the reduced tested shear by selecting a shear equal to the tested value that has 20% of test values lower and 80% of the test values higher (Fig. 9-1). The allowable shear, v_a, in any URM pier or wall shall be calculated as $v_a = 3/4\ (3/4\ v_t + P/A)$. See Appendix D for commentary.

The yield capacity of vertical elements, Table 9-2, was developed from static testing and reviews of reported research. The yield capacity of nailed wall sheathing was developed from a review of several sources (APA, 1976; FPL, 1958, and Anderson, 1981). The yield capacities of the materials were reviewed and compared with design code values where possible. The factor utilized for modification of code values to yield capacities was developed by reference to yield capacity testing.

Yield capacities of existing structural systems not listed should be determined by analysis of static testing. Structural systems that do not have acceptable inelastic performance should be evaluated by a special analysis that considers their limited ductile capacity.

Yield restoring shear capacities for URM shear walls that are cracked pier systems are given in Table 9-3. See Section 8.7 for additional commentary. Recommended shear capacity of URM walls and piers was calculated from values determined by testing. Testing of existing URM piers (App. C) and finite element modeling of isotropic piers (App. D) have provided data for these values. The authors recognize that the recommended in-place shear testing does not represent the state of stress in a URM pier subjected to dynamic shear. It is also recognized that the finite element studies do not represent the jointed masonry wall.

The method of testing recommended typically results in a wide scatter of in-plane shear values. This wide scatter can be attributed to wide variation in the cohesion between the brick and the mortar, lack of mortar in the collar joint between wythes, and other flaws that may be associated with the original workmanship. To utilize existing URM as a shear element, a minimum shear capacity, as recommended in Section 4.5.6, is recommended. The testing described in that section is a qualification testing. If the minimum corrected shear value does not meet that criterion, these recommended shear values may not be used and the existing URM may not be considered as part of the structural system.

The in-plane shear tests have been correlated with prior testing of existing URM walls in the Los Angeles area (Schmid et al., 1978, and App. C). The test value is reduced to an equivalent zero axial stress. The shear failure of URM piers subjected to dynamic forces can be related to a diagonal compression test. The shear capacity is strongly influenced by cohesion of the mortar to the brick; however, the shear failure of the pier propagates from a flaw that may occur at random in the pier.

For these reasons it is recommended that the 20 percentile of the lowest values be used for a basic shear stress. This basic shear stress is further reduced by a 3/4 factor to account for possible bonding in the tested brick on the collar joint. The combined shear and axial stress is further reduced by a 3/4 factor to correlate with tested gross shear values that were obtained by diagonal compression testing. Additional conservatism is not recommended, as observed shear offsets in URM buildings shaken by strong ground shaking (EERI, 1983) do not have a significant probability of causing life-threatening damage.

9.7 EXISTING INTERCONNECTION OF ELEMENTS

Analysis of interconnection of existing elements is not recommended. The methodology recommends retrofitting interconnections in seismic hazard zone of EPA = 0.4 g and in special cases in the seismic hazard zone of EPA = 0.2 g. An exception is given for wood framed existing crosswalls. Interconnection of these materials is provided by usual framing techniques and an investigation of the interconnection would irreparably damage the integrity of the finish materials that make the interconnection. As a judgment, the combined value of sheathed existing crosswalls is limited to a yield capacity of 1300 lb/ft (Table 9-2). This limitation is not applicable to retrofitted systems.

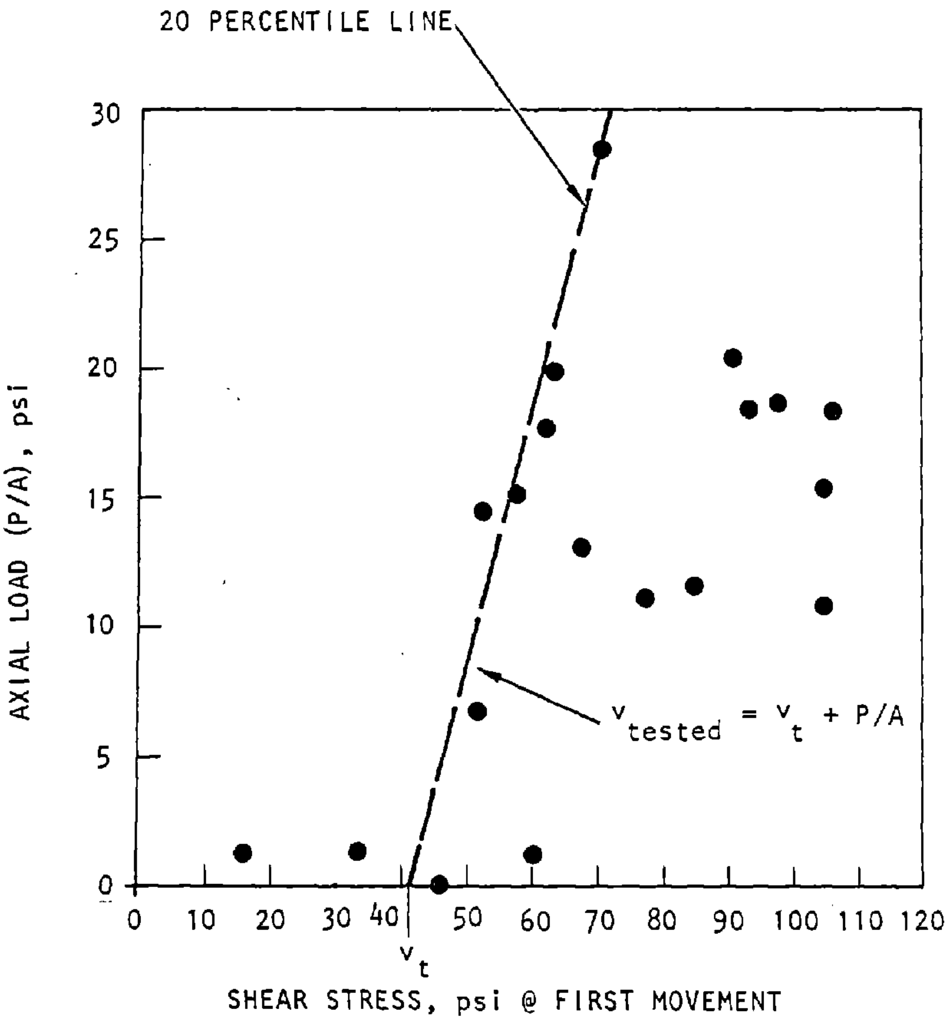


FIGURE 9-1. PROCEDURE FOR PLOTTING OF URM IN-PLACE SHEAR TESTS

SECTION 10

DESIGN OF STRUCTURAL ELEMENTS USED IN CONJUNCTION WITH EXISTING MATERIALS

10.1 PHILOSOPHY

Section 9.1 describes the philosophy used to develop recommended capacities of existing materials. This section uses the same philosophy to match the performance of designed supplemental materials to the yield concepts recommended for analysis of existing materials.

10.2 OBJECTIVES

The objectives of Section 9.2 are appropriate for the design of supplemental structural elements. The materials are not intended to replace the existing material but to provide additional capacity. For this reason, it is recommended that all supplemental materials be designed at their yield capacity.

10.3 FOUNDATION

Foundations designed for supplemental structural systems should be designed by usual seismic design provisions for new buildings. However, the methodology recommends use of structural response forces for analysis of the structural systems that are equated to the yield capacities of the structural materials. Typical seismic design provisions express design earthquake response by use of an inelastic spectra that is equated to a working stress design level. The soils consultant should give special consideration to this difference when preparing a recommendation for soils bearing values. In lieu of the development of special studies, a factoring of foundation loadings by 0.7 is recommended by the methodology.

10.4 ANCHORAGE SYSTEMS

Anchorage systems are generally constructed of structural steel, and are attached to the URM and the wood framing. Design requirements for each of these parts of the anchorage system are given in the appropriate materials section. Anchorage attachment to URM walls should be determined by destructive testing in accordance with Sections 4.5.1 and 4.5.3. However, if the anchorage to the URM walls is by through bolting, with a bearing plate on the exterior surface of the URM wall, destructive testing is not recommended. A recommended capacity for a through wall anchorage system in three wythe URM with a 20 sq. in. bearing plate on the wall surface is 6000 lb. This capacity is based on an estimate of the punching shear capacity of the URM. All other parts of the anchorage should be designed in accordance with the recommendations of Section 10. The spacing of wall anchors should not exceed 6 ft (1.8 m) when anchored in walls 12 in. (305 mm) thick or 4 ft (1.2 m) when anchored in walls less than 12 in thick. Parapet heights greater than 1-1/2 times the wall thickness above the anchorage level should be braced as recommended in Sections 6.2, 7.2, and 8.2.

The recommended minimum distance from a wall anchor to the upper edge of the URM wall is 12 in. (305 mm) unless a reinforced bond beam is at the top of the wall. In that case, the minimum distance to the top of the bond beam may be 6 in.

10.5 MASONRY

Masonry elements designed as supplemental shear elements in URM buildings should be reinforced. To reduce the response forces given in the methodology to usual working stress design, a reduction factor of 0.7 times the calculated force is recommended. This reduction factor anticipates that usual working stress capacities for masonry are increased by 1.33 for seismic design.

10.6 REINFORCED CONCRETE

Reinforced concrete systems should be designed by provisions such as ACI 318 or current code design requirements. For correlation with the recommendation of the methodology, a load factor of 1.0 should be used for calculation of dead, live, and seismic forces. See the reference (ATC, 1978) used for seismic hazard zoning for additional commentary. All concrete frames used for supplemental seismic load resistance should meet the construction requirements of ductile moment frames.

10.7 STRUCTURAL STEEL

Structural steel systems should be designed in accordance with current AISC recommendations or current code design requirements using a reduction factor of 0.8 for design loads recommended by the methodology. The reference used for seismic hazard zoning (ATC, 1978) provides an alternate method.

10.8 WOOD FRAMED ASSEMBLIES

The forces recommended in the methodology may be correlated with recommended practice manuals such as the National Design Specification for Wood Construction or current design codes by use of a multiplier of the capacities given in those requirements or recommendations. The recommended multiplier for connection design is 2-1/2 for combinations of dead, live, and earthquake forces or for earthquake forces acting alone. The recommended multiplier for member design is 2.

Recommended yield capacities of plywood sheathed assemblies are 2-1/2 to 4 times shear values listed in current requirements (Tables 9-1 and 9-2). Capacities of nailed sheathing systems used for crosswalls are given in Table 9-2. The combined shear capacity limitation given in Section 9.6 for existing elements is not applicable for designed wood framed assemblies retrofitted to existing buildings. Interconnection of retrofitted assemblies is designed using the recommendations of this section.

SECTION 11

CONCLUSIONS

11.1 COMPARISON OF THE METHODOLOGY AND CURRENT SEISMIC HAZARD REDUCTION REQUIREMENTS

Seismic hazard mitigation programs have been required for some time on the Pacific Coast. The requirements used for these programs generally attempted to apply concepts used for design of new buildings to existing URM buildings. The reconstruction of URM public school buildings in California has been a continuing program, and the reconstructed buildings have been shaken by moderate intensity ground shaking. The structural performance has been good but the reconstruction cost has been high.

An ordinance using concepts of this methodology is now in effect in certain cities in southern California. This section will compare the methodology with this ordinance. The writing of the methodology was completed after writing of the ordinance, and the comparisons generally will discuss concepts that have been recently refined and qualified.

The general differences are:

- The methodology uses EPA as a description of ground motions.
- The seismic hazard map recommended has a significant variation from current seismic zoning maps.
- The methodology describes recommendations for seismic hazard mitigation by hazard zone rather than using a seismic zone reduction factor.
- The methodology describes the structural response at a realistic inertial force level and recommends use of materials resistance capacity at yield level.

These general differences have little effect on a comparison with current hazard reduction requirements since the current requirements are only applicable in seismic zone 4 and this seismic zone is closely correlated with the seismic hazard zone described in the methodology of EPA = 0.4 g. Specific differences that have been developed since the writing of the ordinances are:

- The methodology does not recommend static force analysis methods for diaphragms.
- The methodology refines and quantifies the requirements for design of crosswalls.
- The methodology provides an analysis method for control of displacement at "open fronts" of URM buildings that may not require the introduction of a shear wall at the open front.
- The methodology recommends a significantly different method of determining the in-plane shear capacity of URM walls.
- The methodology recommends that an in-plane shear capacity for URM shear walls be calculated independently from a restoring shear capacity and recommends a significantly smaller restoring shear capacity.

The authors of the methodology have participated in the development and updating of current earthquake hazard reduction ordinances. The ordinances are continually undergoing change to improve the hazard reduction requirements and increase their cost-benefit. The procedures for testing of URM embedments and wall anchors were taken from the current ordinances. Participation on the Ad-Hoc Committee for development, review, and updating of Southern California earthquake hazard reduction ordinances has aided in the development of this methodology.

11.2 APPLICATION OF SEISMIC HAZARD METHODOLOGY TO CURRENT SEISMIC DESIGN RECOMMENDATIONS FOR NEW BUILDINGS

The research to develop the methodology was strongly oriented to quantify the response of diaphragms to ground motions. This facet of the research has the greatest importance for modification of current seismic design requirements.

Many low-rise buildings designed by current design requirements are closely related to the typical URM building, except that the URM building may have a significant quantity of effective crosswalls. The currently designed low-rise building will probably have no effective crosswalls. Diaphragm design by current design methods requires a static design analysis, and this analysis procedure greatly overstates diaphragm strength requirements. Near elastic behavior of these diaphragms increases wall anchorage forces to levels that have caused separation of concrete and masonry walls in recent earthquakes. The diaphragm research developed for the methodology should be considered for modification of current seismic design recommendations.

SECTION 12

RESEARCH RECOMMENDATIONS

The research work involved in developing this methodology included analytical and experimental investigations as well as observations of the response of URM buildings in past earthquakes. Additional research is recommended in both the analytical and experimental areas, as outlined in the following paragraphs.

The dynamic component tests on full-scale URM walls subjected to out-of-plane motions were somewhat simplified from the more complex response of a building wall subjected to three-dimensional motions of an earthquake. Additional testing and/or analyses should be considered to define the significance of combining in-plane and out-of-plane motions in the URM walls. The in-plane motions could be induced by vertical earthquake components or by the horizontal response of the URM wall.

Additional research is needed to determine cost-effective retrofit methods for URM walls subjected to out-of-plane motions. Systems employing simple construction procedures that provide desirable inelastic behavior should be investigated. Also, systems that provide stabilizing forces to supplement the restoring gravity moments in the wall can be investigated. The stiffness and strength requirements of these systems need to be developed.

The full-scale, dynamic and static diaphragm tests need to be supplemented by additional static tests. Future static tests should be performed with full-cyclic loading so that the complete hysteretic behavior can be recorded. Tests should be conducted to confirm the effects of varying aspect ratios, and diaphragms deeper than 20 ft should be tested. In addition, the effects of ceilings on floor and roof diaphragm response need to be investigated. This would be of greater significance with the more flexible diaphragm systems.

Additional analyses of complete buildings need to be conducted using the data and methodologies developed in this research. Multistory buildings should be included, as well as buildings with crosswalls. There is a need to develop a larger data base for buildings with a wide range of span/depth ratios and demand/capacity ratios. These analyses should identify the range of parameters that result in velocity and deformation amplification.

Analyses and/or tests need to be conducted on URM piers of varying aspect ratios and construction types. This information is needed to develop simplified procedures for determining the failure mode (i.e., flexural or shear) of these piers.

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APPENDIX A

SEISMIC RESPONSE MODEL OF A RIGID BLOCK ON SOIL

APPENDIX A

SEISMIC RESPONSE MODEL FOR A RIGID URM END WALL

During an earthquake the ground motion is transmitted from the building/ foundation interface through the end walls (in-plane response) to the floor and/or roof diaphragms, and the diaphragms drive the URM side walls in the out-of-plane direction. Accordingly, the in-plane response of the URM end walls directly influences the kinematic environment delivered to the ends of the diaphragms and to the side walls.

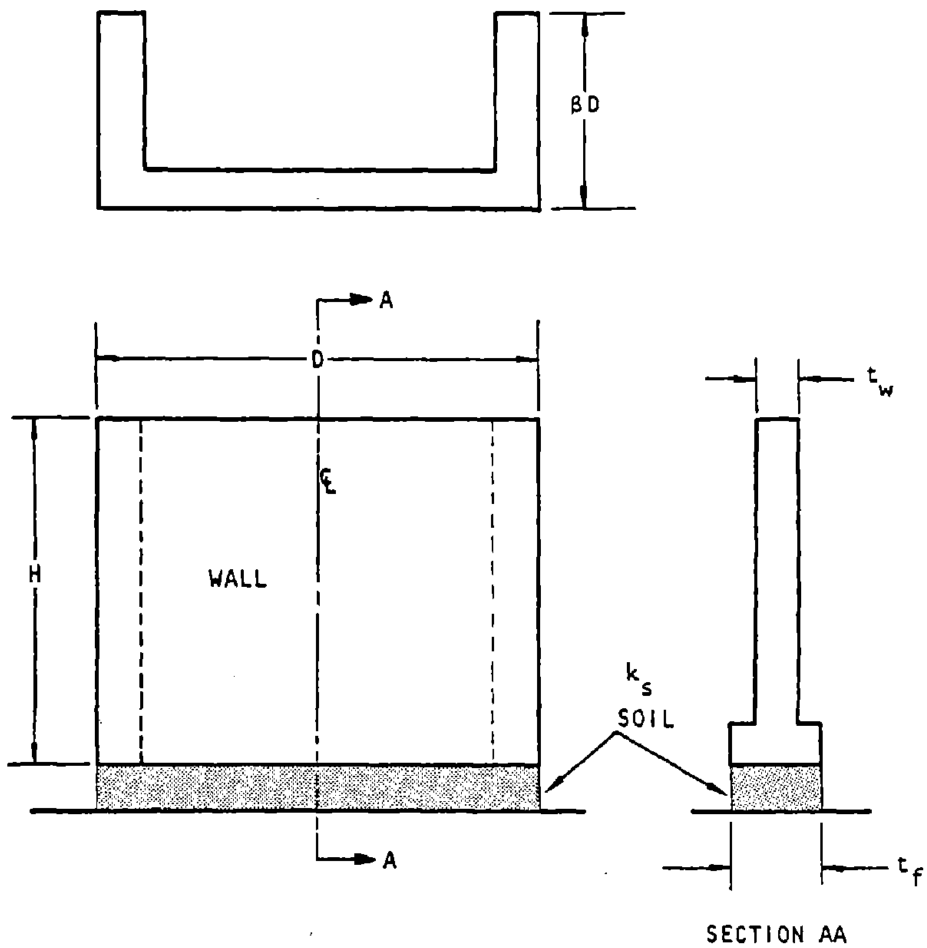
It is clear that a rigid URM end wall will deliver larger motions to the diaphragms than flexible and ductile end walls. An analytical study of the response of rigid end walls of varying aspect ratios resting on a wide range of soil foundations was conducted. The URM end wall considered in the analysis is shown in Figure A-1. The URM end wall is modeled as a rigid block with participating side walls (i.e., βD) resting on a soil. The lumped parameter analytical model of the wall is shown in Figure A-2.

The soil is represented by 10 compression springs (springs 1 through 10 in Figure A-2) that have bilinear, inelastic characteristics and carry only compression loads, as shown in Figure A-3. In addition, impact dampers (springs 11 through 20 in Figure A-2) are included in parallel with the soil springs to provide impact damping, if the wall separates and recontacts with the soil. Spring 21 is a rotational spring and is included to account for the gravity component term in the equations of motion (i.e., $- WH/2\ \theta$). The input is the kinematic earthquake motion applied at degree-of-freedom 1. The analyses were accomplished using the STARS/III computer code (AA, 1981b).

The input to the model was the 1940 El Centro S00E component scaled to an EPA of 0.4 g (1.25 scale factor) as shown in Figure A-4 and to an EPA of 0.2 g as shown in Figure A-6. The calculations were performed using a building width, D, of 60 feet and a 33% participating side wall mass ($\beta = 1/3$). The greatest displacement amplification at the top of the end wall occurs when the softest soil is used (k_s = 100 psi/in.). For an EPA of 0.4 g, H/D = 1.5, and k_s = 100, the displacement amplification at the top of the end wall is 4% (Figure A-5). For an EPA Of 0.2 g, H/D = 1.5, and k_s = 100, the displacement amplification at the top of the end wall is 7% (Figure A-7). The amplification for the stiffer soils is less than those given above, as is the amplification for smaller aspect ratios, H/D.

It should be clear that these results are conservative for at least two reasons. First, a rigid end wall will amplify the motions more than a flexible and ductile wall; second, for very soft soils, the earthquake motion intensity applied at the base of the wall cannot be as high as that for very stiff soils, since soft soils cannot completely transmit the high intensity and high frequency motions.

The results show that over a realistic range of building and soil characteristics the ground motion is transmitted through the end walls with little amplification. Accordingly, in actual URM buildings it is sufficient to assume that the ground motion is transmitted unmodified through the end walls (in-plane) and is directly transmitted to the diaphragms. However, buildings with high aspect ratios (H/D) sited on soft soils will require special analyses.



H = WALL HEIGHT

D = WALL WIDTH

t_w = WALL THICKNESS

t_f = FOUNDATION WIDTH

k_s = SOIL STIFFNESS

β = FACTOR RELATING THE AMOUNT OF PARTICIPATING SIDEWALLS

FIGURE A-1. MASONRY WALL AND SUPPORTING FOUNDATION

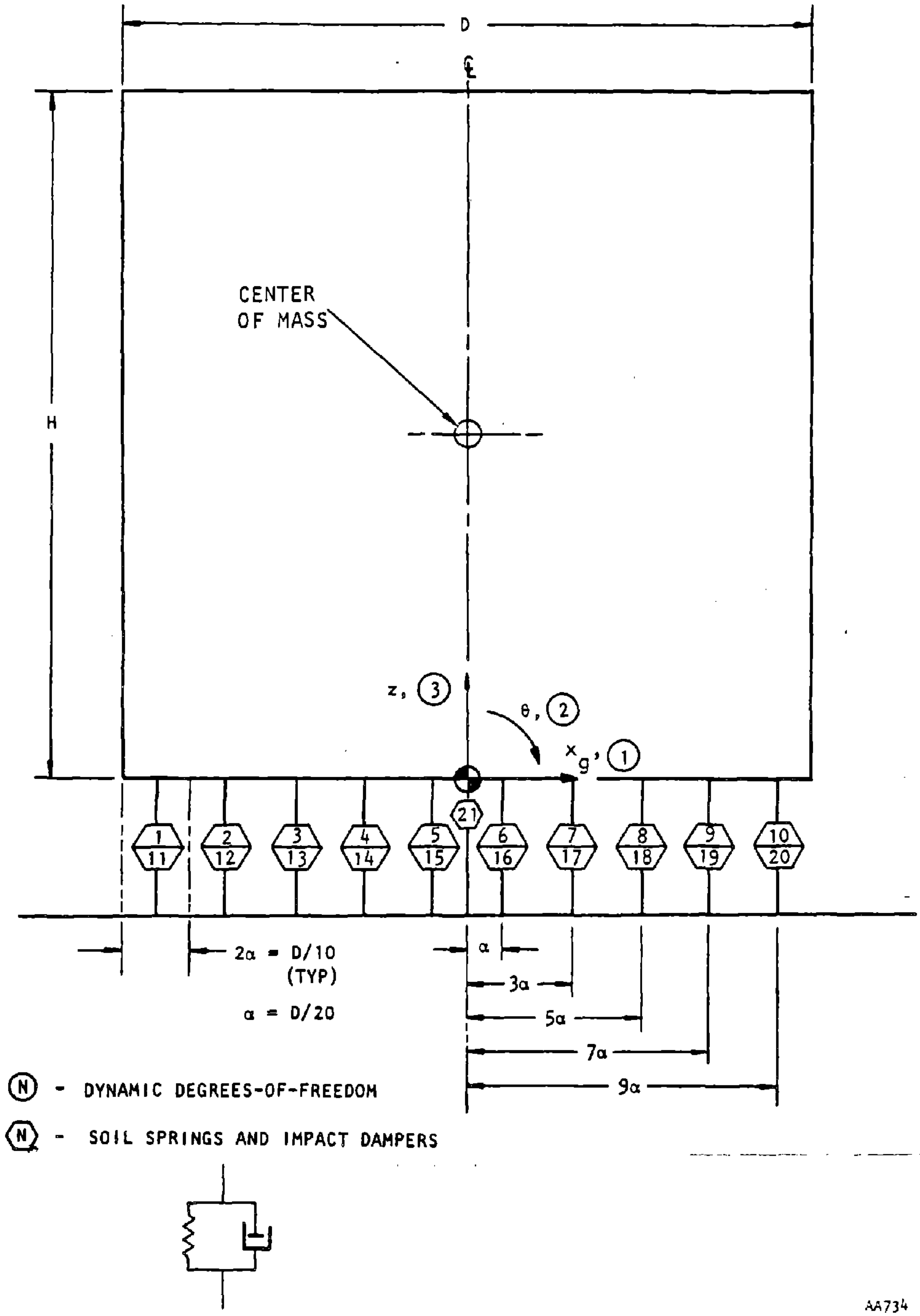
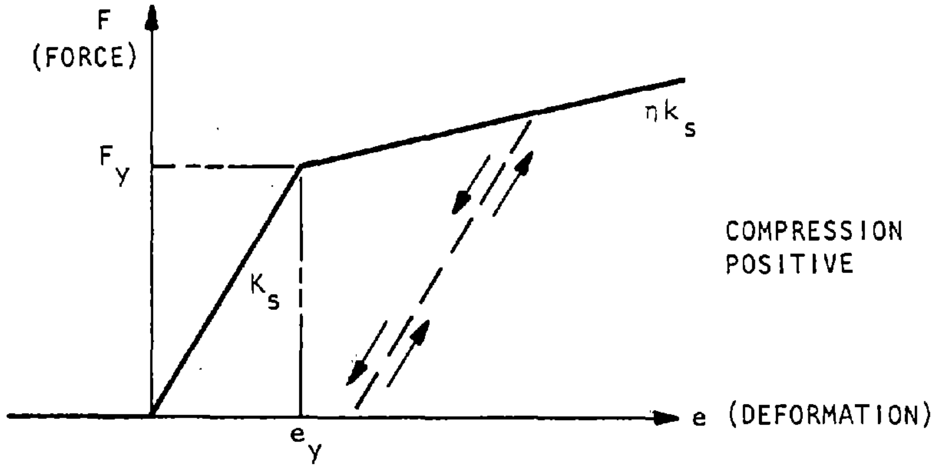


FIGURE A-2. LUMPED PARAMETER MODEL FOR ROCKING ANALYSIS



Bilinear Hysteretic Spring (Compression Only)

k_s = Soil compression stiffness, ksi/in.

f_b = Soil bearing stress, ksf

e_y = Soil yield deformation, in.

F_y = Soil yield force, kips

η = Post yield stiffness factor

K_s = Soil stiffness, kips/in.

δ_{ST} = Static deformation of soil, in.

Soil Type	k_s ksi/in.	f_b ksf	η	e_y/δ_{ST}
Soft	0.1	1	0.5	2.5
Medium	0.4	2	0.25	3→4
Hard	0.8	3→4	0.0	4→10

FIGURE A-3. SOIL SPRING CHARACTERISTICS FOR WALL ROCKING ANALYSIS

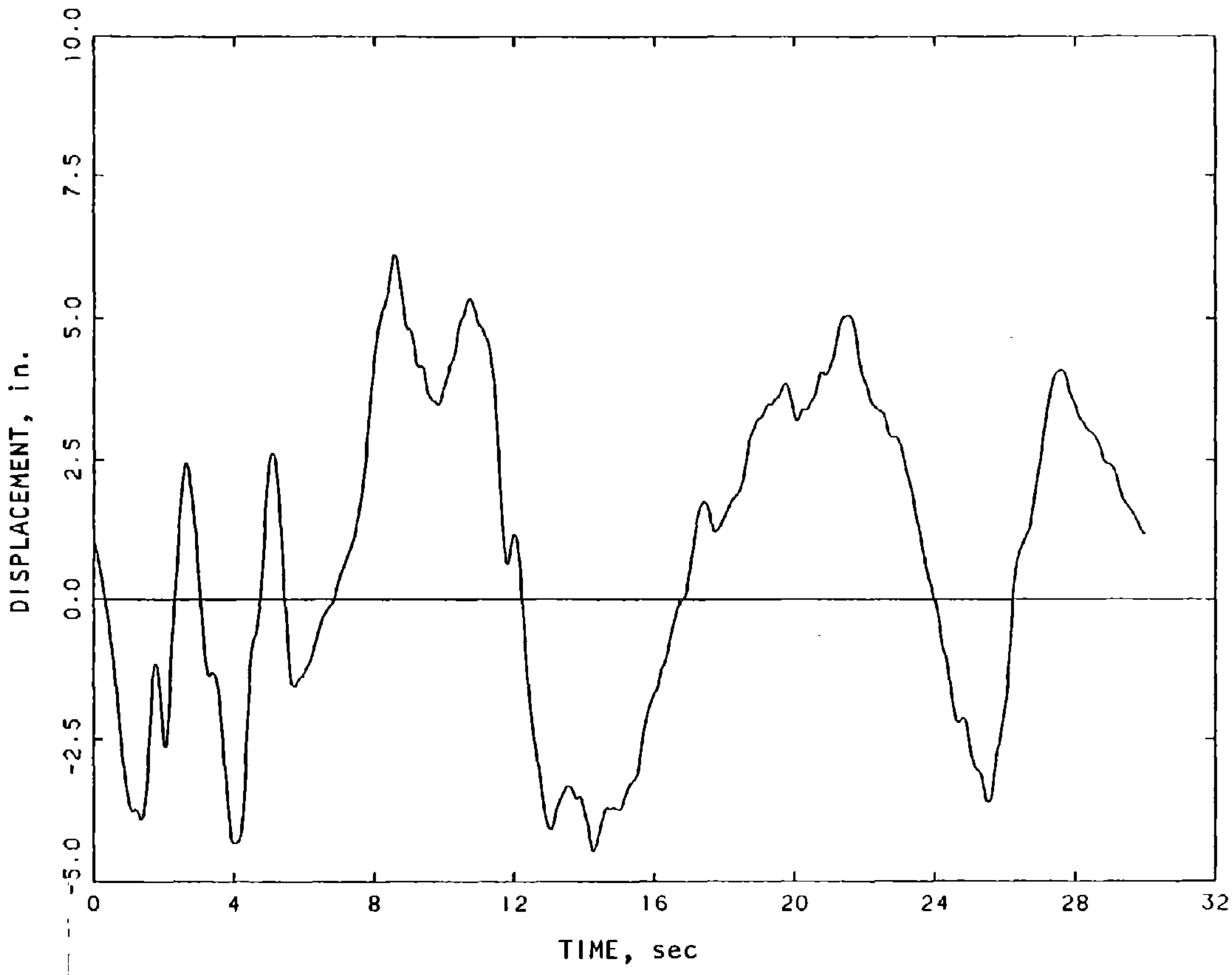


FIGURE A-4. INPUT BASE DISPLACEMENT, EPA = 0.4g
(EL CENTRO x 1.25)

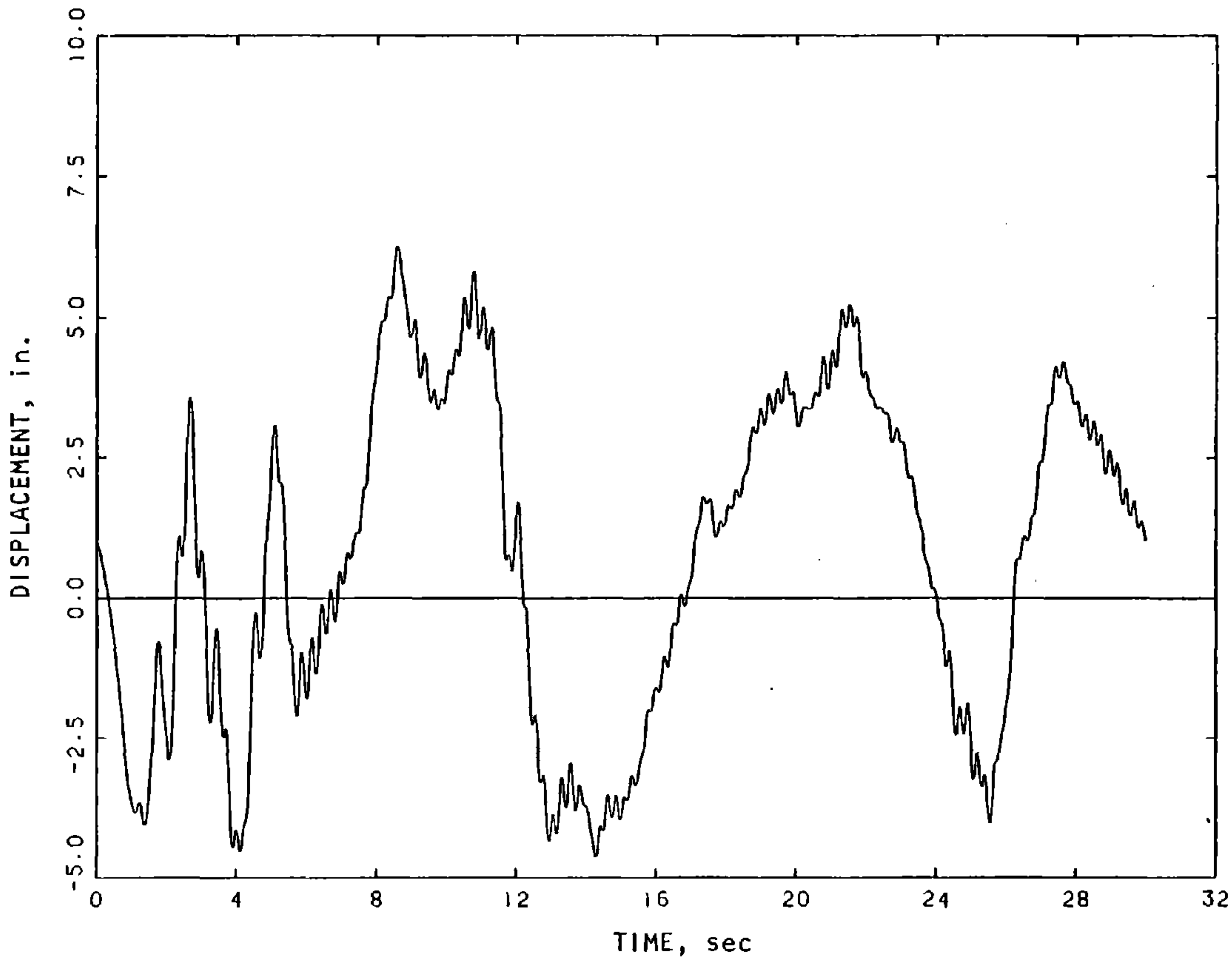


FIGURE A-5. HORIZONTAL DISPLACEMENT AT TOP OF WALL, H/D = 1.5, k_s = 100, EPA = 0.4g (EL CENTRO x 1.25)

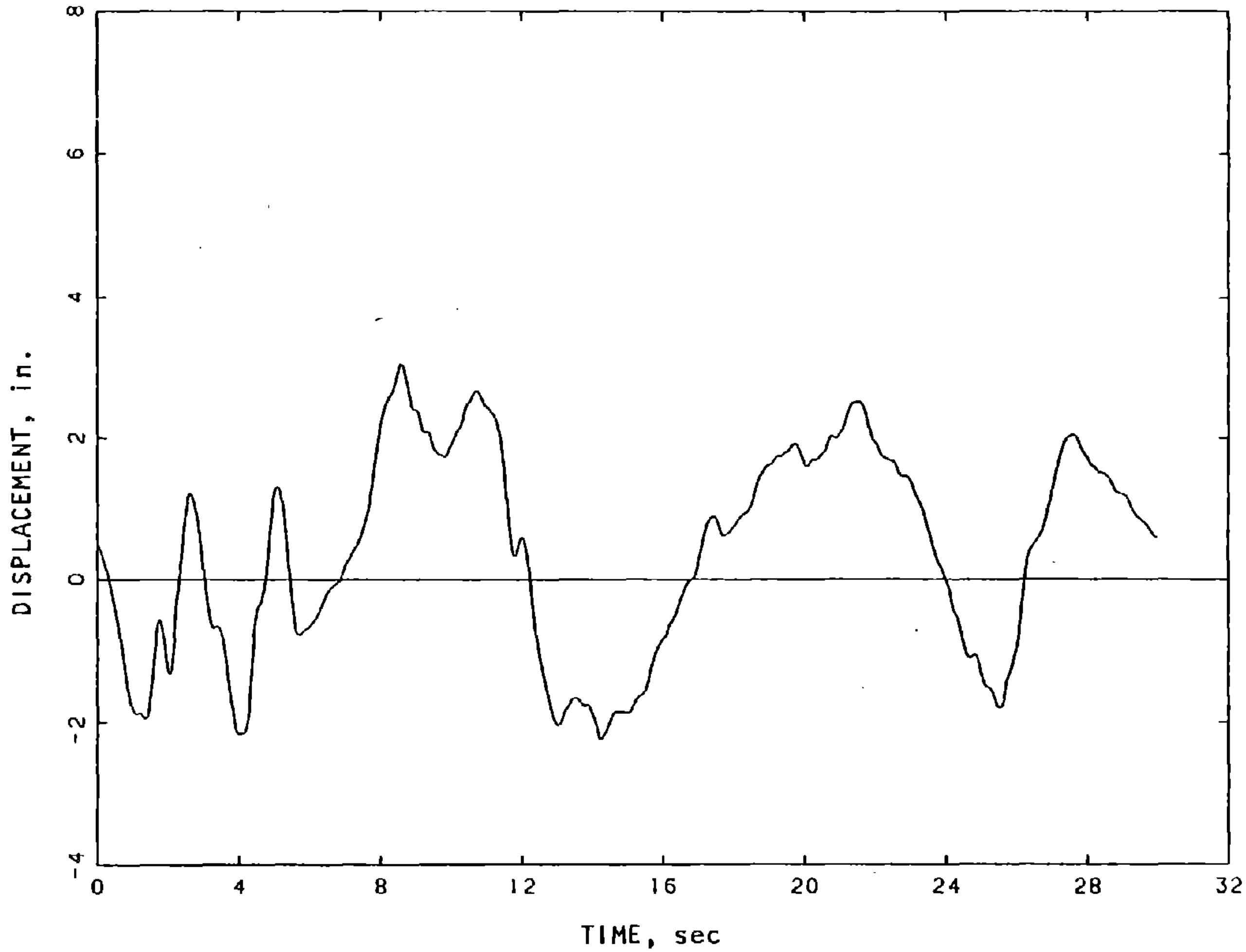


FIGURE A-6. INPUT BASE DISPLACEMENT, EPA = 0.2g
(EL CENTRO x 0.625)

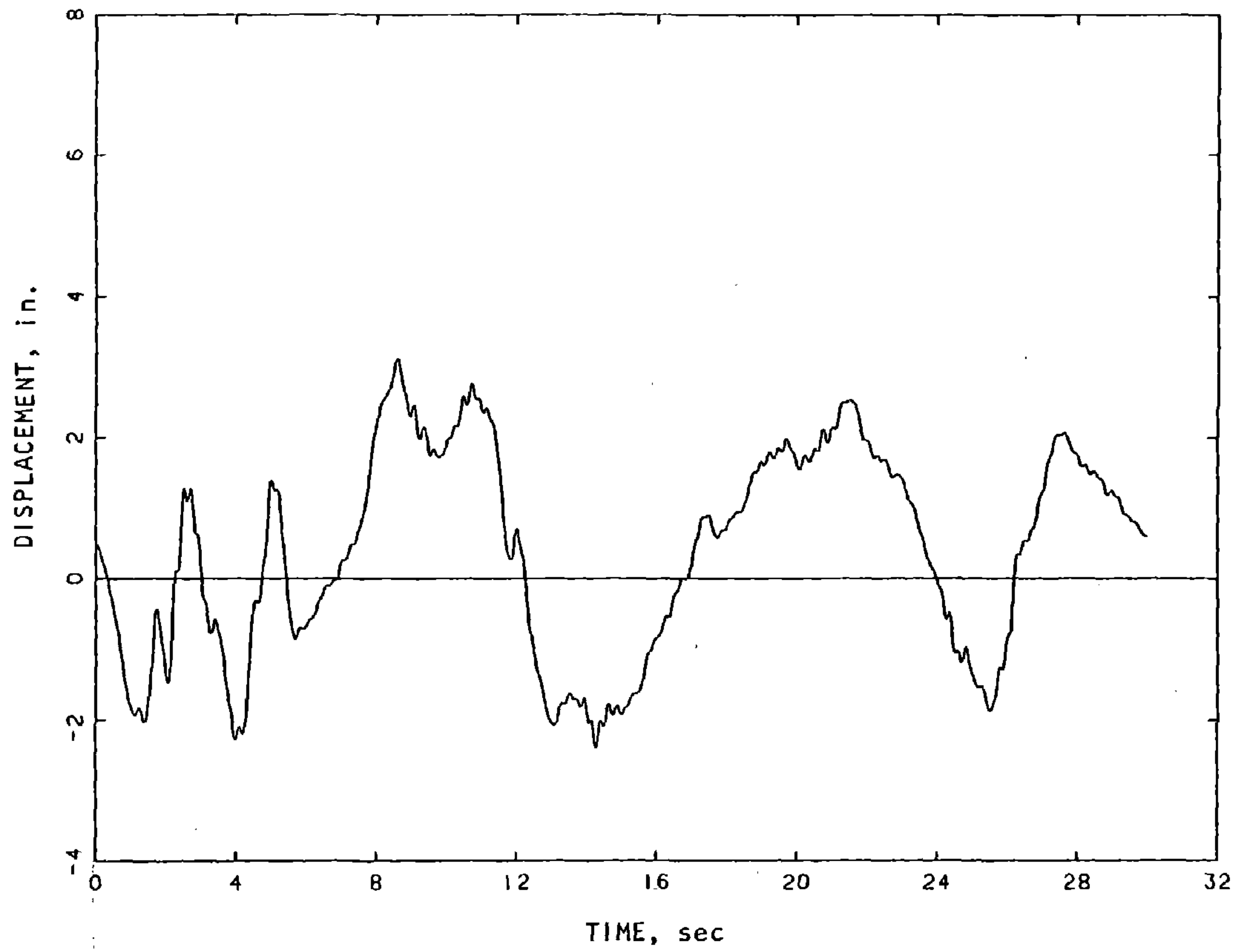


FIGURE A-7. HORIZONTAL DISPLACEMENT AT TOP OF WALL, H/D = 1.5, k_s = 100, EPA = 0.2g (EL CENTRO x 0.625)

APPENDIX B

SEISMIC RESPONSE MODEL OF DIAPHRAGMS WITH CROSSWALLS

APPENDIX B

SEISMIC RESPONSE MODEL OF DIAPHRAGMS WITH CROSSWALLS

An analytical model has been developed for the static and dynamic analysis of horizontal diaphragms (ABK, 1982a), and this model has been correlated with tests on full scale diaphragms subjected to seismic inputs. In addition, this model has been incorporated into an URM building model that was used to develop the horizontal displacement control data provided in Figure 8-2 of this report (ABK, 1982a). This same building model has been adapted to evaluate the seismic response of diaphragms with and without crosswalls to develop data on velocity control (i.e., reduction of velocity amplification).

The building model is shown in Figure B-1. The horizontal diaphragm is represented by nonlinear, hysteretic springs, labeled D, and their force-deformation characteristics are shown in Figure B-2. The diaphragm characteristics are basically defined by two parameters, F_u and K_i, and these values are obtained from static tests on diaphragms. Vertical crosswalls can be added at the potential locations shown in Figure B-1. The crosswalls are represented by bilinear, hysteretic springs, labeled CW, and their force-deformation characteristics are shown in Figure B-3. In this model, the crosswall characteristics are completely defined by F_{cw} and e_y, the displacement at yield. In this report crosswalls are referred to by a percentage, for example k% crosswalls. This means that the sum of the crosswall resistance values, F_{cw}, at a given level in a building are k% of the horizontal diaphragm's ultimate capacity, or

$$\Sigma F_{cw} = \frac{k}{100} F_u$$

or in terms of the methodology

$$\Sigma V_c = \frac{k}{100} v_u D$$

as defined in Section 3. In the analyses, a range of crosswall capacities, k, were examined, where k varied from 10 to 50%. The yield displacement, e_y, was selected to account for the crosswall construction materials and ceiling height, and values ranged between 1/2 and 1 in. The construction materials included lath and plaster and most nailed systems.

Figure 8-2 has been reproduced in Figure B-4 and some peak velocity amplification factors have been overlaid on the figure. A velocity amplification factor is the ratio of the peak velocity induced in the diaphragm to the peak input velocity. The amplification factors were obtained using the building model shown in Figure B-1, with and without vertical crosswalls.

Based on out-of-plane stability considerations (Table 8-1), velocity amplification ratios of 1-3/4 or larger for an EPA of 0.4 g will create stability problems for URM walls in the out-of-plane direction. Moreover, it is clear from Figure B-4 that there are regions where velocity amplification is not a concern (region 1). In region 2 where velocity amplification is a concern, the addition of crosswalls may help reduce velocity amplification. For example, for a diaphragm span of 240 ft and a demand-capacity ratio of 2, the velocity amplification is unacceptable at 1.97 without crosswalls. When only 20% crosswalls are added the velocity amplification is reduced to an acceptable value of 1.68.

Data on displacement control is provided in Figure B-5, which is similar to Figure B-4, except peak relative story displacements are overlaid on the figure. From the figure, it can be seen that crosswalls can be effective in controlling relative displacements. For example, for a diaphragm span of 240 ft and a demand-capacity ratio of 2, the relative displacement of 4.87 in. is reduced to 4.06 in. by the use of 20% crosswalls.

These analyses have led to the recommendations in this methodology as defined in Section 8.

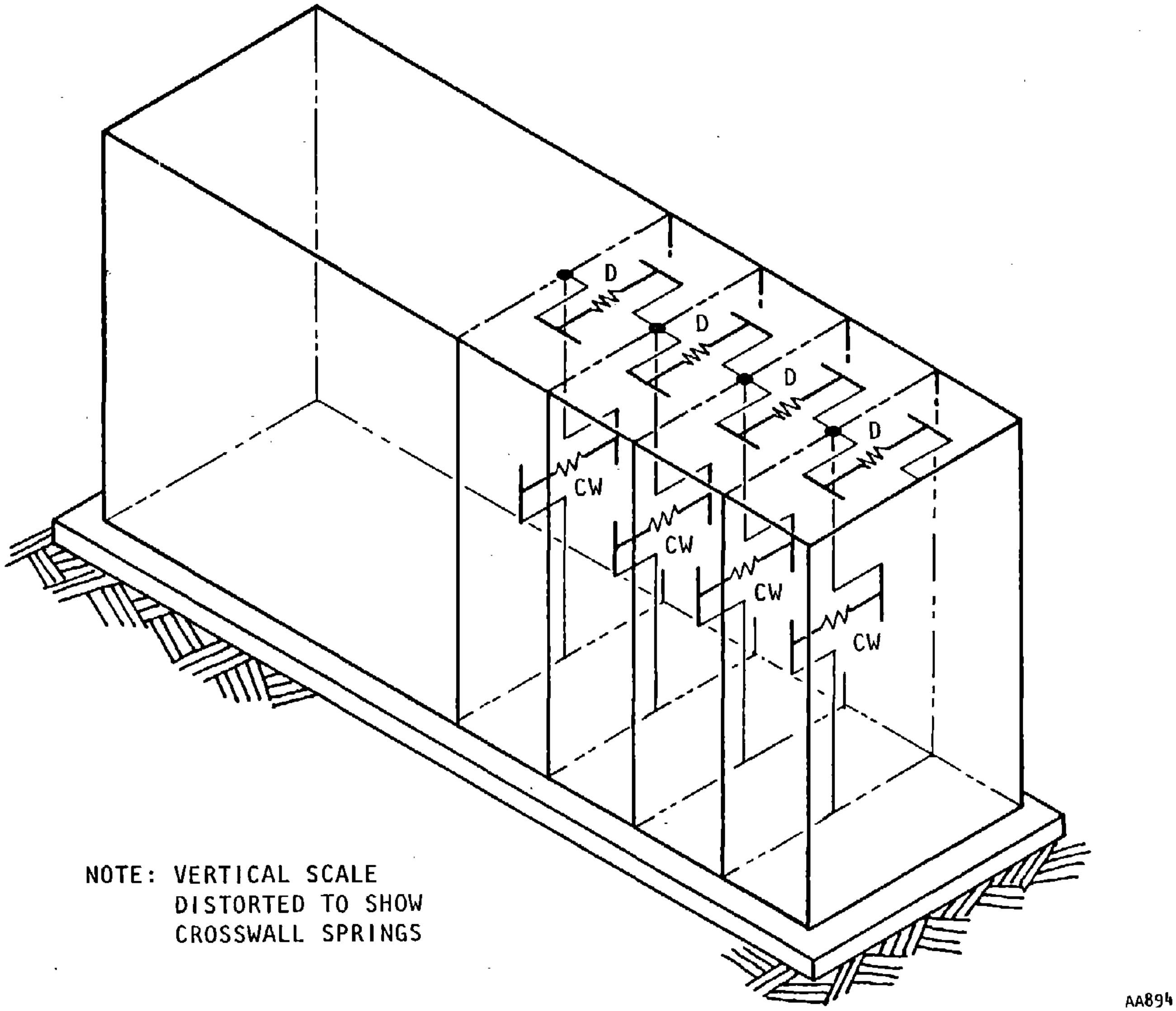
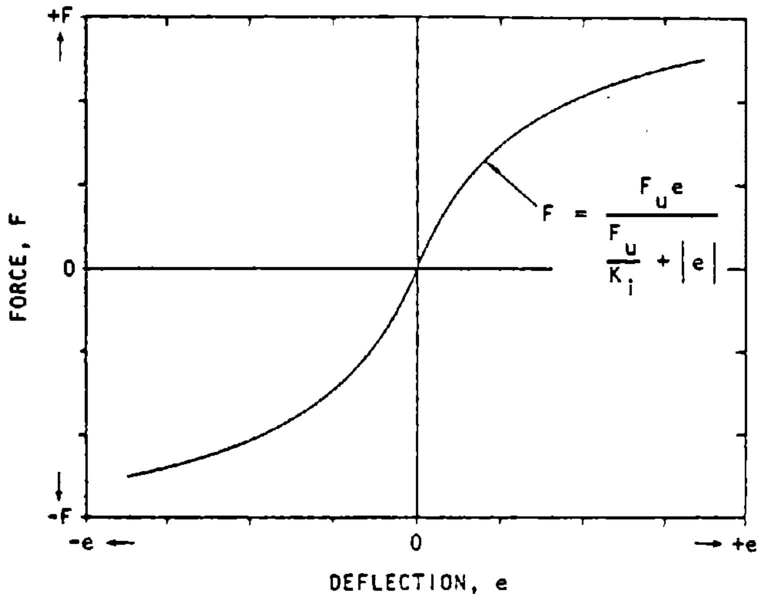
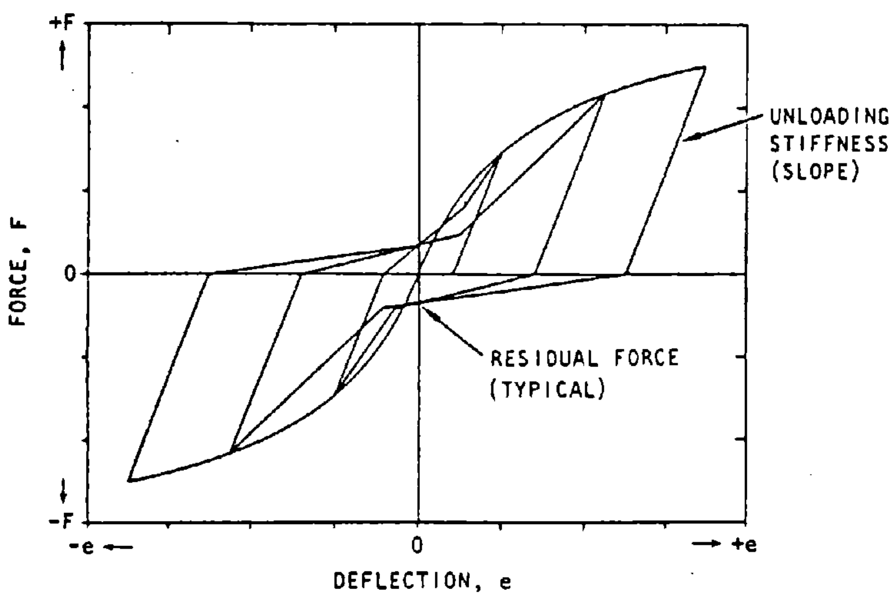


FIGURE B-1. ANALYTICAL MODEL FOR URM BUILDING WITH CROSSWALLS

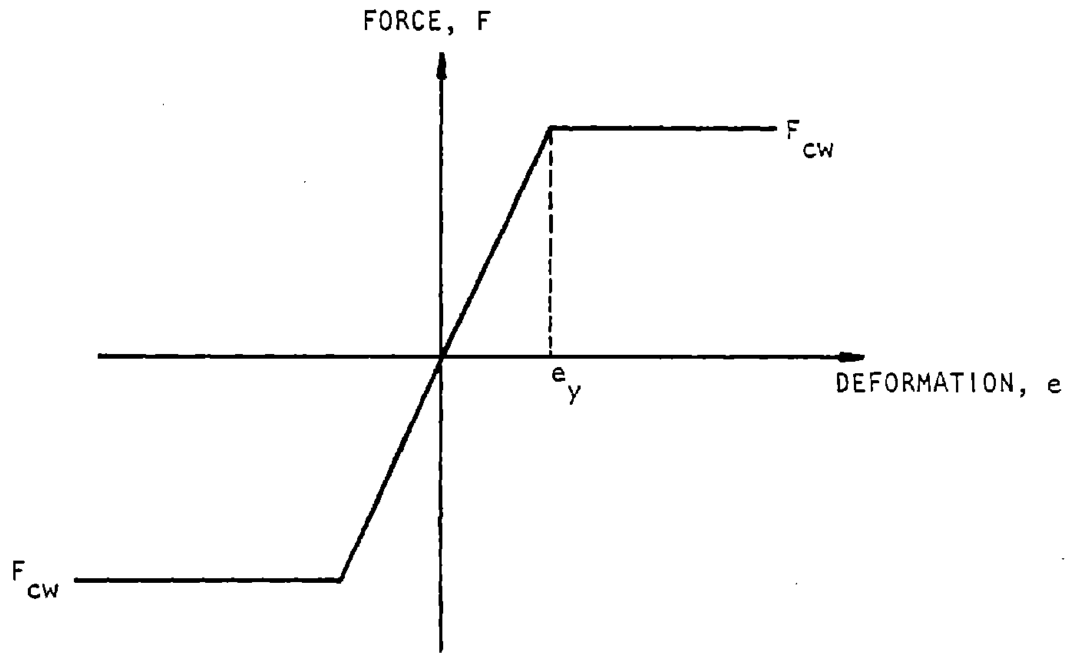


(a) Force-deflection envelope of model

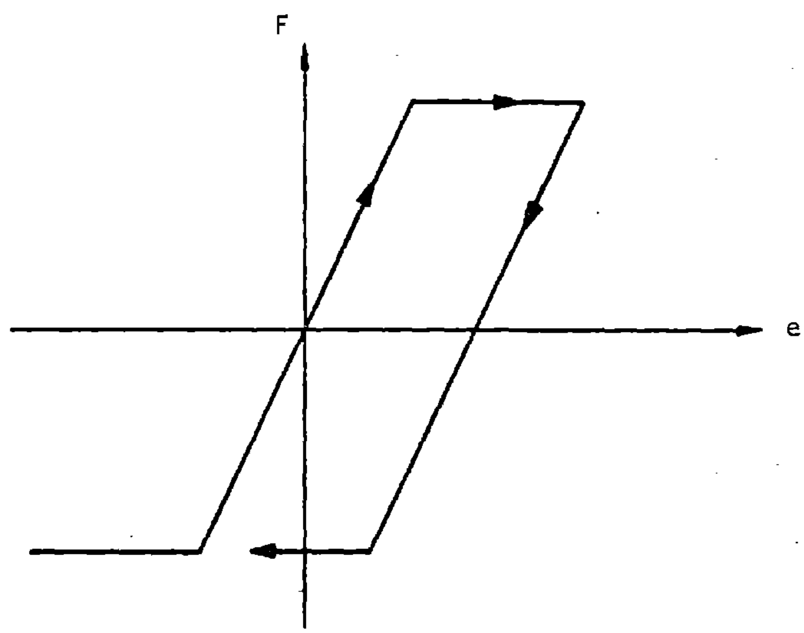


(b) Typical cyclic load-deflection diagram for model

FIGURE B-2. LOAD DEFLECTION MODEL FOR WOOD DIAPHRAGMS



(a) Force-deformation envelope



(b) Typical load and unload path

FIGURE B-3. FORCE-DEFORMATION CHARACTERISTICS OF CROSSWALL SPRING

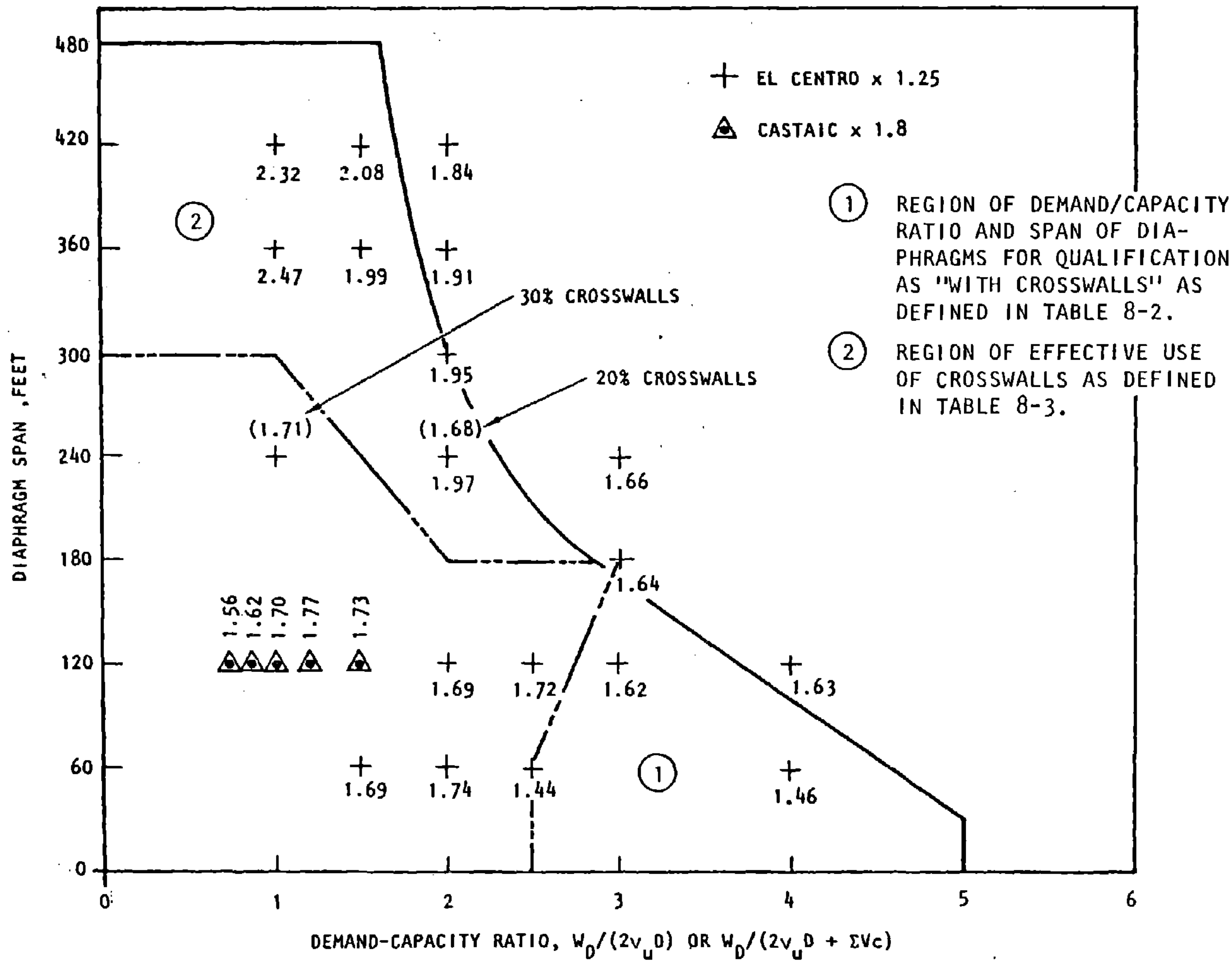


FIGURE B-4. VELOCITY AMPLIFICATION FACTORS FOR DIAPHRAGMS WITH AND WITHOUT CROSSWALLS

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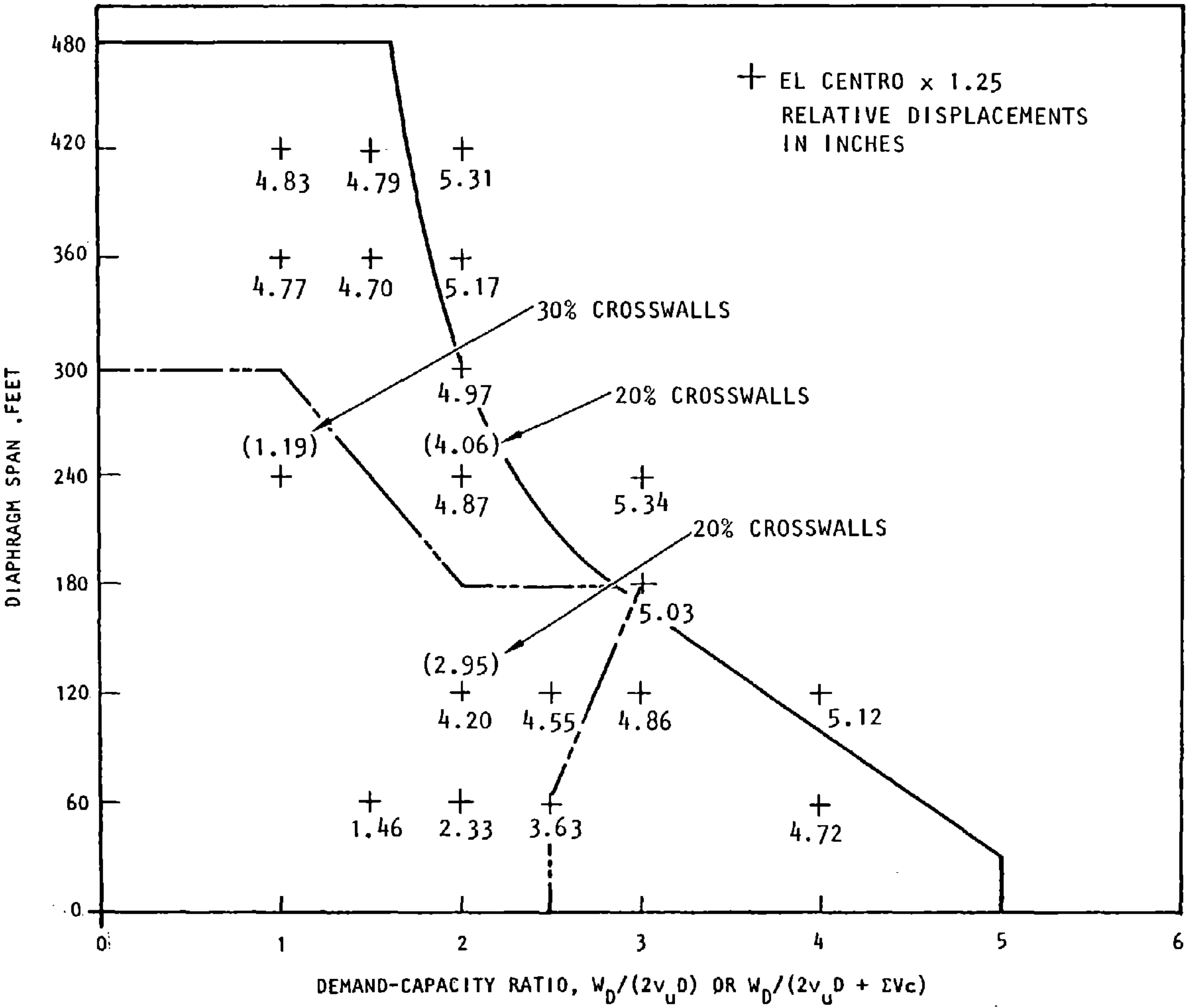


FIGURE B-5. PEAK RELATIVE STORY DISPLACEMENTS FOR DIAPHRAGMS WITH AND WITHOUT CROSSWALLS

APPENDIX C

IN-PLANE TESTING OF EXISTING URM MASONRY

SECTION C-1

INTRODUCTION

C-1.1 OBJECTIVES

The objective of in-place testing of URM is to determine the relationship between the shear capacity of URM piers parallel to the bed joints and the in-place shear capacity that is determined by displacing a single brick relative to the adjacent brick units. Shear displacement caused by seismic inertial forces in URM piers typically follows a diagonal path through a pier. This diagonal path is usually in bed and head mortar joints, rarely passing through the brick unit. If the tensile capacity of the head joint is discounted, the horizontal shear capacity of a pier can be described as a function of the shear capacity of the critical part (bed joints) of the probable failure path.

The capacity of a pier is typically expressed as an average shear times the pier area. It is recognized that the state of shear stress is not uniform across the pier section and that an average shear does not exist. The objective of in-place testing is to determine coefficients that may be used to relate the probable shear that initiates the propagation of a shear crack to a calculated response shear.

C-1.2 TEST BUILDING

The in-place URM tests were conducted in a building scheduled for demolition. Permission to perform destructive testing in the existing URM walls was given to ABK by the City of Los Angeles through its Department of Public Works. This assistance and cooperation is gratefully acknowledged.

C-1.3 TEST PROCEDURE - STATIC TESTING

Free-standing URM piers were constructed by sawing specimens from a three-wythe URM bearing wall (Fig. C-1). The space around the specimen perimeter provided space for hydraulic jacks, and in-plane displacement gauging (Fig. C-2). All test specimens were 13 in. (330 mm) in thickness and approximately 11 ft (3.35 m) high. Test piers 1 and 4 were 4 ft (1.2 m) wide, piers 2 and 3 were 5 ft (1.5m) wide.

A total of 19 in-plane shear tests were made in the URM walls adjacent to the test piers. A series of shear tests were made in an area of minimum axial stress in an adjacent wall. The tests 2, 2 opposite, 4, 13, and 16 were located under large windows.

The sequence of static testing was planned to obtain the following data:

- Determine a flexural modulus of rupture for a horizontal crack on bed joints.
- Determine a Coulomb shear for a horizontal surface with cohesion reduced to zero by flexural cracking.
- Determine if the Coulomb shear on the horizontal crack is related to the pier rotation on the flexural crack.
- Determine the shear capacity of the URM pier if the state of stress in the pier is such that flexural cracking at the pier base is improbable.

The flexural modulus of rupture of the bed joint crack was ascertained by placing a hydraulic jack in position a, as shown in Figure C-2. The modulus of rupture was calculated from the jacking force recorded when the top displacement indicated variance from a linear load-displacement relationship. The pier was displaced both right and left to double the data quantity.

After a crack was fully propagated across the base of the pier, the pier was rotated on the base crack to 6 to 8 times the elastic (uncracked) displacement. The jack position was lowered for each successive test to positions b, c and d. The height of load at position d was selected to force the pier to slide on the cracked surface. Piers 3 and 4 were not cycled on the cracked bed joint. This procedure was planned to preserve a near-virgin crack for subsequent dynamic testing.

After dynamic testing of the specimen piers, concrete was placed in the open space (see Fig. C-3) to above the level of the base crack, and load jacks were placed in the locations noted. The jack pressures were increased in a sequence such that a diagonal shear failure was caused. The shear failure was near-instantly propagated across the pier and was accompanied by a very audible sound.

C-1.4 TEST PROCEDURE - DYNAMIC TESTING

After completion of the first sequence of static testing, the piers were instrumented with three Kinemetrics Model FBA-1 accelerometers attached to the piers by studs epoxied into the masonry (Fig. C-4). The pier was displaced to the planned top displacement by a hydraulic jack. A quick release device (designed by A.W. Johnson) was inserted into the space and the jack was removed (Fig. C-5).

The quick release device was triggered and the pier was allowed to displace and close the base crack. The pier rocked on the base crack and came to rest. Continuous acceleration recordings obtained during the displacement decay cycle were analyzed to determine an apparent viscous damping.

SECTION C-2

RESULTS OF THE STATIC TESTING

C-2.1 IN-PLACE SHEAR TESTING

The 19 in-place shear tests were plotted in accordance with the recommendations of Section 9.6 of the Methodology (Fig. C-6). The value of in-place shear as determined by the recommendation is 47 psi. If each of the in-place shear tests is reduced by use of the formula $v_{reduced} = v_{test} - f_a$ at point of test, the mean of the reduced test shears is 75.6 psi. The standard deviation of the calculated mean value is 30.8 psi.

After the diagonal shear testing of the piers, eight additional in-place shear tests were made. These shear tests were made as close to the diagonal fracture path as possible. Plotting of these tests on Figure C-6 would give a tight clustering of the data points on the plotted line and would not change the basic test value as determined by the original 19 data points.

C-2.2 MODULUS OF RUPTURE ON BED JOINTING

The modulus of rupture was calculated using typical elastic bending theory. The calculated modulus of rupture varied from 10 psi to 20 psi. The mean of 7 tests was 16 psi. The standard deviation was 3 psi.

The jack load used for calculation of the modulus of rupture was taken when the appropriate displacement gauge (Fig. C-2) indicated a displacement that was nonlinear with prior recorded displacements. The occurrence of cracking was generally confirmed by the reloading of the pier. The reloading force-displacement plot confirmed the existence of a flexural crack.

The testing for modulus of rupture indicated that a cohesion capacity generally exists on the bed joints of existing URM masonry. A near identical tensile capacity of the mortar/masonry unit interface was determined by a prior URM test program (Schmid et al., 1978).

C-2.3 COULOMB SHEAR CAPACITY OF A CRACKED BED JOINT

Testing of URM piers with the horizontal load jack in position d (Fig. C-2) displaced the pier horizontally on the cracked joint. For pier 1, the force required to displace the pier was 2.0 times the available axial load the first time sliding occurred, and 1.75 times the axial load the second time the cracked bed joint was forced into Coulomb shear displacement. The displacement at the top of the pier was about 6 times elastic displacement when horizontal shear displacement occurred.

Test 5, pier c, used a load jack above the test pier to increase the available axial load on the cracked bed joint. Horizontal displacement on the bed joint was caused by a horizontal load of 1.15 times the available axial load. At this horizontal load, the top displacement of the pier was about 3-1/2 times the elastic pier displacement.

C-2.4 SHEAR CAPACITY OF URM PIERS

Four URM piers (Fig. C-1) were loaded by hydraulic jacks to force a diagonal shear crack. The application of vertical load to the URM pier was limited by the available weight of the URM wall above the test pier. The computation of axial and bending stress at the base of the pier (on the assumption that Hookes Law is valid) indicates that a flexural crack would propagate horizontally across the base of the pier. Finite element studies (App. D) indicate that cracking oriented perpendicular to the tensile stress would propagate into the pier base and not contribute to a shear failure.

The pier stress at the shear failure loading is given as average stresses in Table C-1. Average axial and shear stresses are calculated as $P \div A$ and $V \div A$, respectively.

TABLE C-1.

Pier	Average Axial Stress	Average Shear Stress
1	23.5 psi	26.0 psi
2	17.8 psi	23.1 psi
3	14.4 psi	23.9 psi
4	17.9 psi	33.0 psi

The failure surface propagated through the pier with an audible sound. The failure surface generally traversed mortar joints but occasionally passed through brick units.

Interpretation of the results of these shear tests is presented in Appendix D.

SECTION C-3

RESULTS OF THE DYNAMIC TESTING

C-3.1 EQUIVALENT VISCOUS DAMPING OF CRACKED URM PIERS

The dynamic testing of cracked URM piers consisted of displacing the top of the pier a predetermined displacement. Each top displacement was related to the recorded top displacement at the elastic limit determined by prior static testing. This procedure was selected to correspond with probable excursions of the pier top relative to the pier base in the inelastic range. The displacements used for dynamic testing for equivalent damping ranged up to 8 times elastic displacement.

Piers 1 and 2 had been extensively tested through static cycles that rotated the pier at the base crack up to 6 to 8 times elastic displacement. In addition, the crack at the base had been subjected to horizontal displacement in testing for determination of Coulomb shear on a cracked surface.

Pier 3 had only been cycled once in each direction to determine the modulus of rupture of mortar bed joints. Pier 4 was cycled once to the left (Fig. C-1) to determine its modulus of rupture.

The data report prepared by Kinemetric, Inc. is excerpted in this section (Nigbor, 1983). The test was planned to produce data to clarify understanding of energy dissipation of rocking URM masonry walls or piers. It was theorized that loss of kinetic energy was sustained on each cycle of crack closing. The energy loss was expected to be related to a coefficient of restitution of less than 1.0.

Figure C-7 lists the results of pertinent data runs on Piers 1 and 2. Figures C-8, C-9, and C-10 are plots of accelerometer data at Stations A, B, and C, respectively. For displacements of the pier top nearly equal to elastic limits (initial cracking displacement), the data indicate little energy dissipation. Data plotted at Stations A and C (Figs. C-6 and C-10) indicate impact crushing of the previously disturbed mortar at the base crack. As the top displacement is increased (Run 6, Pier 1, Figs. C-11, C-12, and C-13), the energy dissipation on impacting becomes much more apparent. Figure C-12 indicates a near-elastic viscous damped decay plot beginning about 2 sec after pier release. The earlier record models inelastic performance with a high frequency mode imposed on the pier rocking period. This high frequency mode is interpreted as the elastic period of the uncracked pier above the base crack.

To present a record that can be related to apparent viscous damping, the acceleration-time records were integrated to displacement-time records. Figures C-14, C-15, and C-16 present displacement-time records for Pier 1, Run 5 for Stations A, B, and C, respectively. From these displacement-time records, an apparent viscous damping was computed. Figure C-17 indicates a displacement-time record obtained from a large initial displacement of the pier top. The initial displacement was about 8 times elastic displacement.

Table C-2 presents damping data derived from the dynamic testing. The damping ratio calculated was consistent, with the exception of Pier 3 for large displacements. This record indicated equivalent damping of 12.4% on the first cycle, the ratio reduced to 4.7% on the 5th cycle.

SECTION C-4

CORRELATION OF IN-PLACE URM TESTING DATA WITH PRIOR IN-PLACE TEST DATA

C-4.1 RESULTS OF PRIOR URM TEST PROGRAMS

A test program of URM masonry was conducted in a three-story URM apartment building in 1978. Results of this testing were reported in Schmid et al. (1978). The City of Los Angeles made the building available for destructive testing prior to its scheduled demolition for street realignment.

The in-place test program included an extensive series of in-place shear tests. Two in-place shear tests of large URM piers (Fig. C-18) and two diagonal compression tests of in-place URM (Fig. C-19) were made. The in-place shear tests varied from the procedure now used for shear testing. About one-half of the tested bricks were separated from the collar joint prior to shear testing. When the results of the shear testing were plotted in the recommended format (Fig. C-6), the test bricks separated from the collar joint totally controlled the plotting of the line determining the test shear reduced to zero axial stress.

The plot of the in-place shear test gave a tested shear value of 30 psi at zero axial stress. The average shears for the piers shown in Figure 3-10 were 22.9 and 27.4 psi. The average axial stresses were 11.2 and 16.2 psi for each pier.

Shears for the diagonal compression specimens were computed by use of a formula for isotropic materials. These computations indicated shears parallel to the bed joints of 37 psi and 52.7 psi for the specimens. Use of the same formulas indicated the axial stresses normal to the bed joint were 27.6 and 39 psi. The bed-joint shear computations made using formulas for isotropic materials indicate the critical shear is very nearly 1-1/2 times average shear.

C-4.2 CORRELATION OF CURRENT IN-PLACE URM TESTING AND PRIOR TESTING

The in-place shear testing conducted in 1978 indicated that separation of the test brick from the collar joint reduced the test shear value. It is recognized that the collar joint may be infilled with mortar. This condition is one of many workmanship flaws that cause a large scatter of shear test results. However, a methodical removal of bonding on the collar joint will give a reduction in the mean test shear value. The procedure for removal of the collar joint has not been required for testing in conformance with current hazard ordinances. Instead, reduction of the test values by substantial factors were required. These reduction factors were developed by an Ad Hoc Technical Committee. The methodology recommends that presently used reduction factors be revised.

The 20 percentile test shear value reduced to zero axial stress must be modified to remove any influence of a bonded collar joint. A shear failure of a URM pier propagates through bed and head joints; collar joints are not subjected to shear stresses. A reduction factor of 0.75 for modification of the plotted test shear value is recommended. This factor is determined by the relationship of brick surfaces loaded in the test procedure to the surfaces loaded in a URM pier subjected to interstory shear. The top and bottom surfaces of a common brick are about 4 in. (100 mm); the edge of a common brick that may be bonded to a collar joint is about 2-1/2 in. (64 mm). The recommended reduction factor is based on the ratio of horizontal surfaces (8 in.) to horizontal surfaces plus one edge (10-1/2 in.).

Use of this reduction factor lowers the plotted 20 percentile test value (Fig. C-6) from 47 psi to 35 psi. This corrected test shear value is comparable to the test value of 30 psi as determined by prior testing of URM performed in 1978 (Schmid et al., 1978).

C-4.3 RECOMMENDATIONS FOR DETERMINATION OF RESTORING SHEAR CAPACITY OF URM PIERS

The restoring shear capacity of a URM pier is defined in the methodology as the shear force that opposes the inertial forces that displace the top of a pier relative to its base (Fig. C-20). This restoring shear is maintained for relative displacements in excess of 8 to 10 times cracking displacement.

Static testing of URM piers first determined the modulus of rupture of a bed joint at the bottom of the pier. At the point of nonlinear displacement, the jack pressure was dropped to zero. The jack pressure was then slowly increased, and crack propagation across the base was observed. The size of the compression block diminished and then stabilized at top displacements of 6 to 8 times cracking displacement. The URM, laid with lime mortars, had a low modulus of elasticity and the compression block capacity was maintained.

The recommended method for computing the restoring shear of URM piers is simplified to

$$V_R = 0.9\ P\ \frac{D}{H}$$

where

V_R = Restoring shear

P = Axial load on pier

D = Depth of pier measured in the wall plane

H = Height of pier or least height of pier if opening height on sides of pier varies

The restoring shears of all piers in a shear wall can be added to meet the requirements of the recommended restoring shear. The mobilization of the restoring shear of each pier is related to its in-plane dimensions. However, the combined capacities of the URM piers will be mobilized by inelastic displacements of the shear wall.

Restoring shear capacity of a URM pier is a shear wall property that is separate from in-plane shear capacity. If the pier H/D ratio and existing axial load on the pier are such that a flexural crack does not propagate across the pier top and bottom and the inertial response shear exceeds the pier in-plane shear capacity, diagonal shear cracking is probable.

Prediction of flexural cracking cannot be reliably made using usual computational techniques. Typical computations using Hooke's Law assumptions indicate that flexural cracks would propagate across the top and bottom of the pier when $V \geqq P/3\ (D/H)$. This shear can be substantially increased if significant bed-joint tension capacity exists. Static testing, observation of crack development in URM piers, and finite element analysis of URM piers indicate that usual calculation methods cannot predict probable flexural cracking.

If flexural cracking does occur at the top and bottom of a pier, the shear V_R in the pier is limited to 0.9 P (D/H). If the in-plane shear capacity of the pier exceeds V_R, diagonal shear cracking is a very small probability.

C-4.4 RECOMMENDATION FOR DETERMINATION OF ACCEPTABLE URM IN-PLANE SHEAR STRESSES

In-place shear testing of brick units is considered to be the most reliable and cost effective method of determining the shear capacity of high lime mortars bonded to significantly stronger masonry units. It is recommended that the tested value of in-place shear be corrected for the effect of probable bonding at the collar joint. The 20th percentile of the tested shear value reduced to zero axial stress normal to the bed joint is recommended as a basic test bed joint shear. The relationship of this basic test shear and probable peak shear that initiates a diagonal shear value was studied by finite element analysis and is reported in Appendix D.

TABLE C-2. DYNAMIC PIER TEST, DAMPING DATA TABLE

Pier	Run	Initial Displ.(in)	Damping Data (see note) Avg.(%cr)	Std. Dev.	No. Cycles	95% Confidence Interval
1	2	0.12	4.0	0.29	9	3.75 - 4.19
1	5	0.5	3.6	0.46	14	3.35 - 3.89
2	7	0.05	3.4	0.35	10	3.17 - 3.67
2	13	0.73	3.3	0.70	9	2.80 - 3.88
3	14	0.13	4.3	0.51	8	3.87 - 4.73
3	18	0.6	7.6	2.80	5	4.1 -11.1
4	22	0.5	4.0	0.34	8	3.71 - 4.29

Note-Damping was calculated from Station B displacement time histories using the logarithmic decrement method for each of the first n cycles(see 'No. Cycles' column) of vibration. Displacement time histories were integrated from the measured acceleration time histories using B. Burke's algorithm with alpha=1.8.

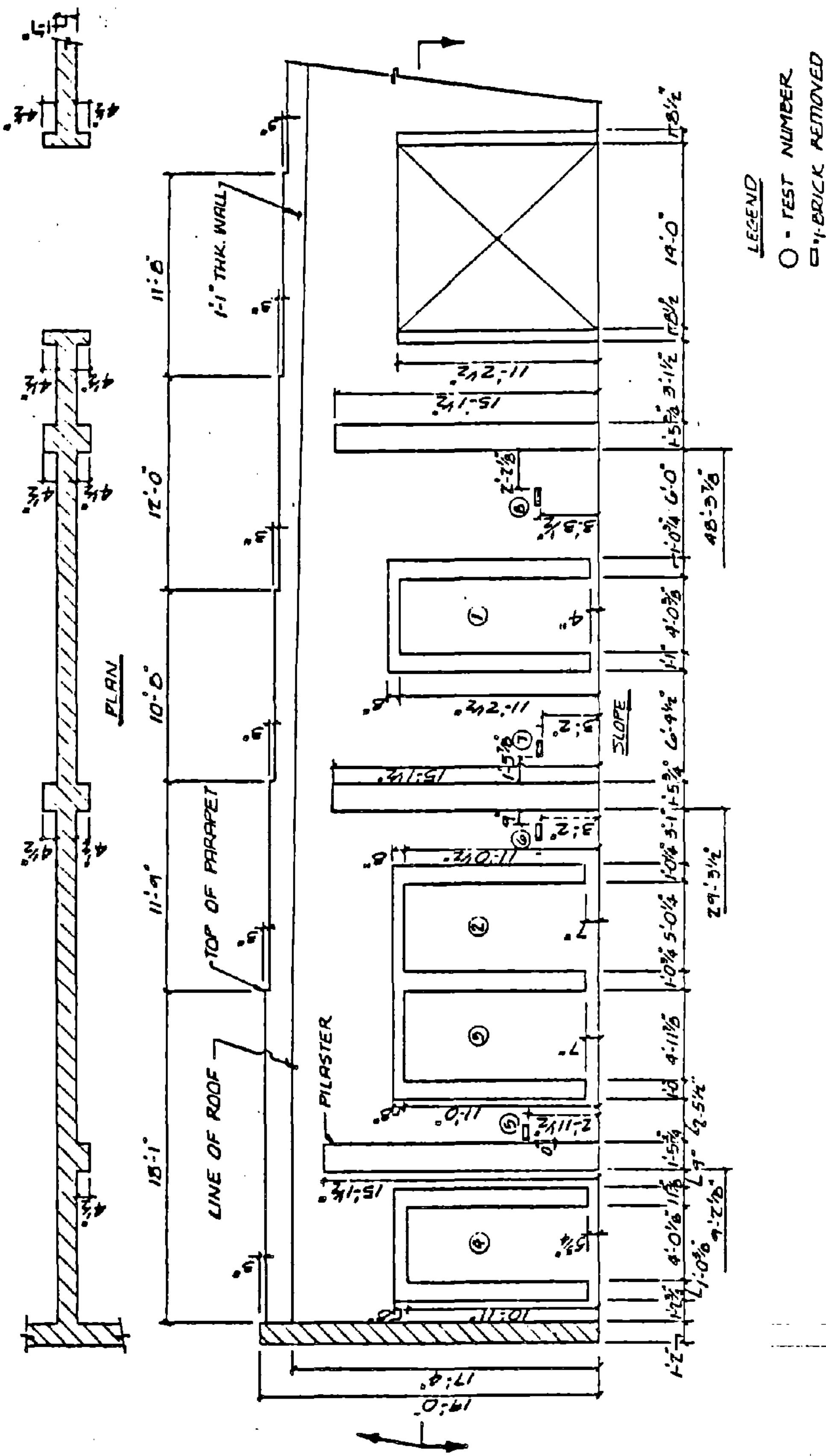


FIGURE C-1. INTERIOR WALL ELEVATION

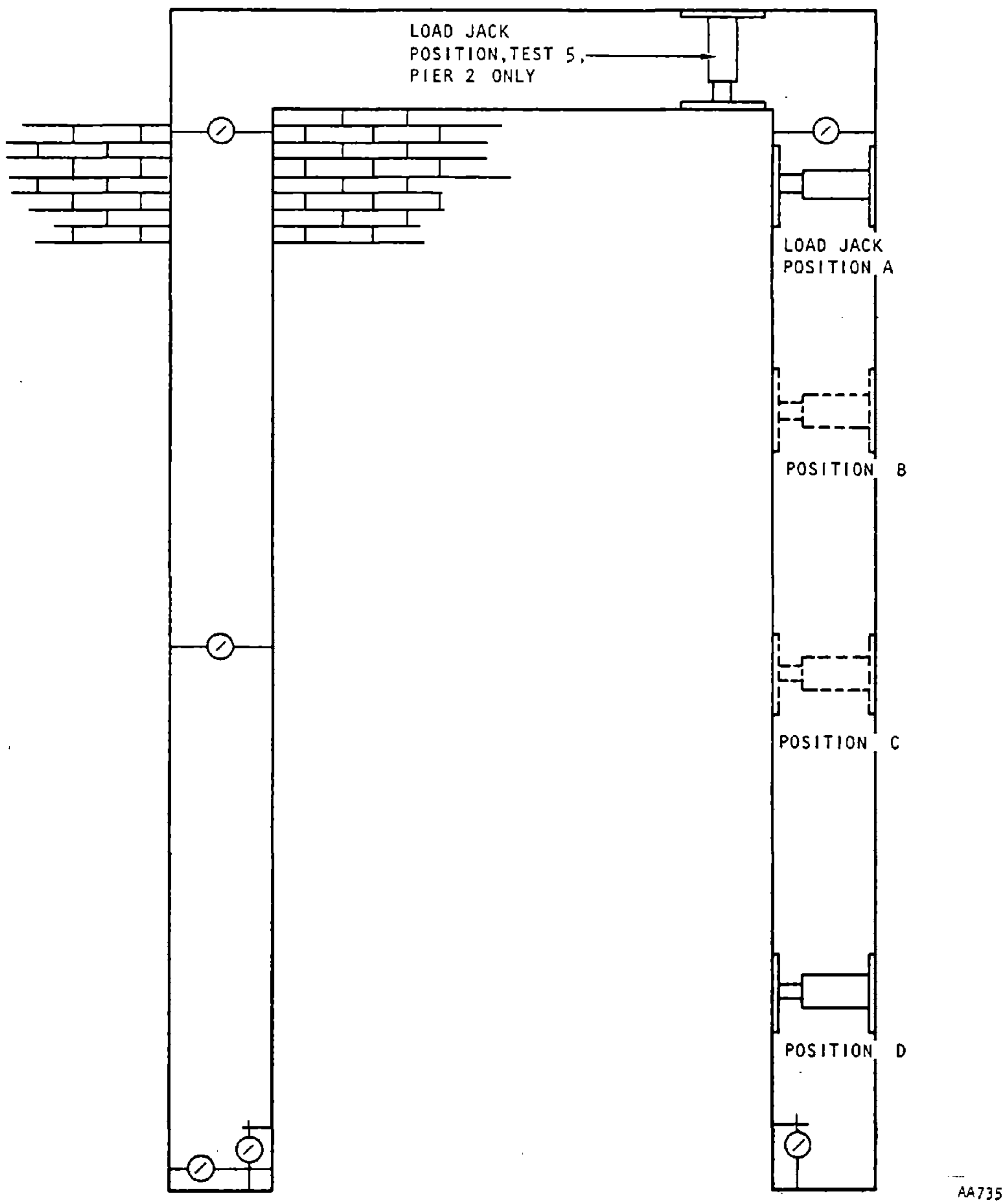

LOAD JACK
POSITION,TEST 5,
PIER 2 ONLY
LOAD JACK
POSITION A
POSITION B
POSITION C
POSITION D
AA735

FIGURE C-2.

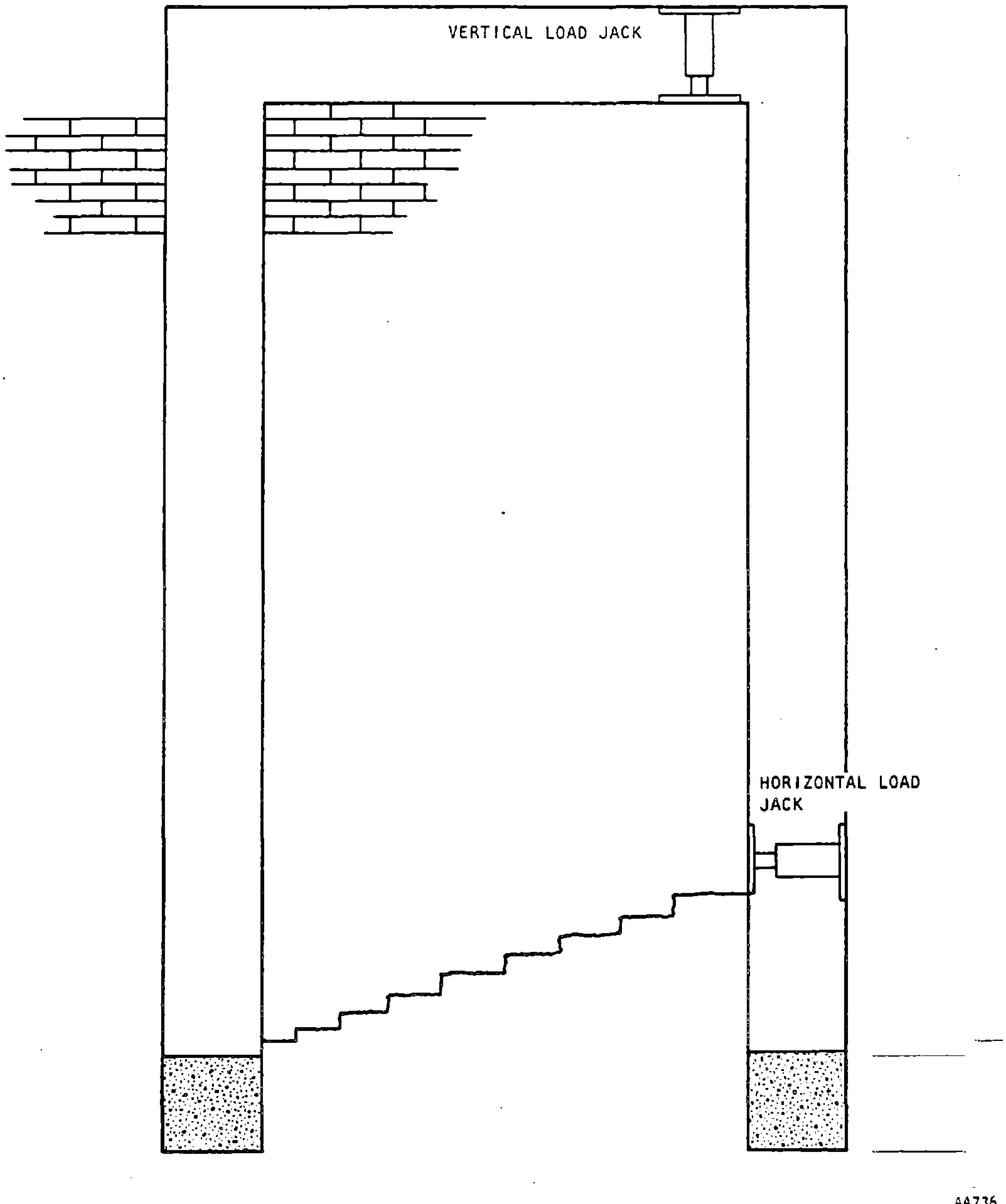


FIGURE C-3.

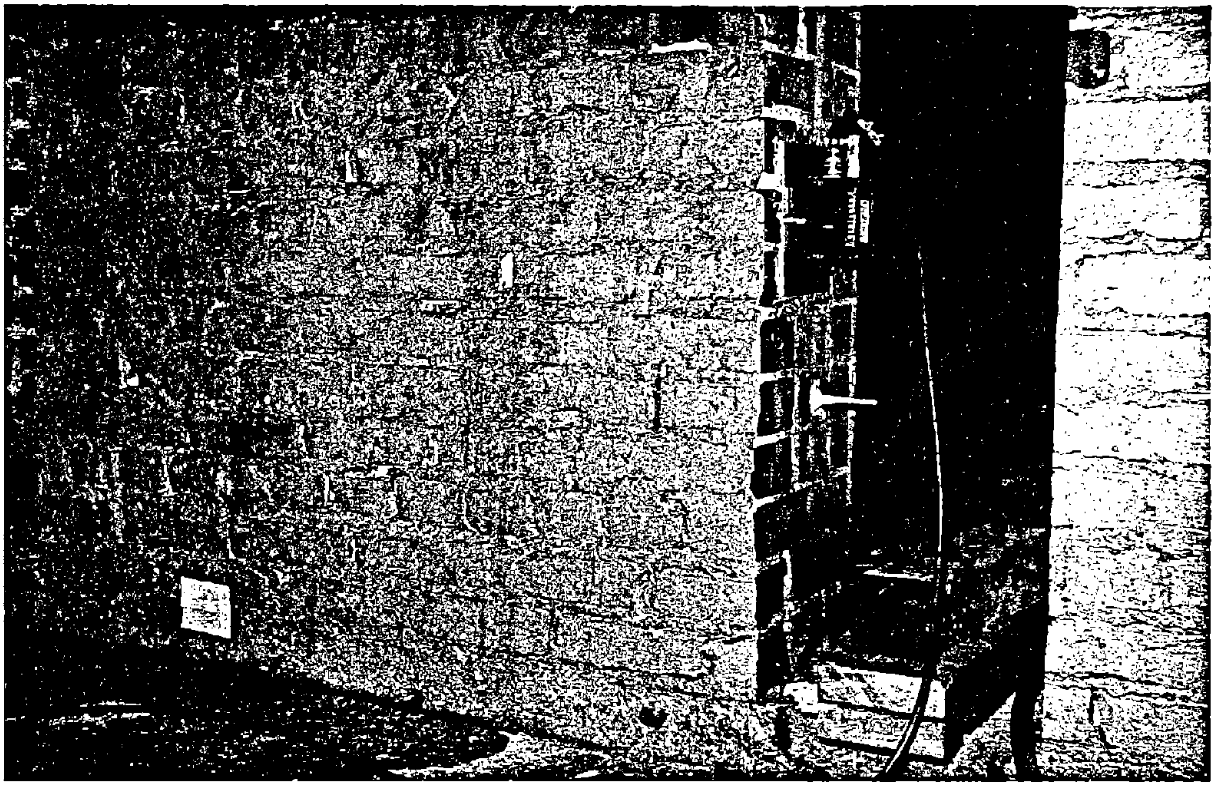


FIGURE C-4. MASONRY PIER WITH ATTACHED ACCELEROMETER

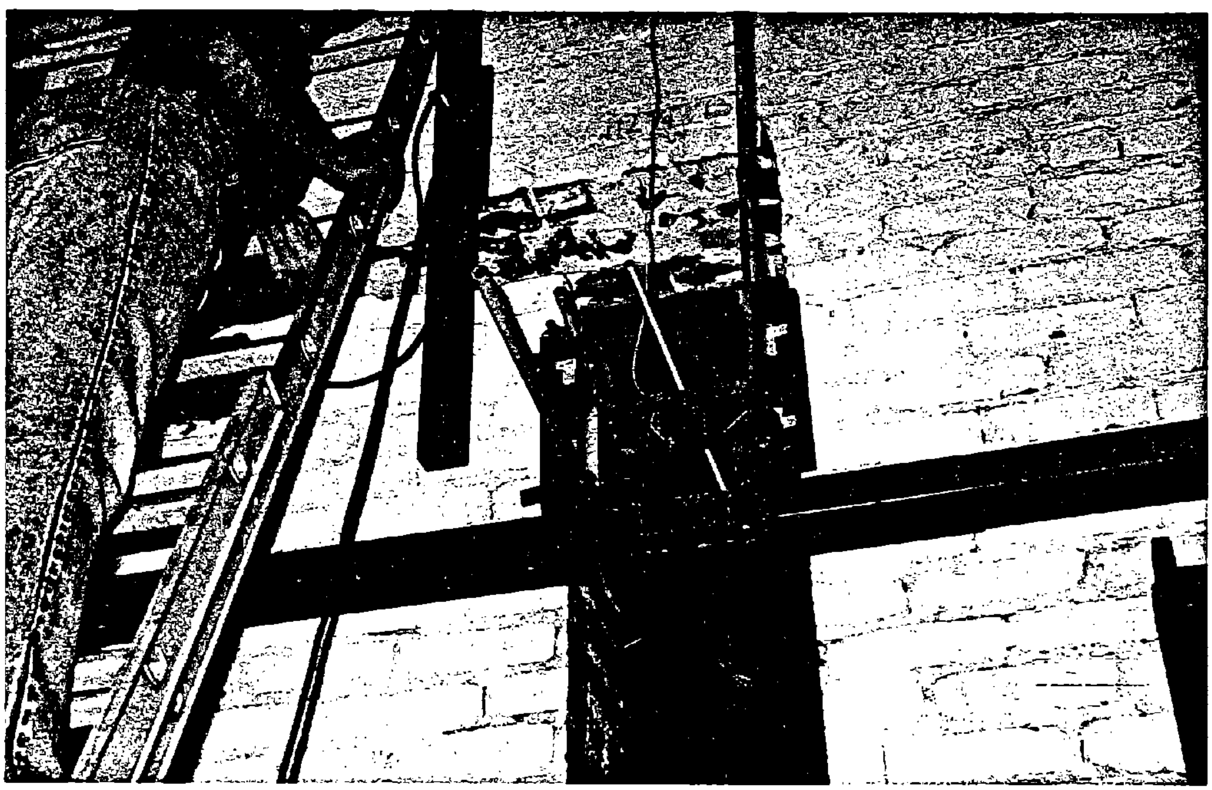


FIGURE C-5. MASONRY PIER WITH QUICK RELEASE DEVICE AT THE TOP

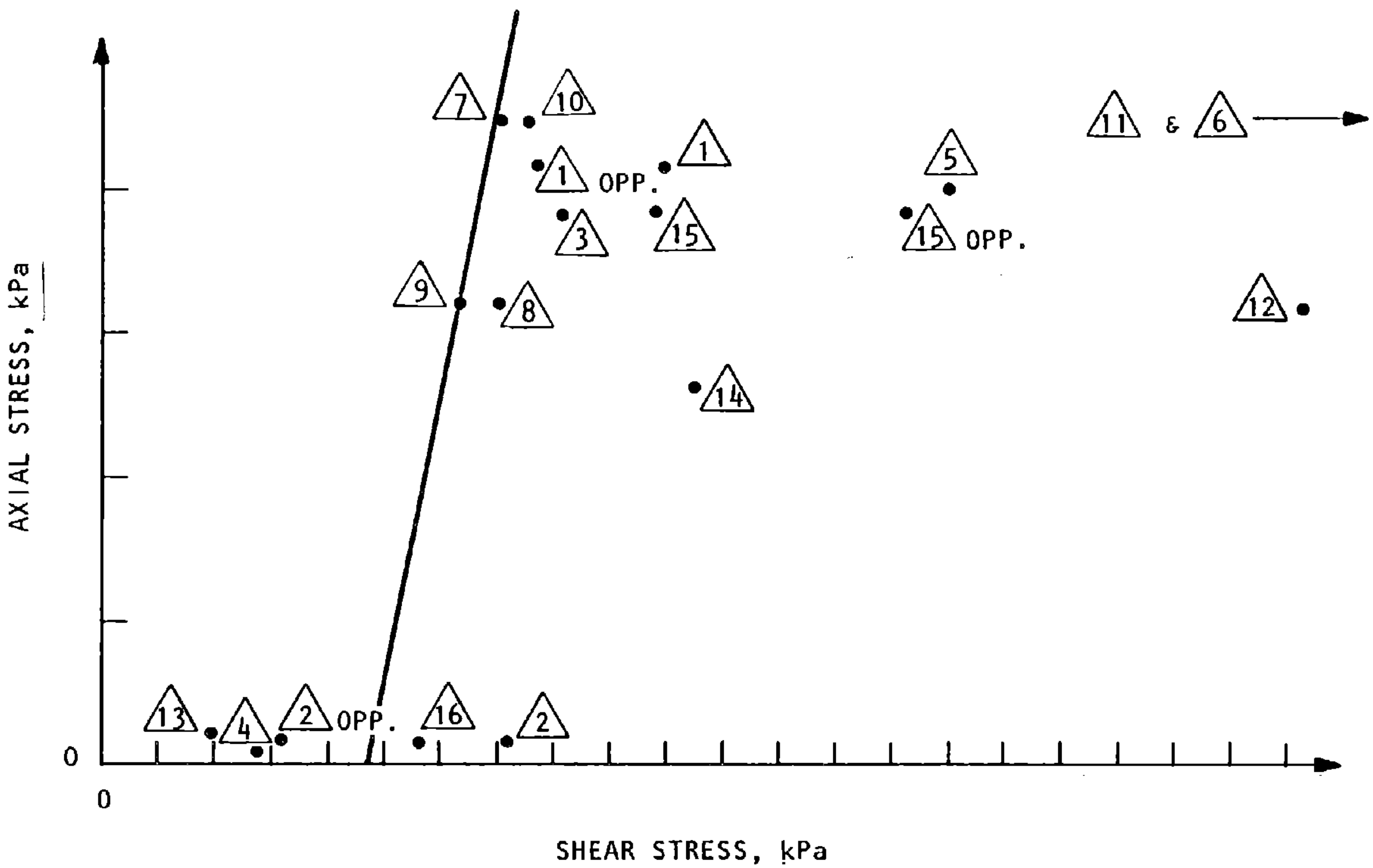
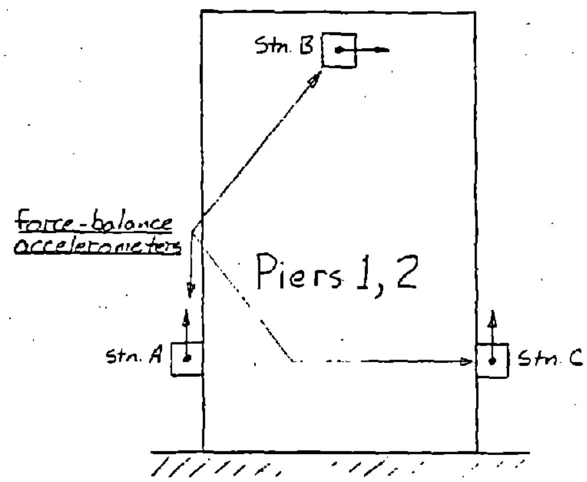


FIGURE C-6. PLOT OF URM IN-PLACE SHEAR TESTS



Data Run	Pier	Initial Deflection (inch)
2	1	1/8" left
3	1	1/4" left
4	1	3/8" left
5	1	1/2" left
6	1	1/4" right
7	2	.053" left
8	2	.136" left
9	2	.24" left
10	2	.388" left
11	2	.498" left
12	2	.60" left
13	2	.734" left

Note: Arrows indicate direction of motion which produces a positive accelerometer output.

FIGURE C-7. PIERS 1 AND 2, MEASUREMENT DESCRIPTION

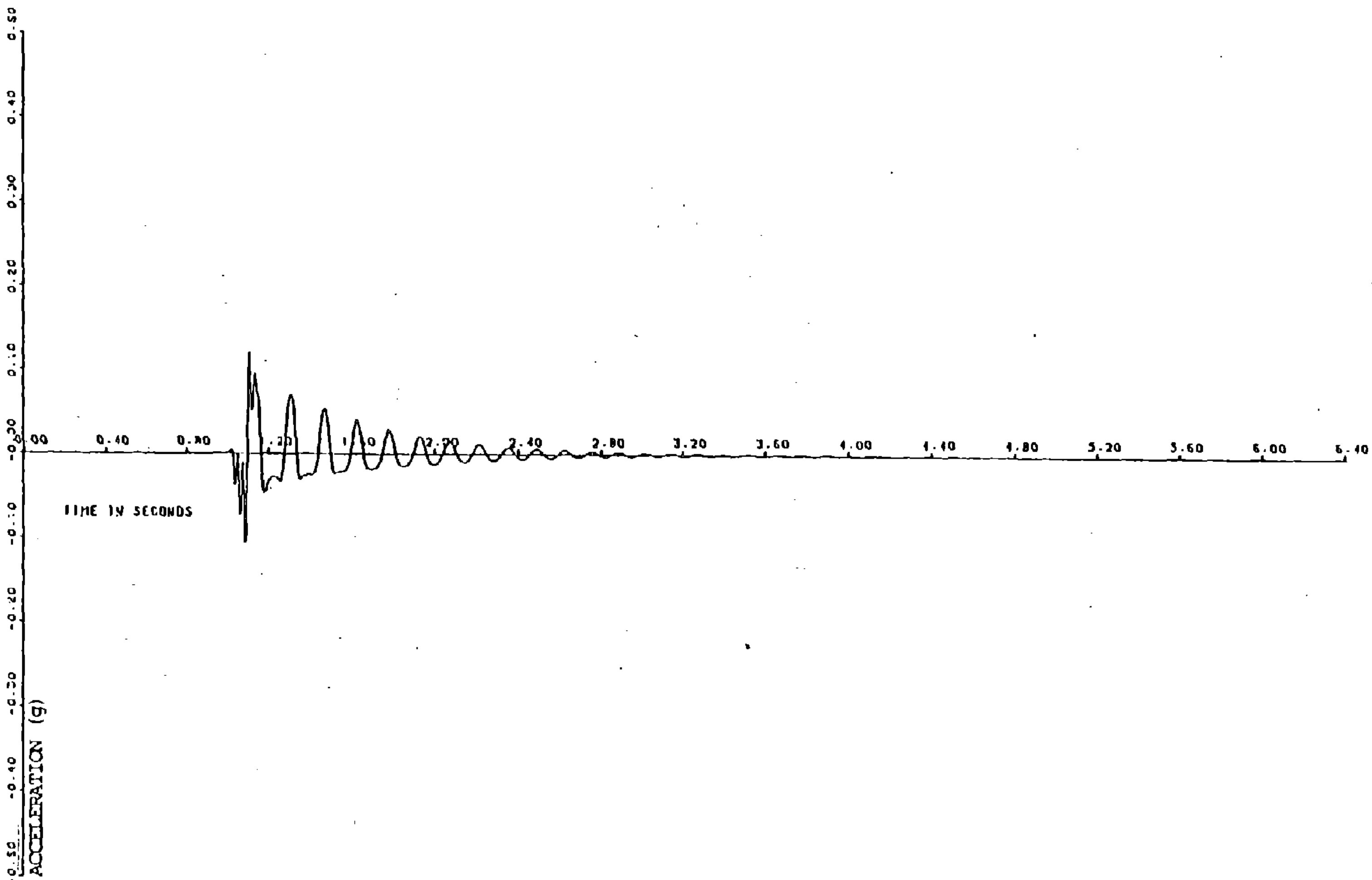


FIGURE C-8. DYNAMIC PIER TEST DATA RUN 2, STATION A (UP)

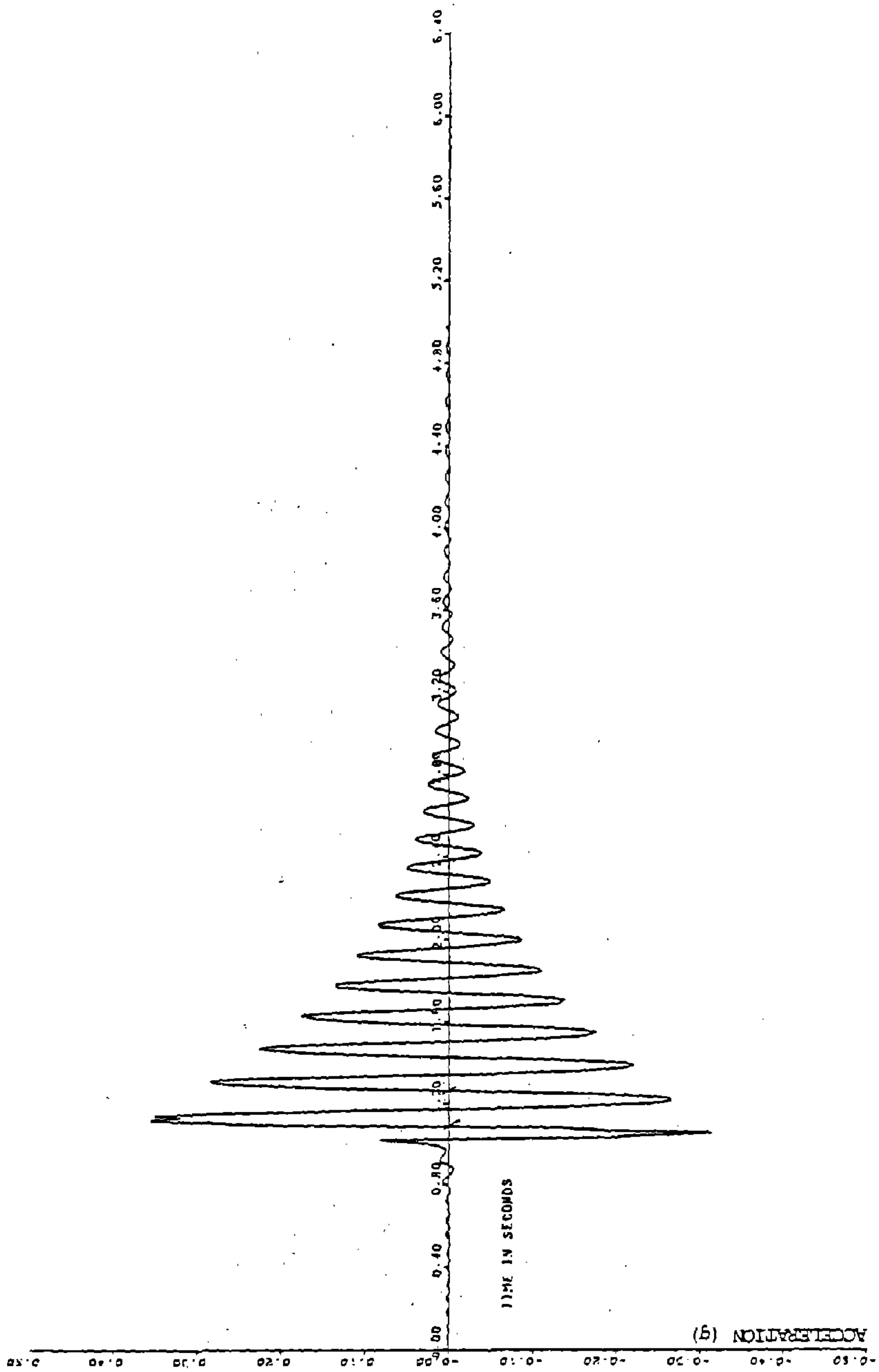


FIGURE C-9. DYNAMIC PIER TEST DATA RUN 2, STATION B (HOR)

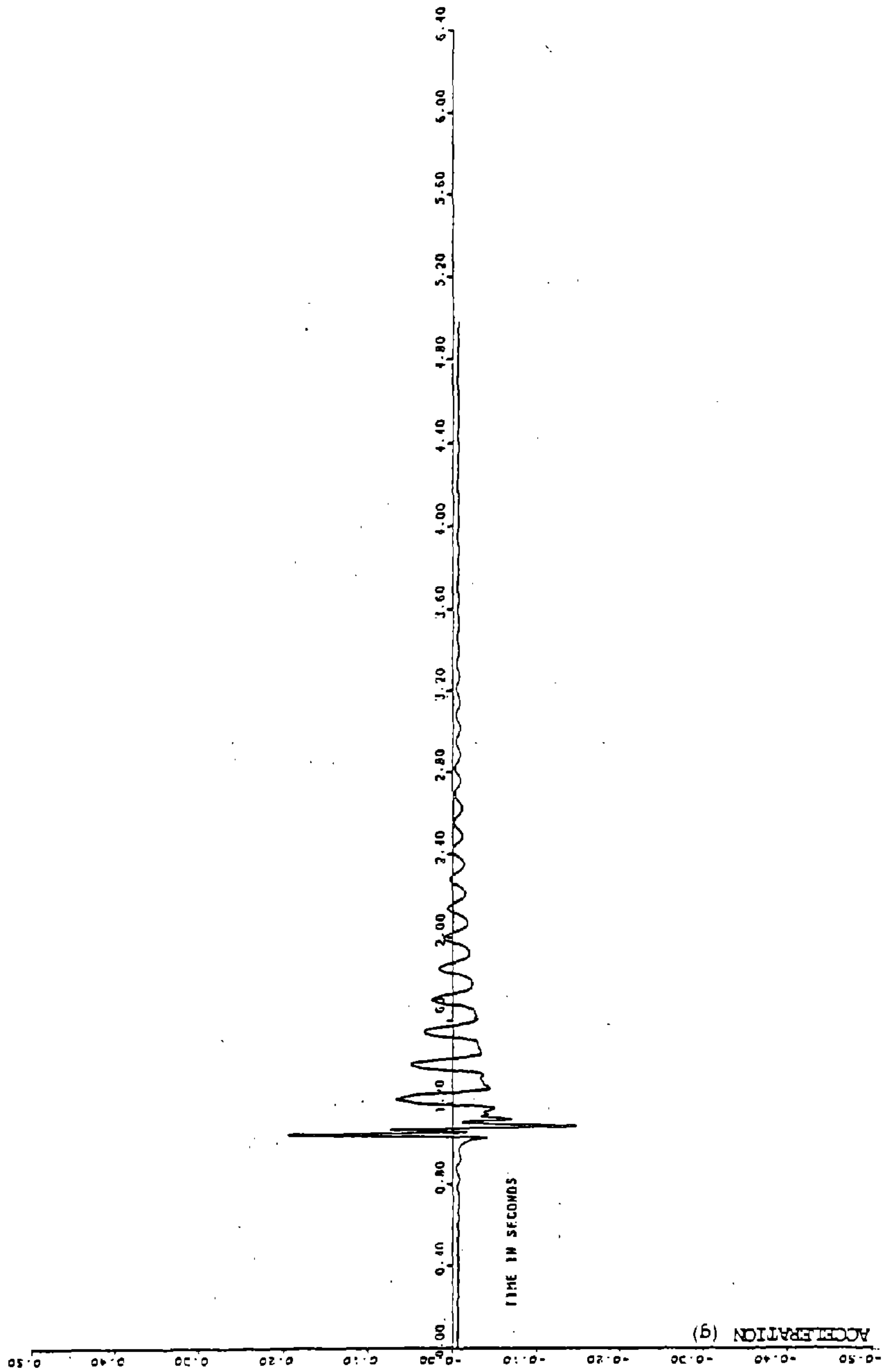


FIGURE C-10. DYNAMIC PIER TEST DATA RUN 2, STATION C (UP)

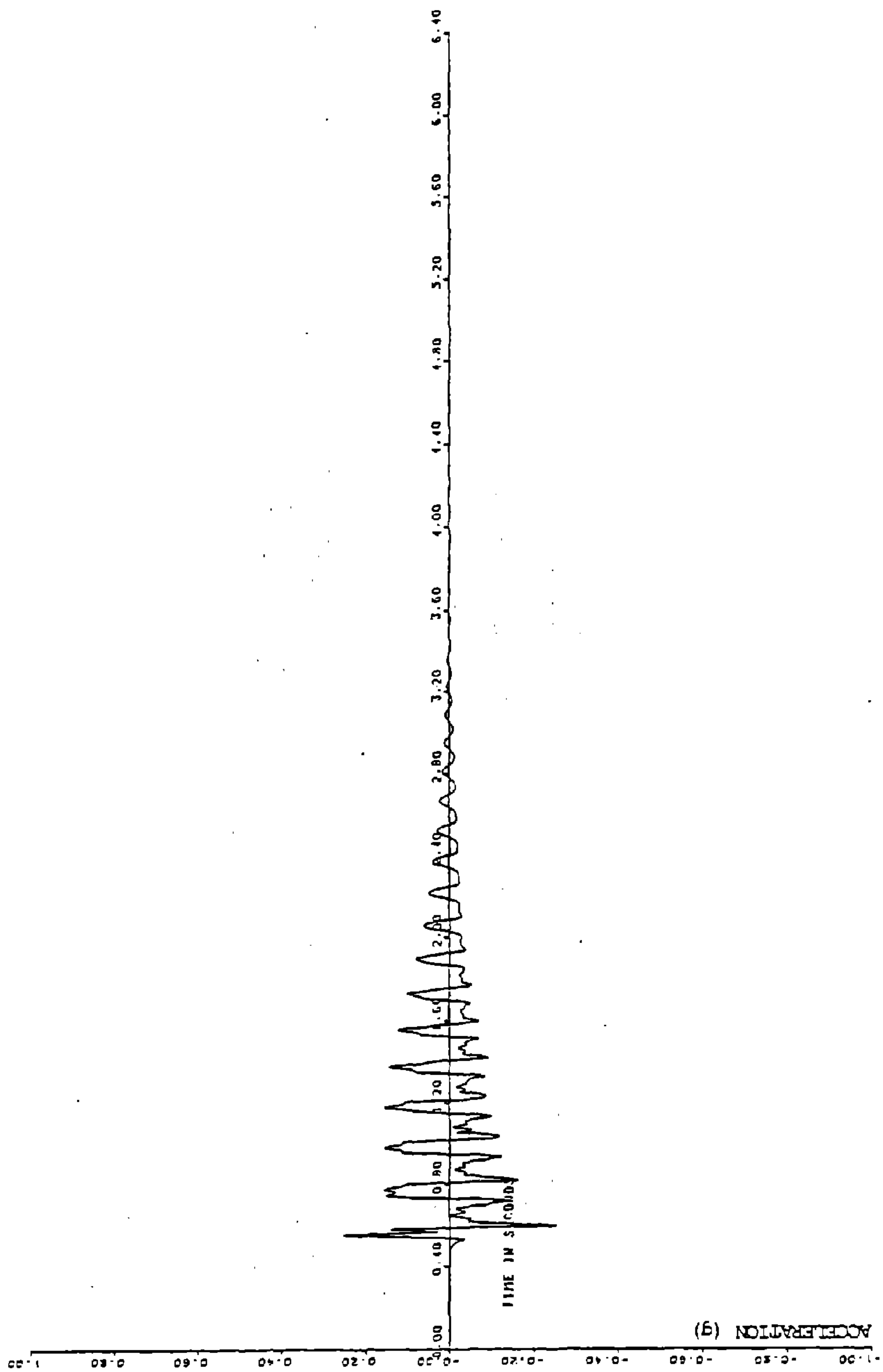


FIGURE C-11. DYNAMIC PIER TEST DATA RUN 6, STATION A (UP)

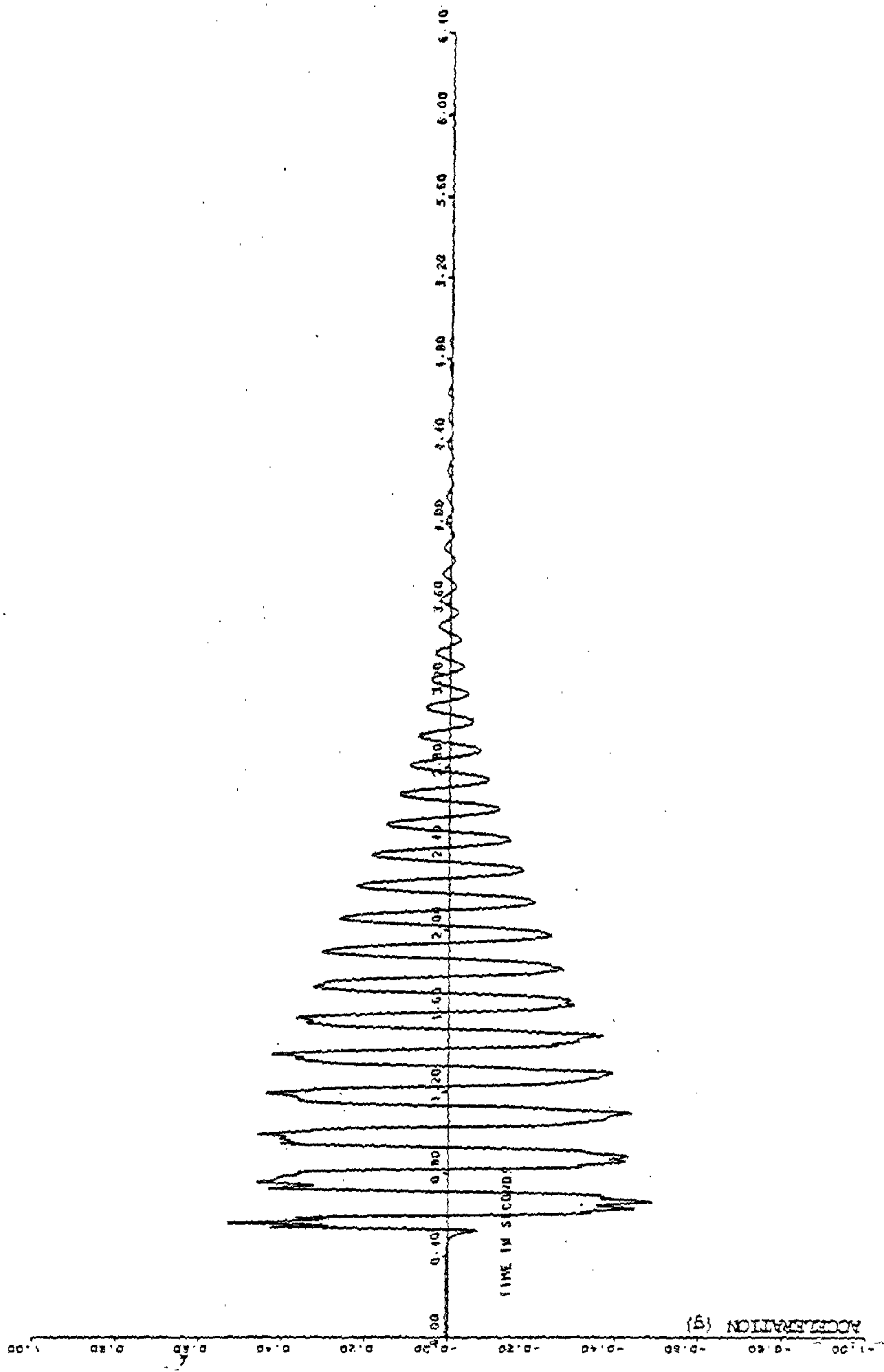


FIGURE C-12. DYNAMIC PIER TEST DATA RUN 6, STATION B (HOR)

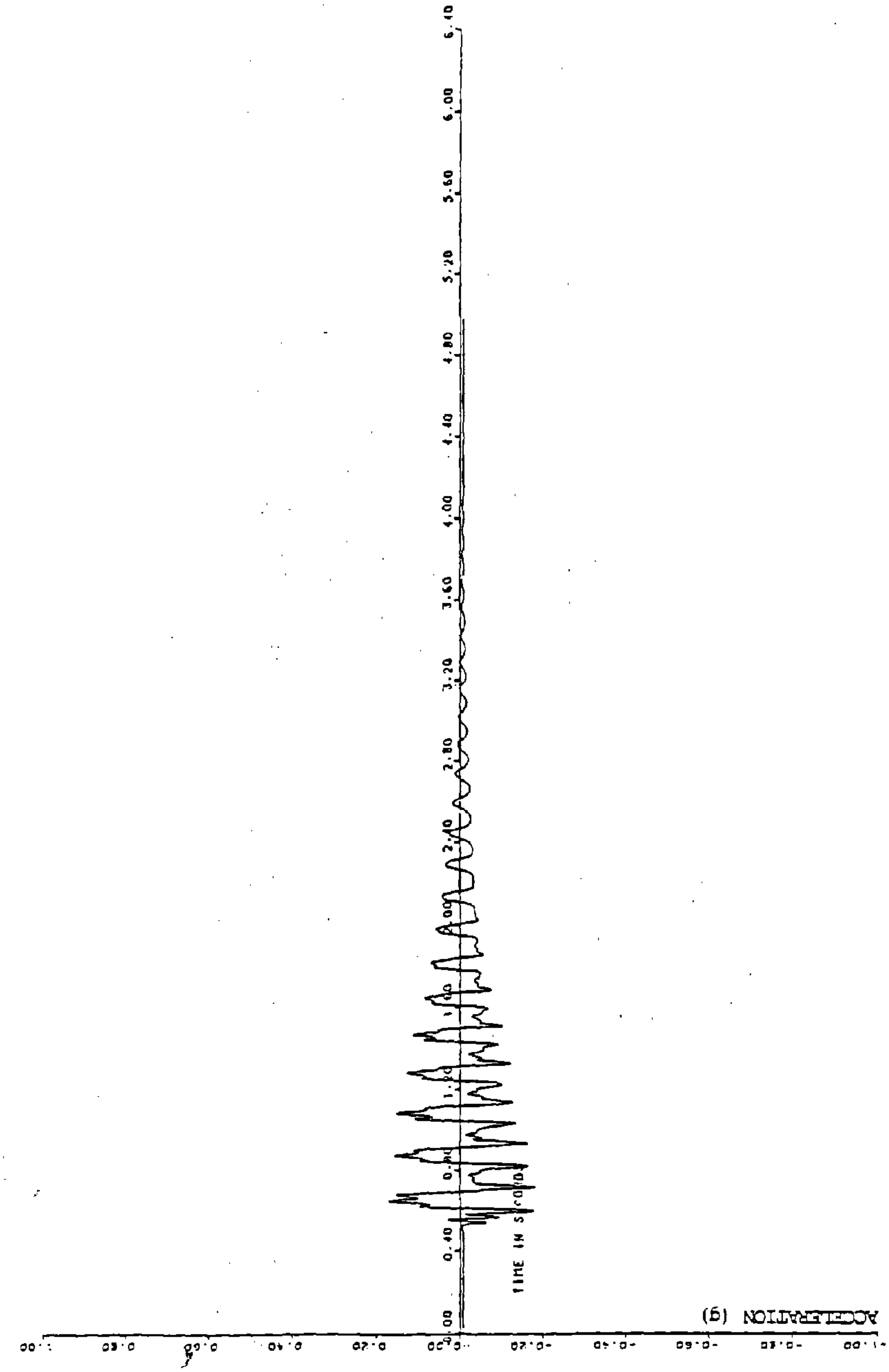


FIGURE C-13. DYNAMIC PIER TEST DATA RUN 6, STATION C (UP)

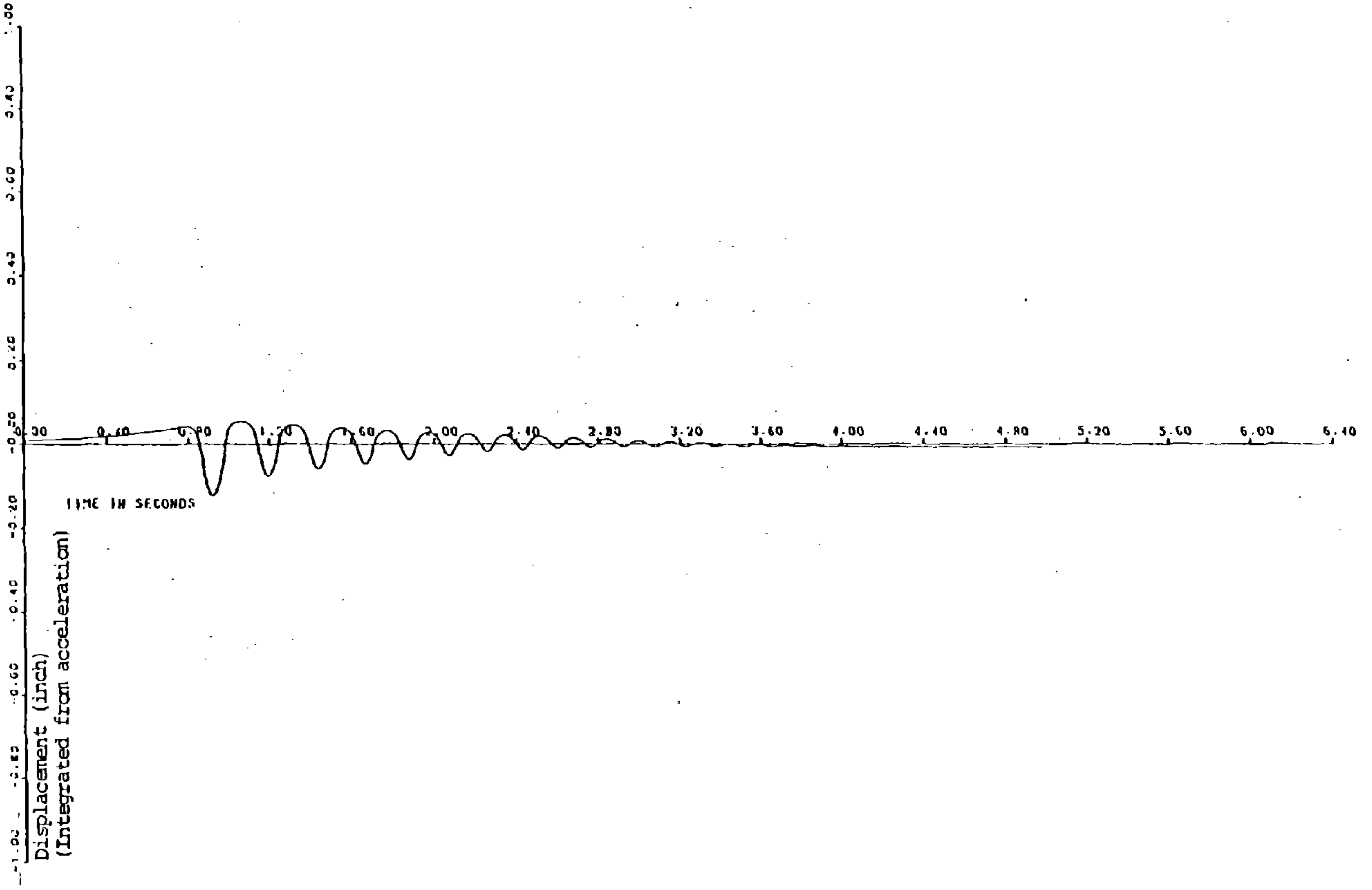


FIGURE C-14. DYNAMIC PIER TEST DATA RUN 5, STATION A (UP)

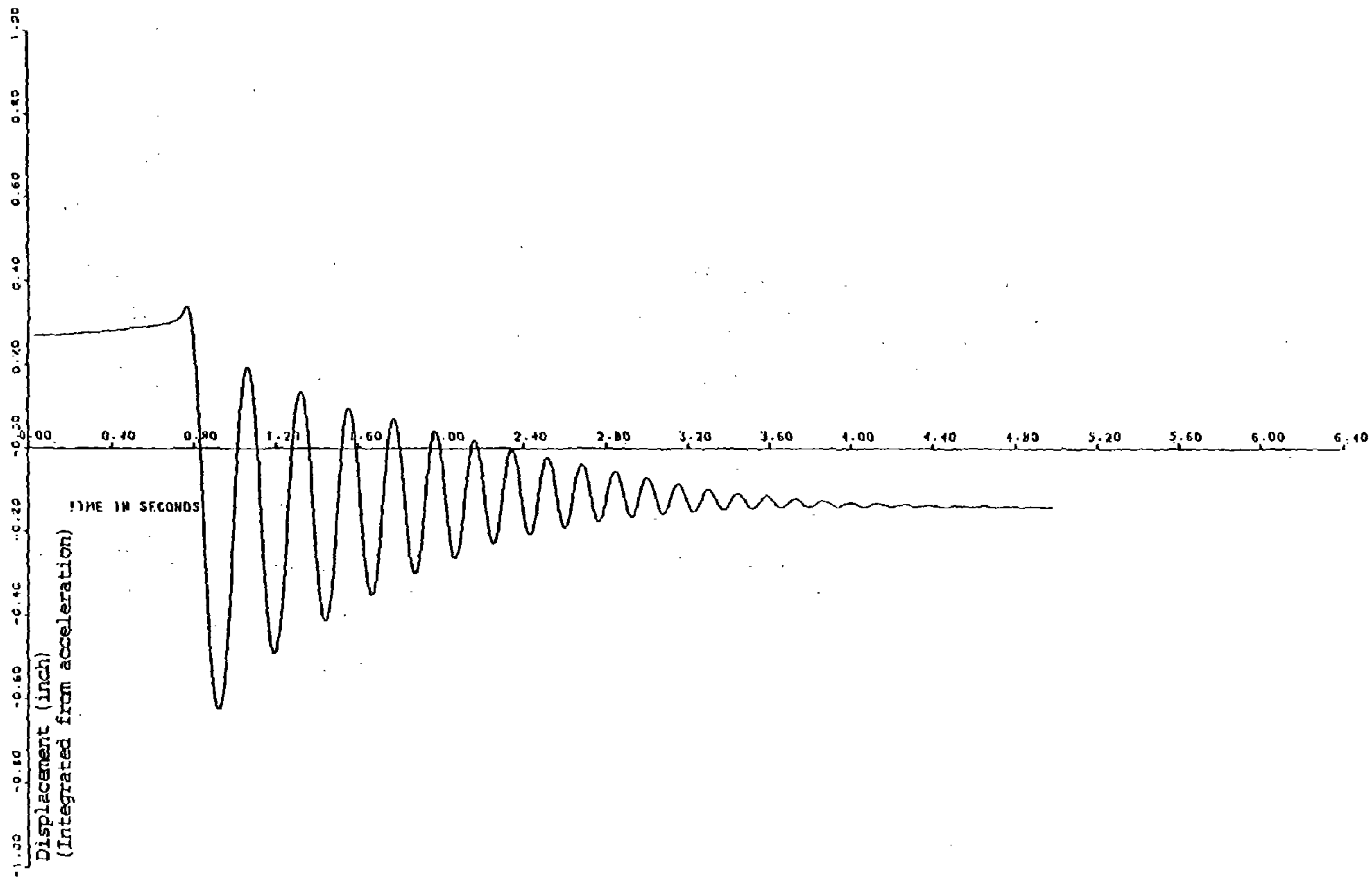


FIGURE C-15. DYNAMIC PIER TEST DATA RUN 5, STATION B (HOR)

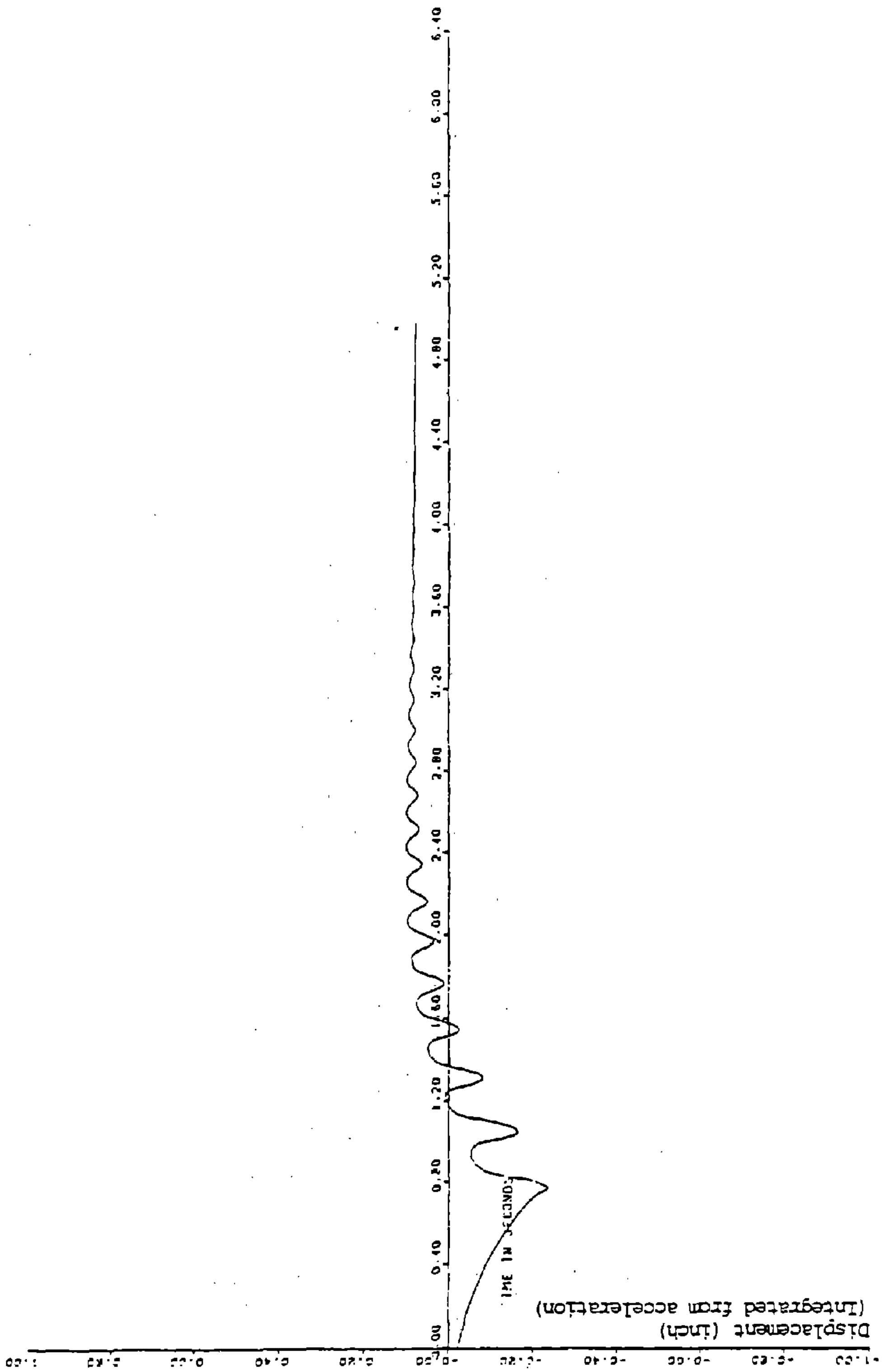


FIGURE C-16. DYNAMIC PIER TEST DATA RUN 5, STATION C (UP)

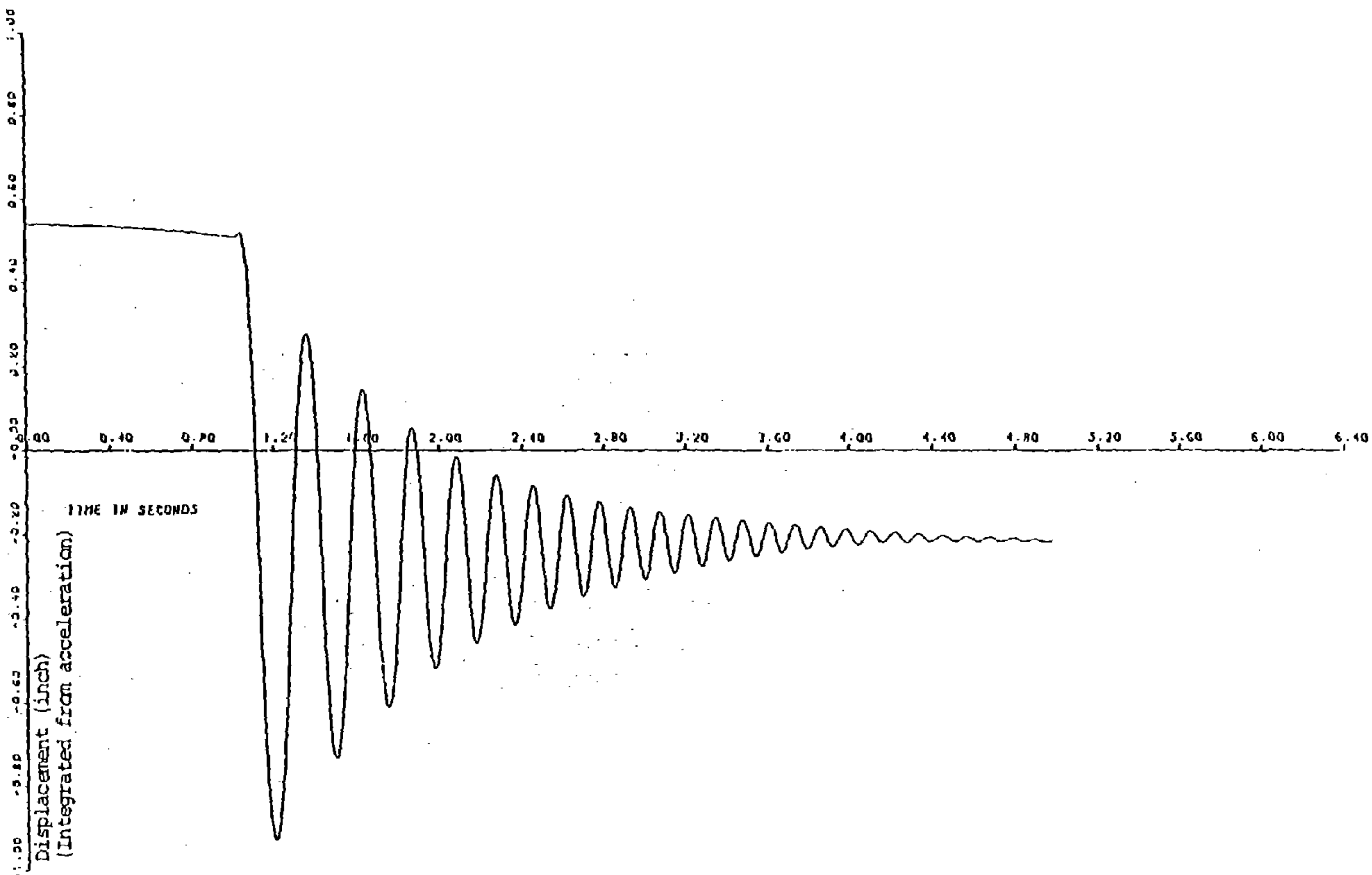


FIGURE C-17. DYNAMIC PIER TEST DATA RUN 13, PIER 2, STATION B (HOR)

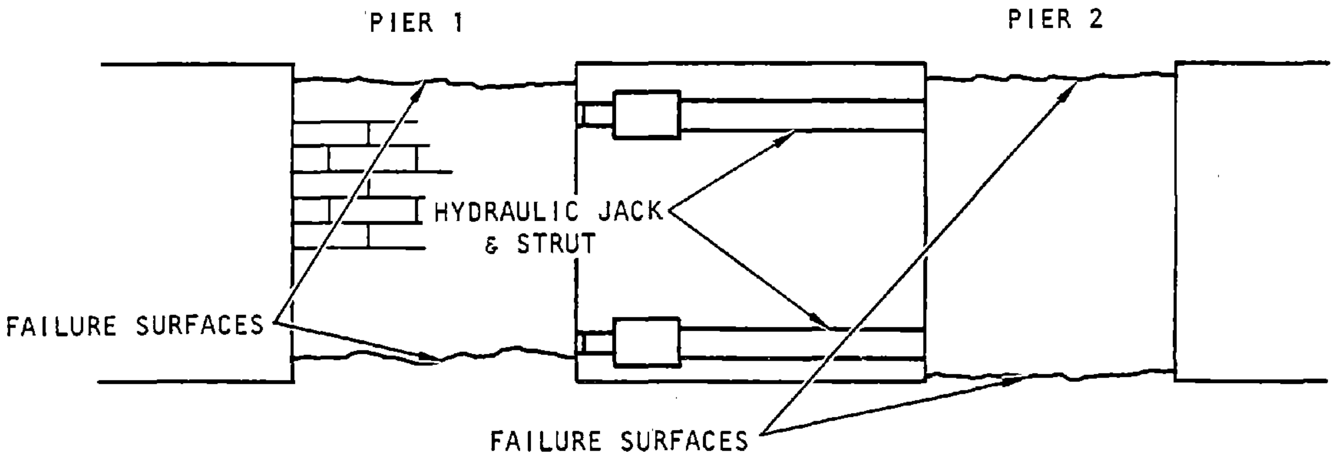


FIGURE C-18.

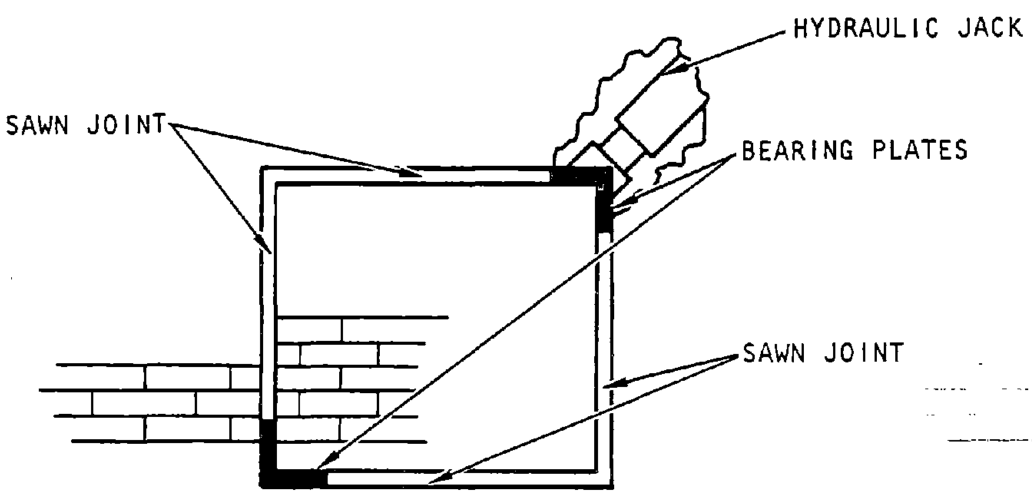
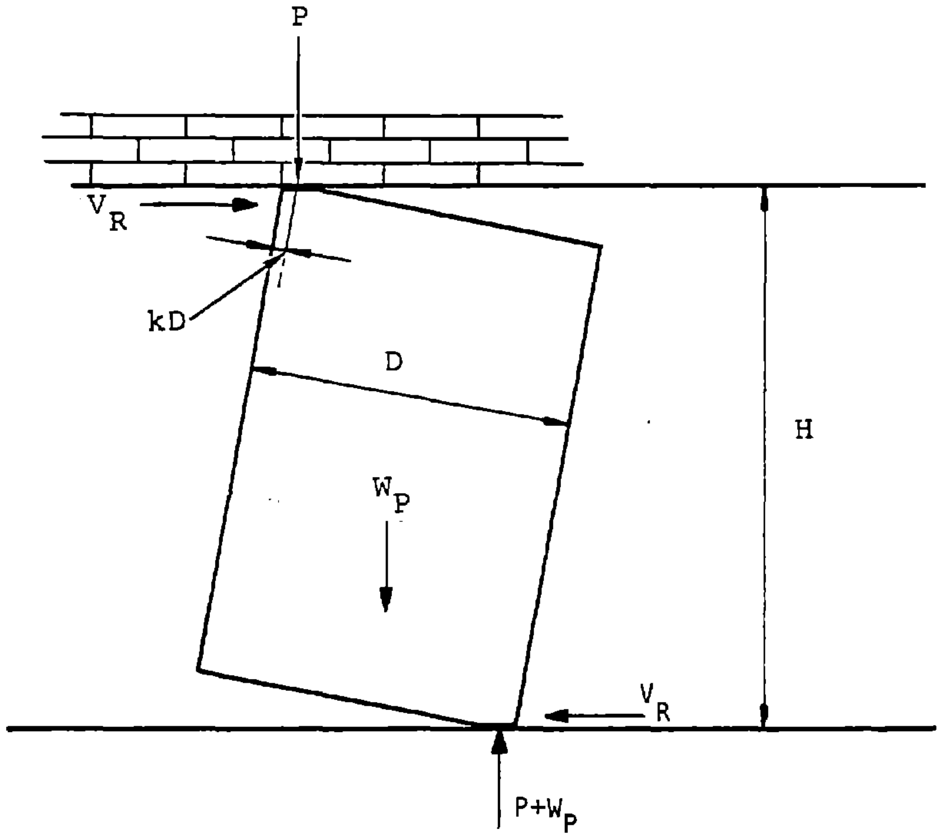


FIGURE C-19.



For small displacements

$$V_R = \frac{P \cdot (D - 2kD) + W_P \left(\frac{D}{2} - kD\right)}{H}$$

and if W_P is small in relation to P

$$V_R \approx P\ (1 - 2k)\ \left(\frac{D}{H}\right)$$

FIGURE C-20.

APPENDIX D

FINITE ELEMENT STUDIES OF URM WALL PIERS

SECTION D-1

INTRODUCTION

D-1.1 OBJECTIVES

These finite element studies were made to improve the understanding of stress distribution in pier elements subjected to axial loads and horizontal shear loads. Isotropic material properties were used for the studies. It is recognized that URM elements are very probably anisotropic and that the stresses indicated by the finite element studies are general. Probable stress concentrations caused by the properties of mortar and brick will be accommodated by factoring of basic tested stresses and average failure stresses that were determined by in-place testing of existing URM. The procedures and results of in-place testing of URM are reported in Appendix C.

D-1.2 ANALYSIS MODEL AND PROCEDURE

The URM wall piers were analyzed using the BMINES finite element computer code (AA, 1981a). The piers were modeled using two-dimensional, plane stress, quadrilateral elements. Although it is recognized that typical URM walls have nonlinear anisotropic properties and the BMINES code has this capacity, the actual properties are not sufficiently known to warrant using these properties. Accordingly, linear elastic properties were used in the analysis and the analysis results were used only to infer general states of stress and stress distributions.

The dimensions and configurations of the piers are given in Figures D-1, D-7, D-13, and D-19. As shown in these figures, the piers were supported on a concrete foundation similar to that in the actual pier tests. The gravity loads (self-weight) were applied to the model by loading each finite element with its weight. The vertical and horizontal jack forces were then applied simultaneously as concentrated forces at the appropriate nodes of the model.

The output of interest from these analyses are stresses and stress distributions; the stresses of interest are given in this appendix.

SECTION D-2

ANALYSIS RESULTS

D-2.1 PIER 1

Data developed by finite element studies of the static shear testing of Pier 1 is presented by the following figures:

Figure D-1 indicates the pier model and loading.

Figure D-2 indicates the shear stress at a section half-way between the horizontal load and the base.

Figure D-3 plots principal stresses in the shear zone.

Figure D-4 plots horizontal (x-x) stresses in the shear zone.

Figure D-5 plots vertical (y-y) stresses in the shear zone.

Figure D-6 plots shear (x-y) stresses in the shear zone.

The shear crack is assumed to have originated at a flaw in the contours related to Figures D-5 and D-6. The critical zone is assumed to be related to highest shear and least axial load (y-y) normal to the bed joint.

D-2.2 PIER 2

Data for Pier 2 is presented in the same format and sequence as Pier 1 by Figures D-7 through D-12.

D-2.3 PIER 3

Data for Pier 3 is presented in the same format and sequence as Pier 1 by Figures D-13 through D-18.

D-2.4 PIER 4

Data for Pier 4 is presented in the same format and sequence as Pier 1 by Figures D-19 through D-24.

SECTION D-3

CORRELATION OF THE FINITE ELEMENT ANALYSIS AND IN-PLACE TESTING OF EXISTING URM

D-3.1 RELATIONSHIP OF AVERAGE HORIZONTAL SHEAR AND FINITE ELEMENT ANALYSIS RESULTS

Shear in masonry or concrete elements is typically calculated as an average shear, $v = V \div A$. The results of the finite element studies indicate that the critical shear to be considered in analysis of URM piers is better estimated by $v = 1.5\ V \div A$. Review of diagonal compression specimens previously tested (App. C) also indicates that critical bed-joint shear is about 1.5 times average shear. It is recommended that critical bed-joint shear for URM piers be calculated as $1.5\ V \div A$. This approximation of critical shear vs. average shear will overestimate the critical shear in long walls without openings. An example of such a wall would be the exterior wall of a URM building on an interior property line. Generally, in-plane shear stress in these walls is not critical. If a critical condition is discovered in the analyses due to special conditions, it is suggested that the 1.5 factor be reduced to 1.0 for solid walls with height/length factors of 0.5 or less.

D-3.2 RELATIONSHIP OF IN-PLACE SHEAR TESTING OF EXISTING URM AND FINITE ELEMENT ANALYSIS RESULTS

For the correlation of test and analysis results, it is assumed that bed-joint shear in existing URM piers has a relationship to the basic tested shear and the axial load normal to the bed joint of

$$v_a = k\,(r \cdot v_t + \phi\, P/A)$$

where

k = Constant < 1 to adjust acceptable shear for workmanship flaws

r = Reduction factor to adjust tested values for probable bonding on the collar joint.

v_t = Basic bed-joint shear stress as determined by in-place testing. v_t is reduced to an equivalent shear for zero axial load applied normal to the bed joint.

ϕ = Factor = 1.0 to account for increase in shear due to applied loads normal to the shear surface.

P/A = Axial stress normal to the bed joint.

In-place shear testing has determined that the critical shear stress is approximately 1.5 times the shear stress calculated as an average shear. Comparison of the results of current in-place shear tests and prior in-plane shear indicates that test values determined by recommended procedures should be reduced to account for probable bonding of the tested brick on the collar joint.

The available data obtained from current large-scale testing of a URM building in the City of Los Angeles is as follows:

Basic in-place tested shear stress = 47 psi

When corrected for probable bonding on the collar joint by r = 3/4, then this shear stress = 35 psi. Shear and normal stresses as determined by finite element studies of the tested piers are:

	σ_{xy}	σ_{yy}
Pier 1	36 psi	25 psi
Pier 2	33 psi	15 psi
Pier 3	36 psi	15 psi
Pier 4	54 psi	25 psi

From prior in-place test programs:

Basic in-place tested shear stress = 30 psi

Tested bricks were separated at collar joints prior to testing. The results are:

	σ_{xy}	σ_{yy}
Pier 1	34.4	11.2
Pier 2	41.1	16.2
Diagonal Compression Specimen 1	37 psi	28 psi
Diagonal Compression Specimen 2	53 psi	39 psi

Using the data in the formula:

$v_a = k\ (3/4\ v_t + P/A)$ and setting v_a equal to tested capacity

$k_{mean} = 0.75$ with a standard deviation of 0.11

If only current data is used:

$k_{mean} = 0.73$

If only prior data is used:

$k_{mean} = 0.78$

SECTION D-4

RECOMMENDATIONS FOR DETERMINATION OF ACCEPTABLE IN-PLACE SHEAR STRESSES FOR EXISTING URM

The recommendations of the methodology for determination of v_a are:

Perform in-place shear tests of the existing URM in accordance with Section 4.5.6 of the methodology.

Determine the basic bed-joint shear, v_t, by selecting the 20th percentile of the tested shear values when reduced to equivalent zero axial stress normal to the bed joint. See Section 9.6 of the methodology.

Determine allowable bed-joint shear, v_a, for tested URM piers and walls as:

$$v_a = \frac{3}{4}\left(\frac{3}{4} v_t + \frac{P}{A}\right)$$

Compare the allowable bed-joint shear, v_a, with the analysis shear calculated as $v = 1.5\ V/A$.

The recommendations were developed from the minimum available test data. Further research to improve the understanding of development of shear cracking in anisotropic masonry materials is needed to improve the data base. The development of the current recommendations has been strongly influenced by the principal investigators' examination of URM buildings shaken by moderate to strong ground motions. These on-site investigations indicate that diagonal shear offsets in wall piers that provide the ground motion excitations to the aboveground building are not common; this is so even in observed cases where nonfailure cannot be rationalized by computations. A partial rationalization can be made by recognizing that the analysis assumes recorded free-field ground motions as input into the base of the building. This assumption probably overstates the effectiveness of the soil medium to transfer high frequency energy to the building base.

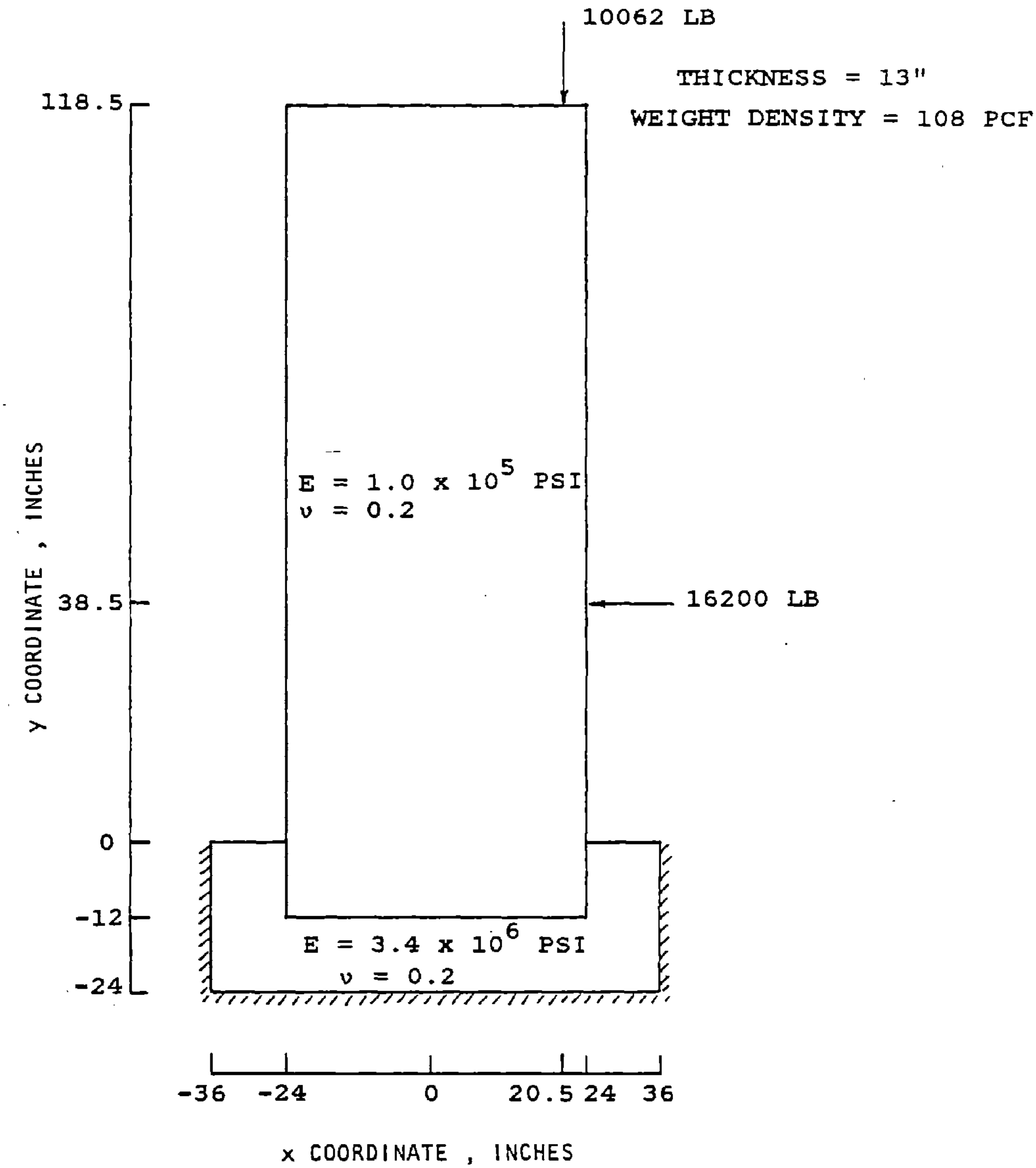


FIGURE D-1. MODEL FOR PIER 1

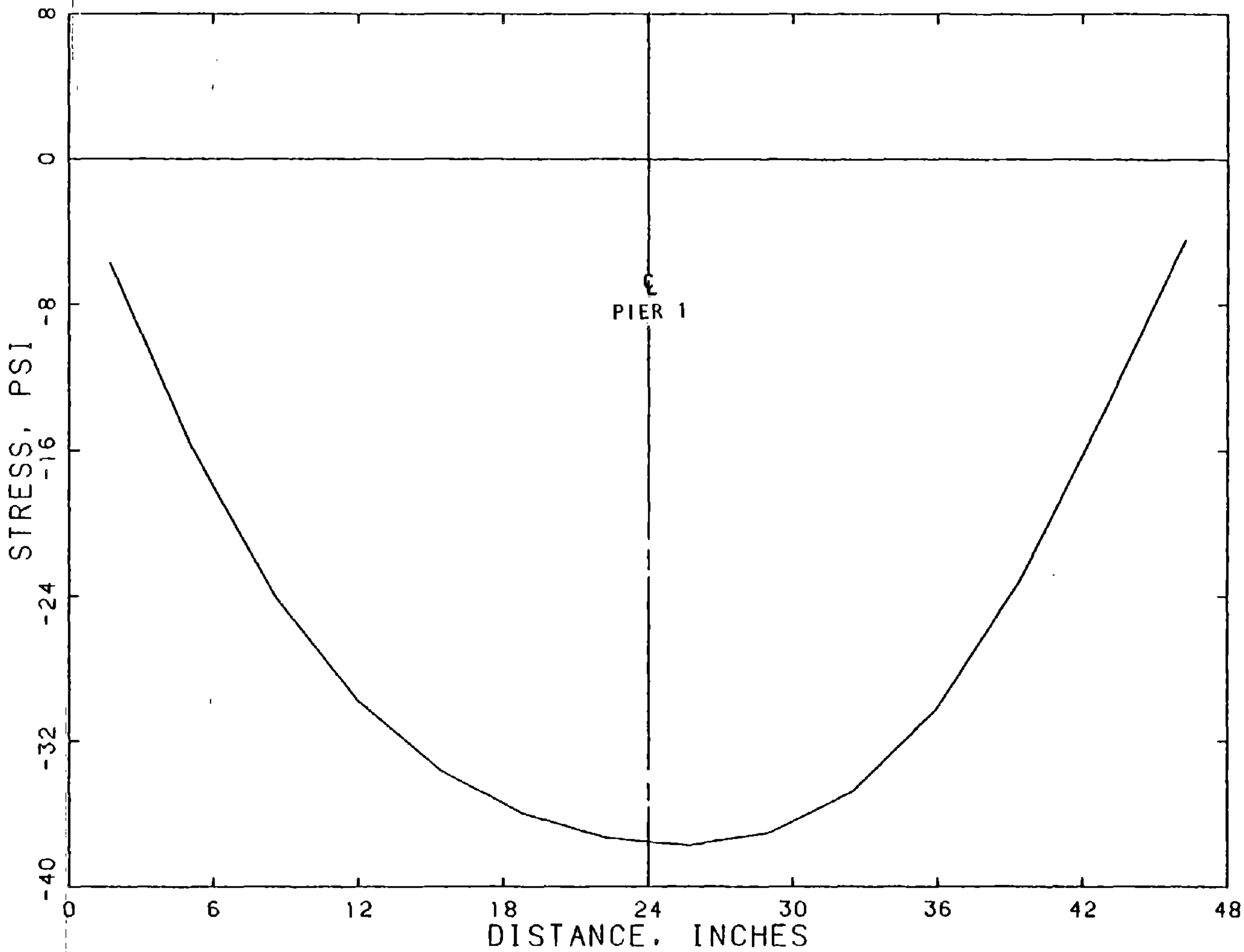


FIGURE D-2. SHEAR STRESS (τ_{xy}) IN PIER 1 AT 19.25" ABOVE BASE

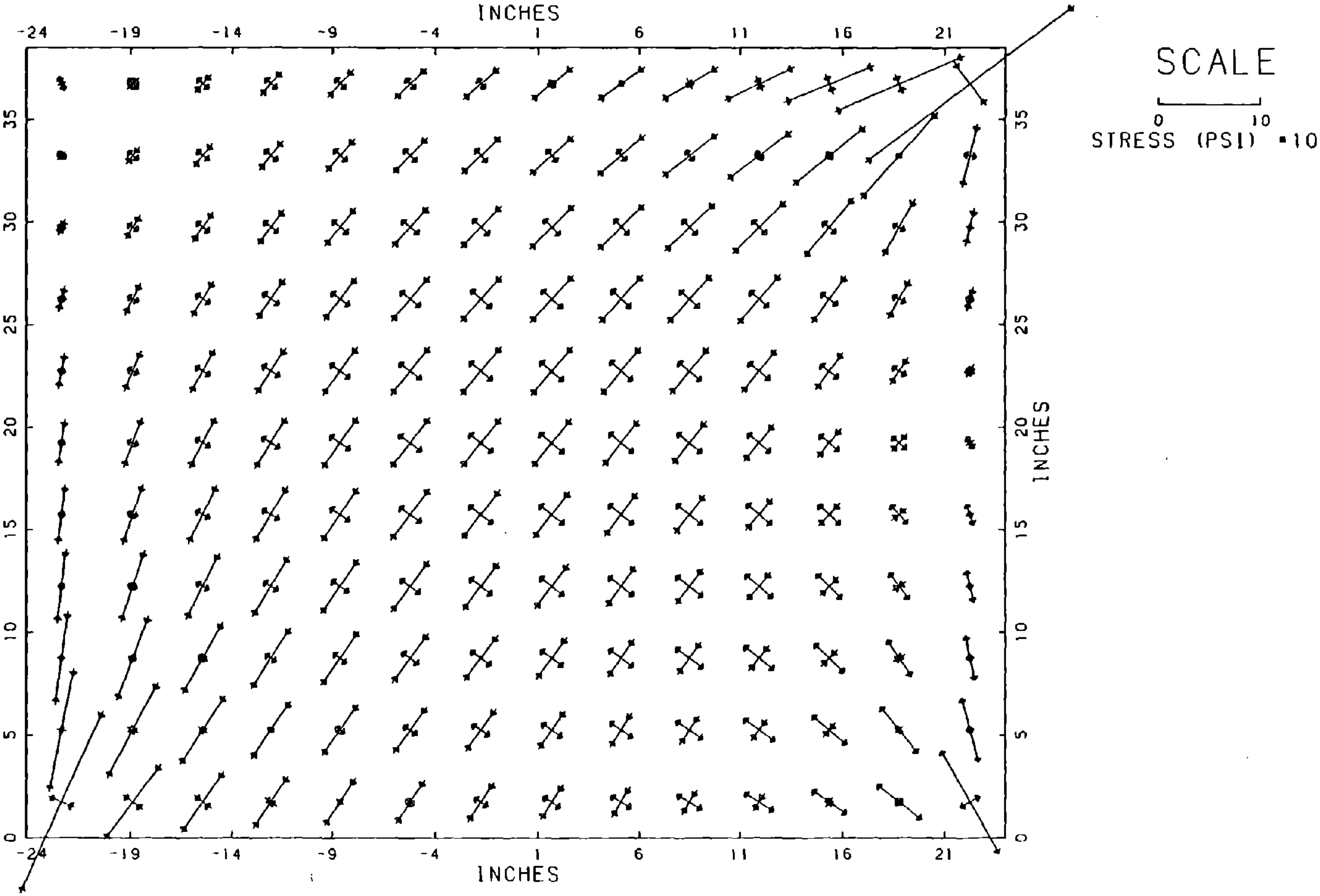


FIGURE D-3. PRINCIPAL STRESSES IN PIER 1

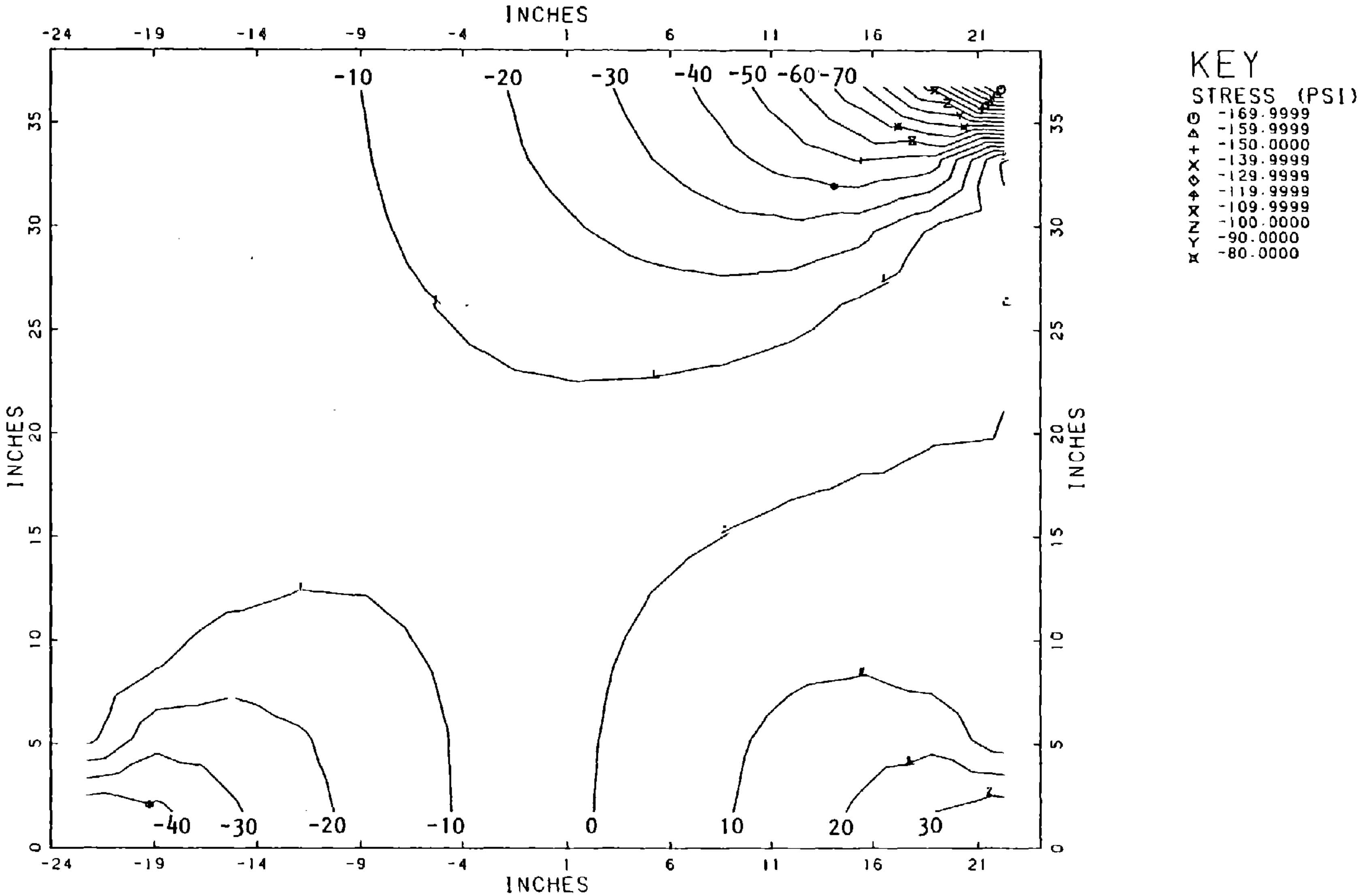


FIGURE D-4. HORIZONTAL STRESS (σ_{xx}) IN PIER 1

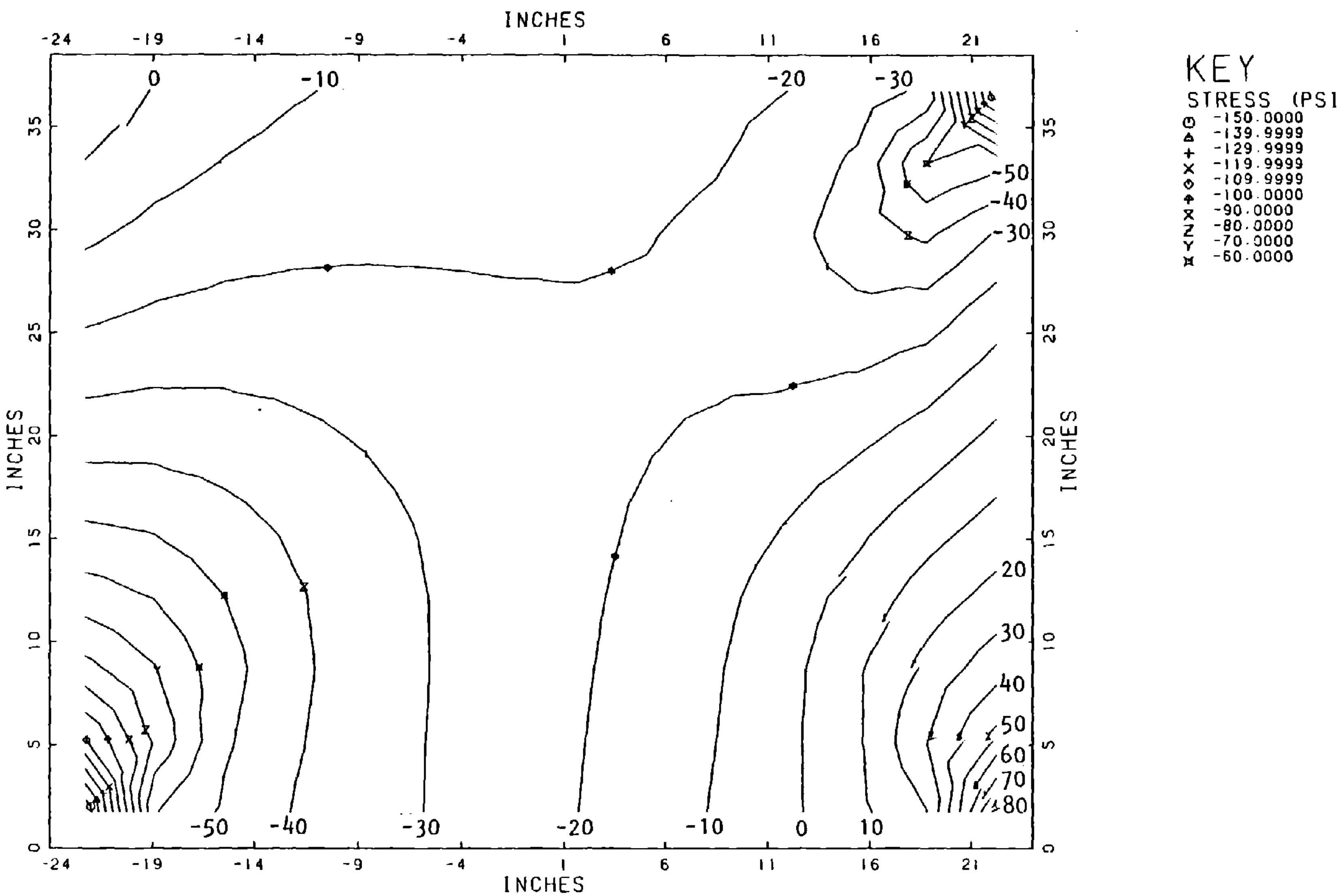


FIGURE D-5. VERTICAL STRESS (σ_{yy}) IN PIER 1

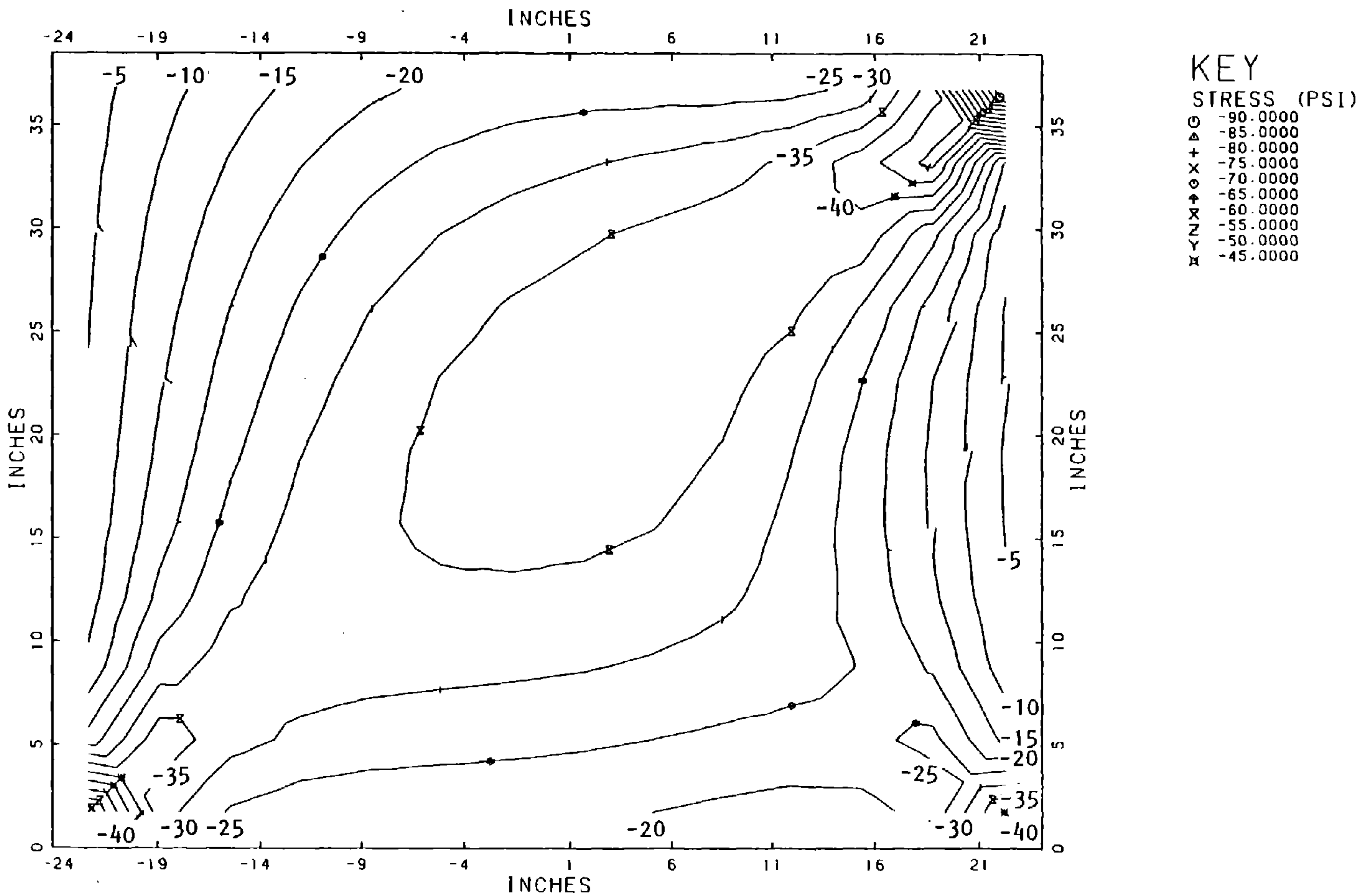


FIGURE D-6. SHEAR STRESS (τ_{xy}) IN PIER 1

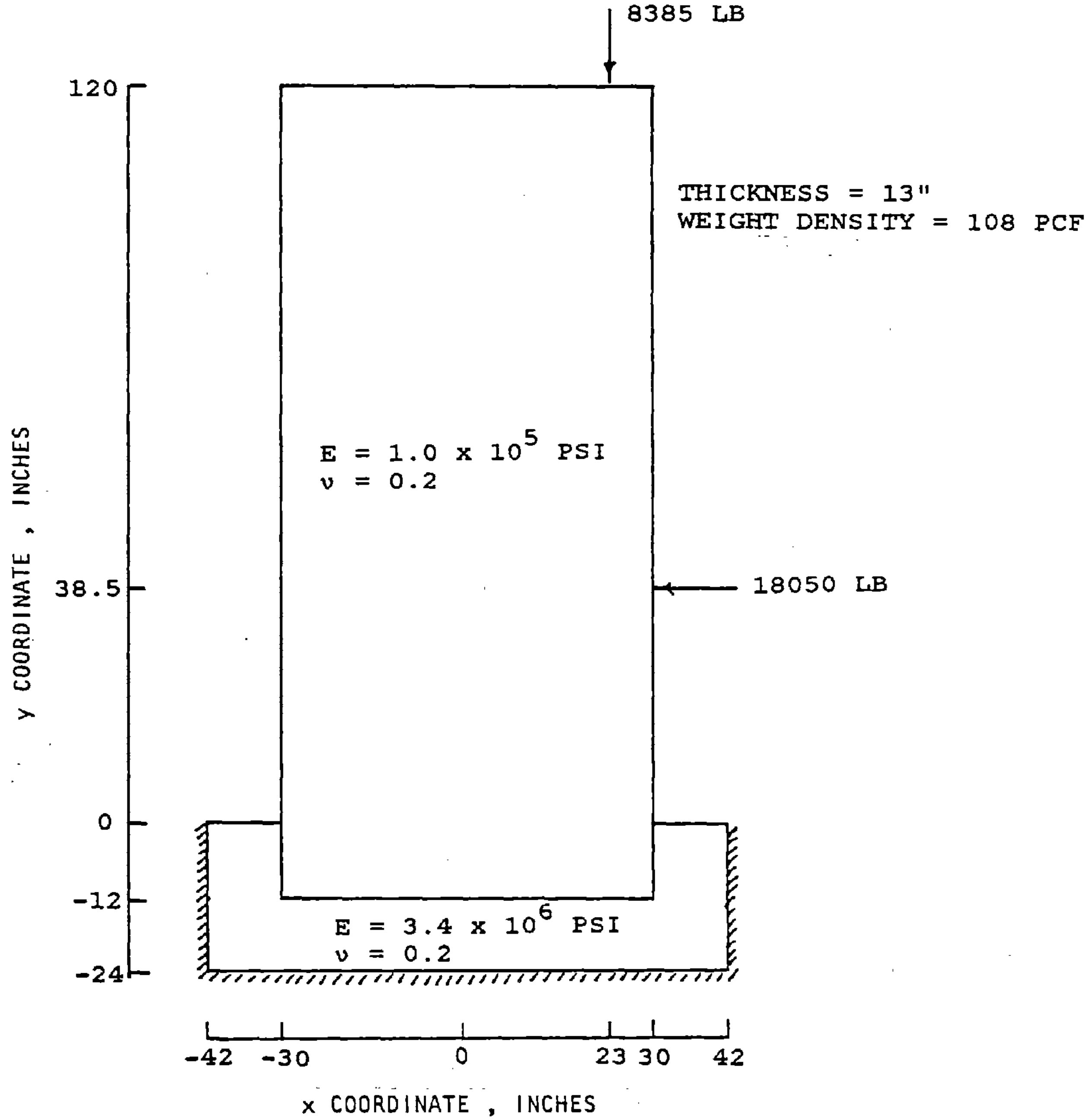


FIGURE D-7. MODEL FOR PIER 2

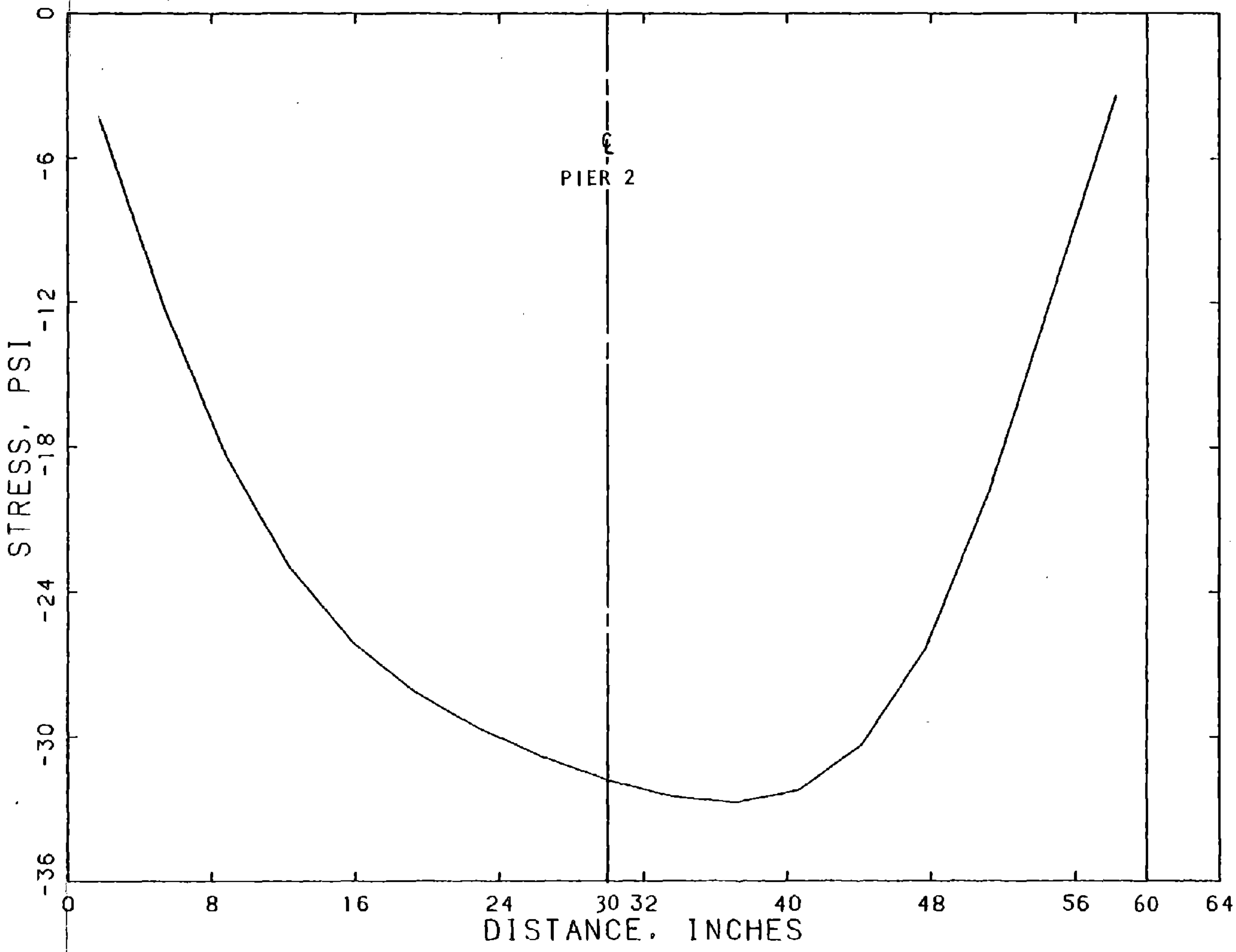


FIGURE D-8. SHEAR STRESS (τ_{xy}) IN PIER 2 AT 19.25" ABOVE BASE

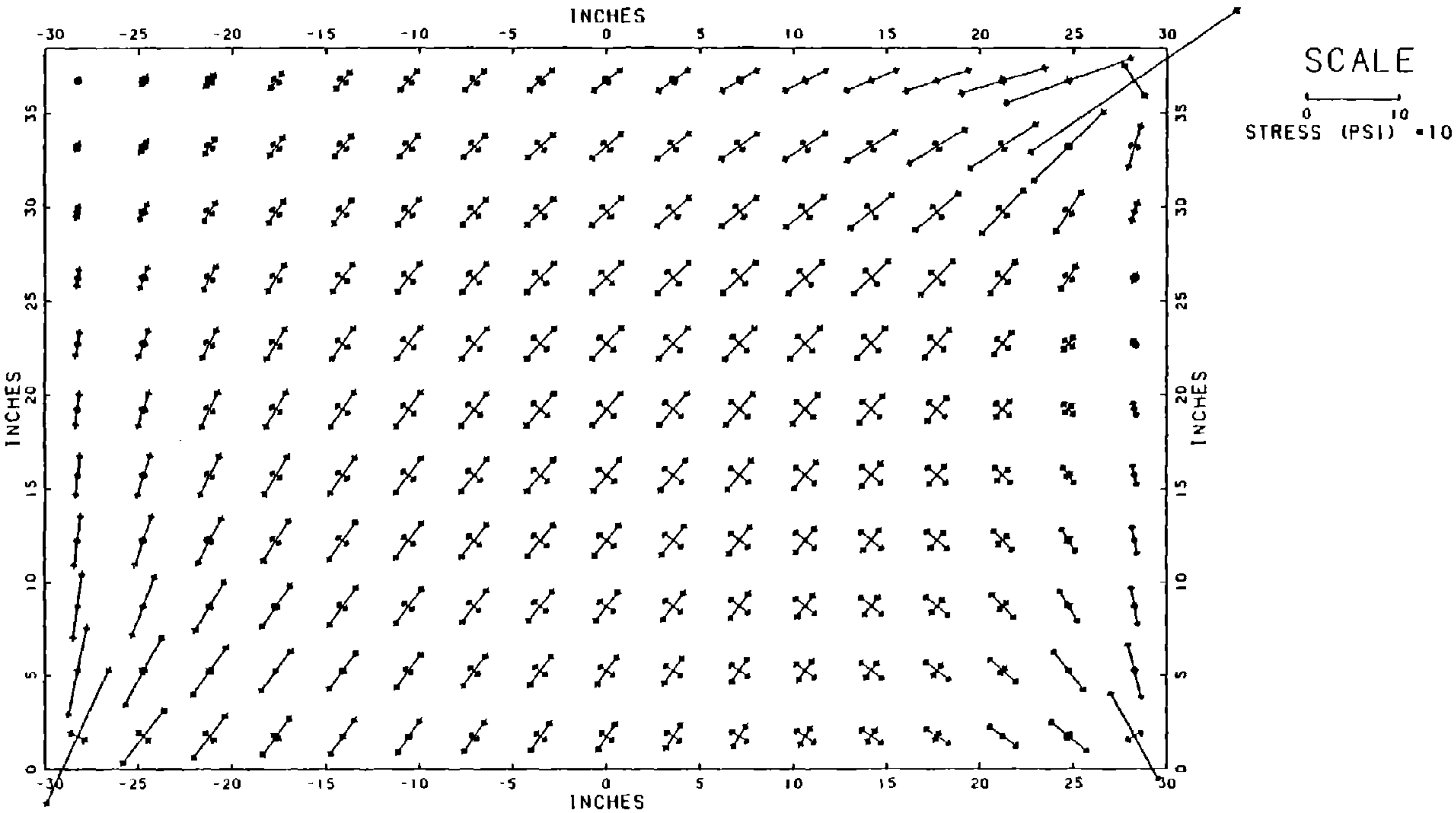


FIGURE D-9. PRINCIPAL STRESSES IN PIER 2

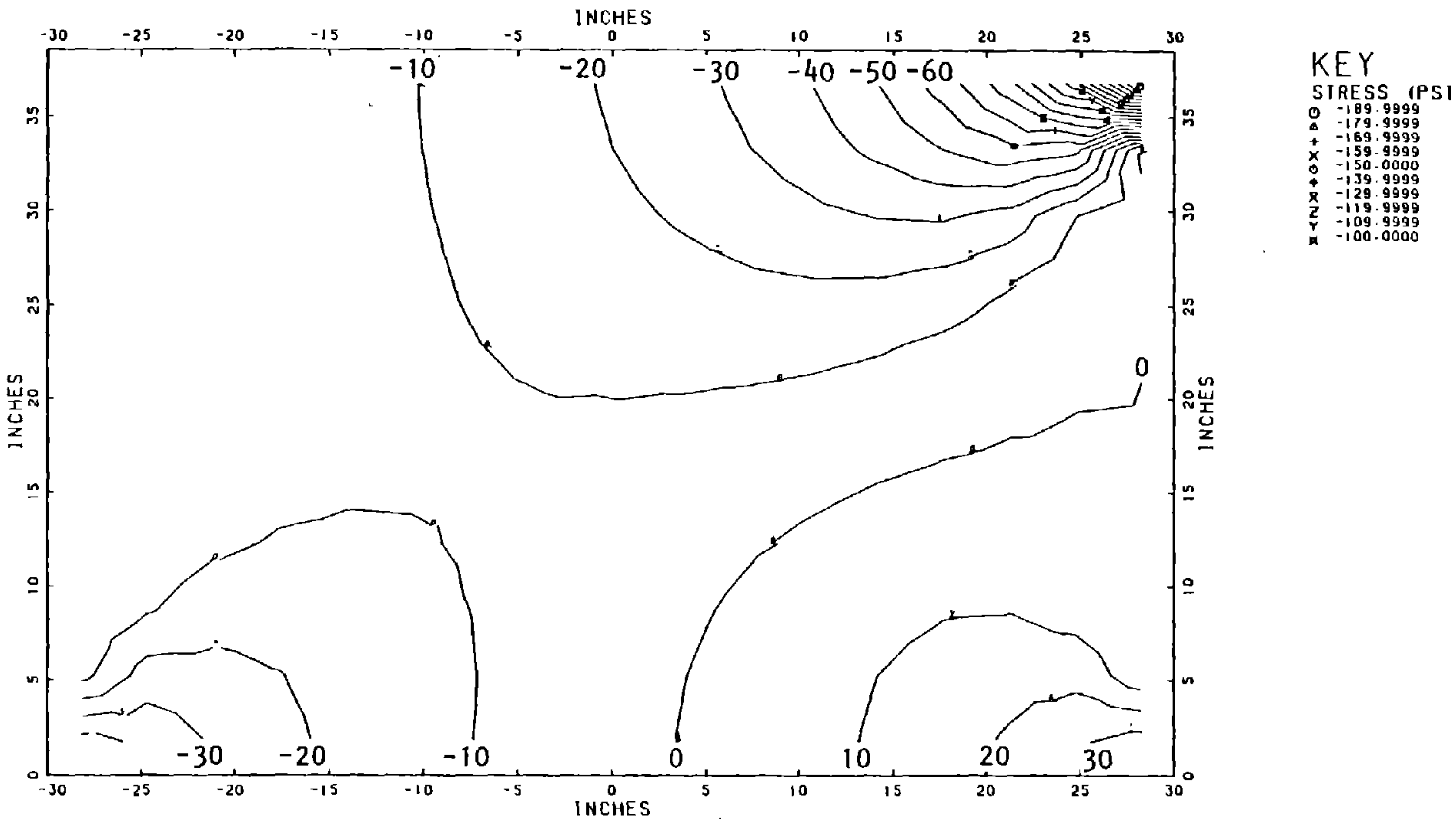


FIGURE D-10. HORIZONTAL STRESS (σ_{xx}) IN PIER 2

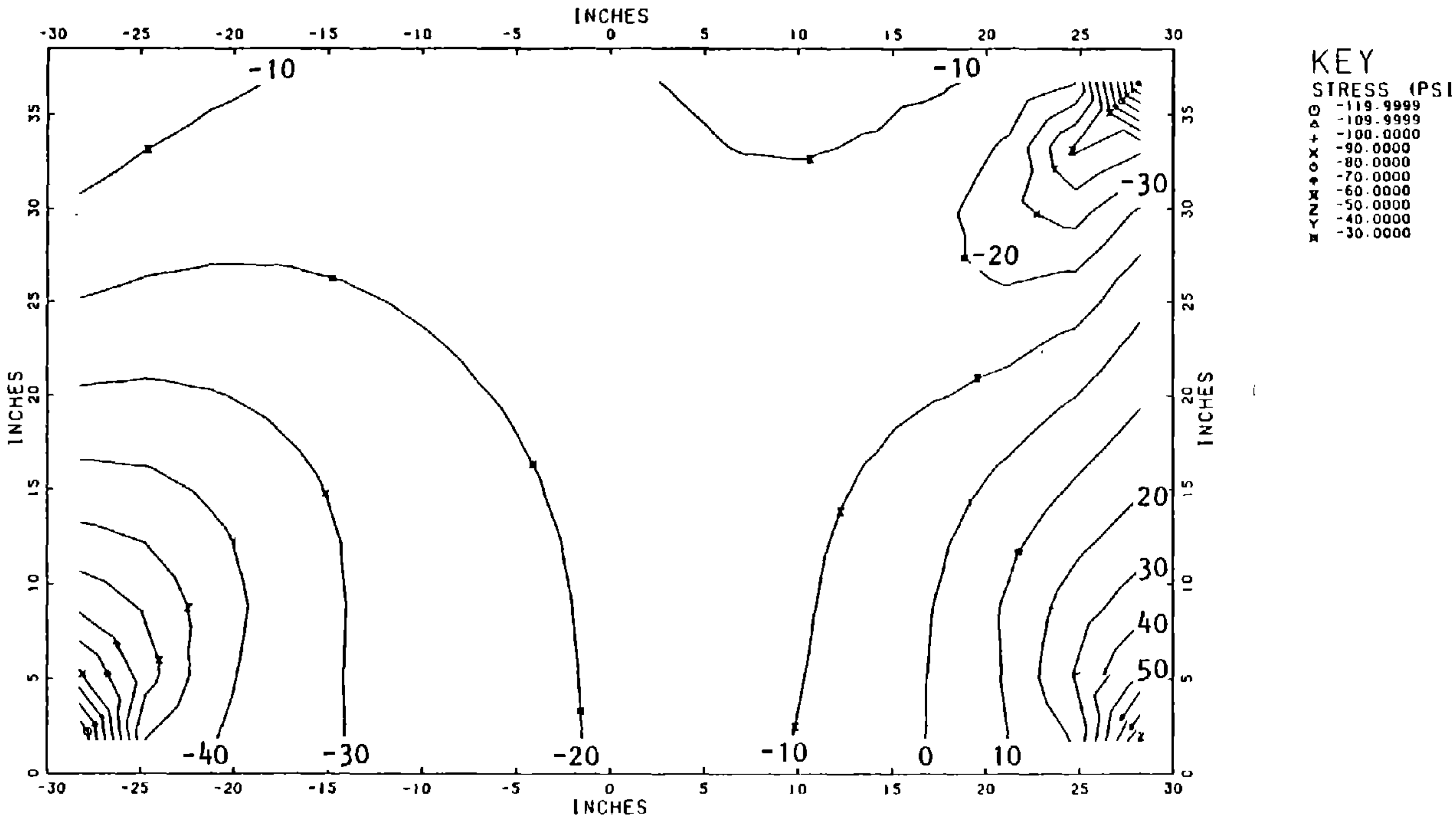


FIGURE D-11. VERTICAL STRESS (σ_{yy}) IN PIER 2

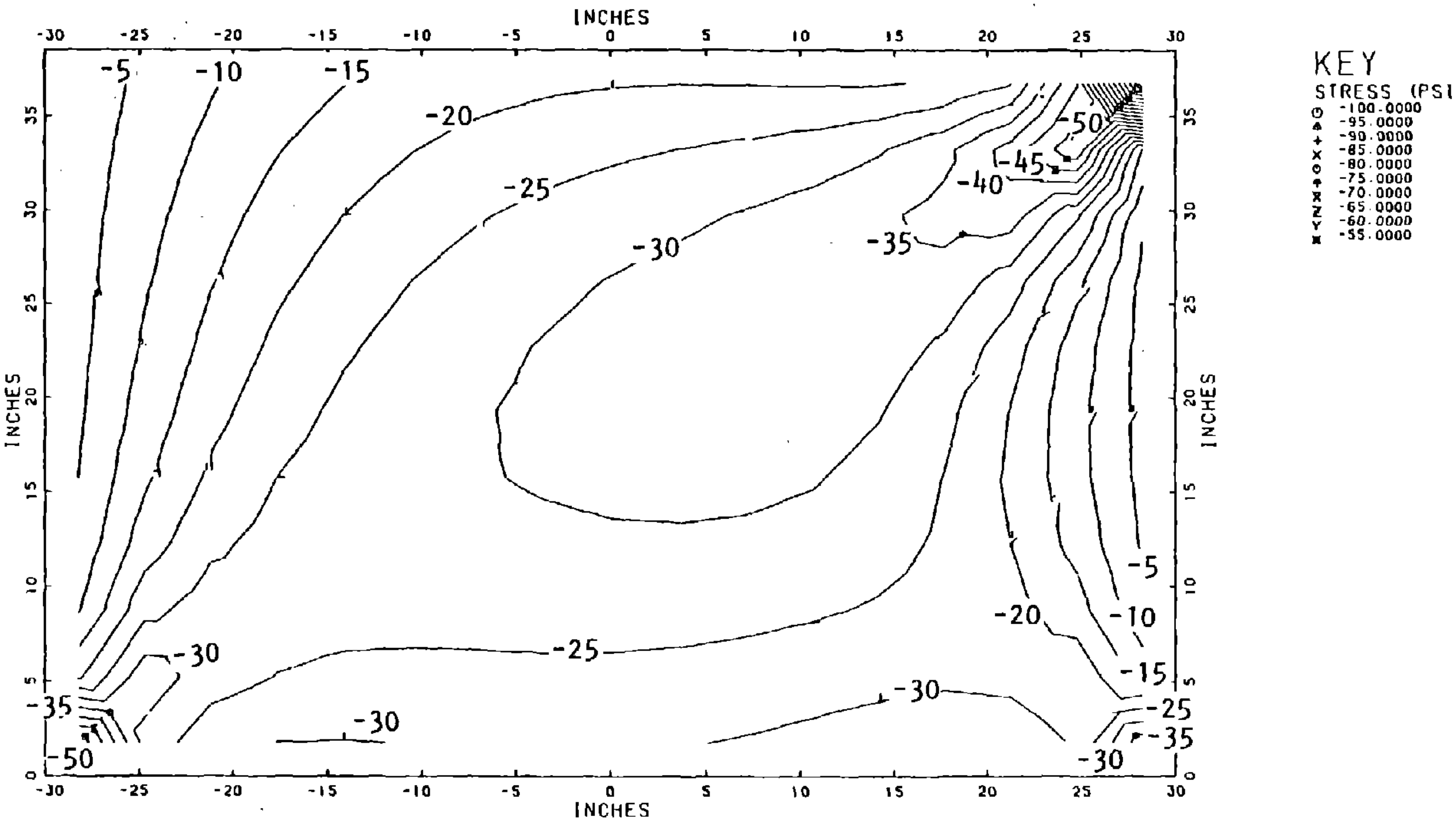


FIGURE D-12. SHEAR STRESS (τ_{xy}) IN PIER 2

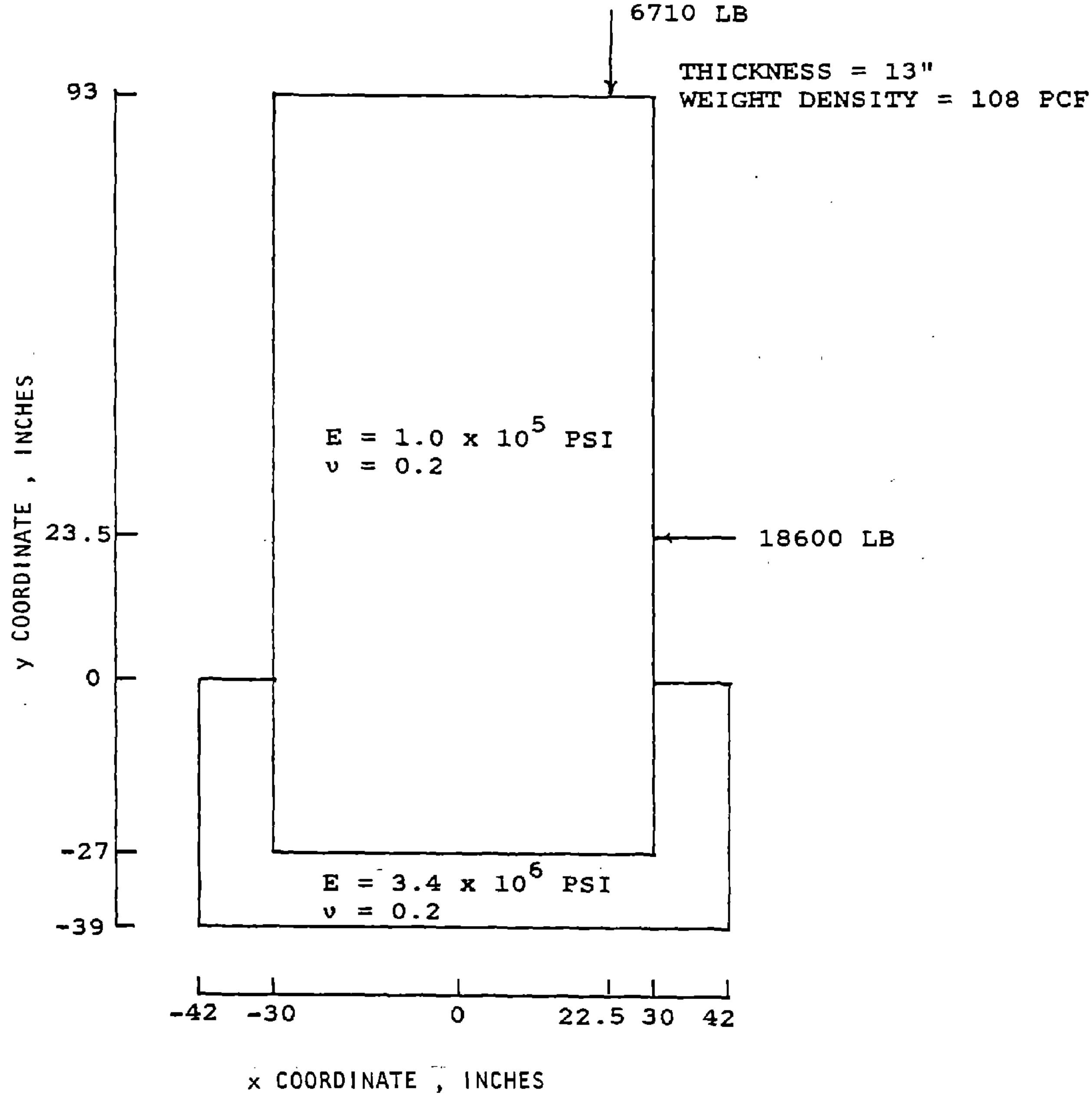


FIGURE D-13. MODEL FOR PIER 3

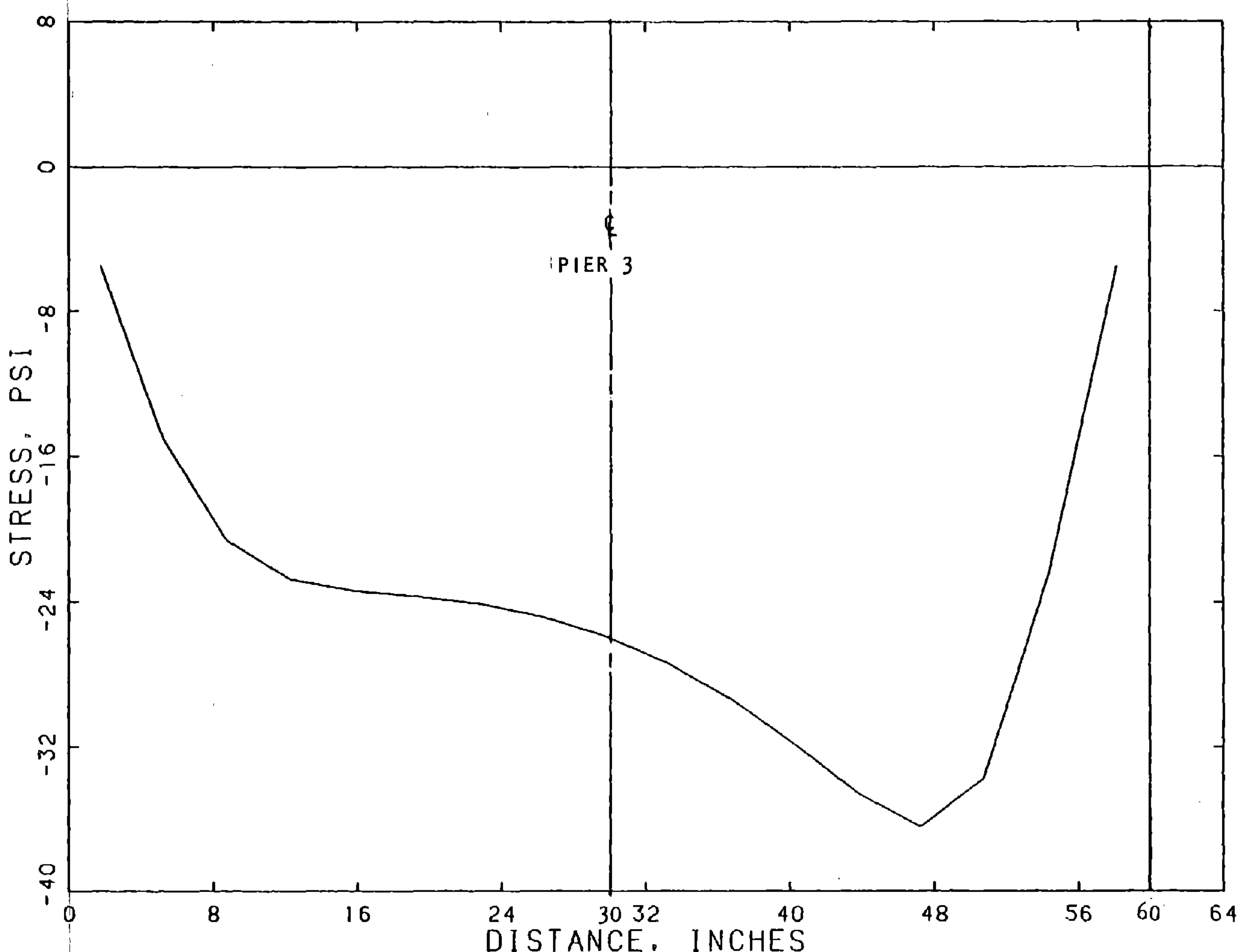


FIGURE D-14. SHEAR STRESS (τ_{xy}) IN PIER 3 AT 11.75" ABOVE BASE

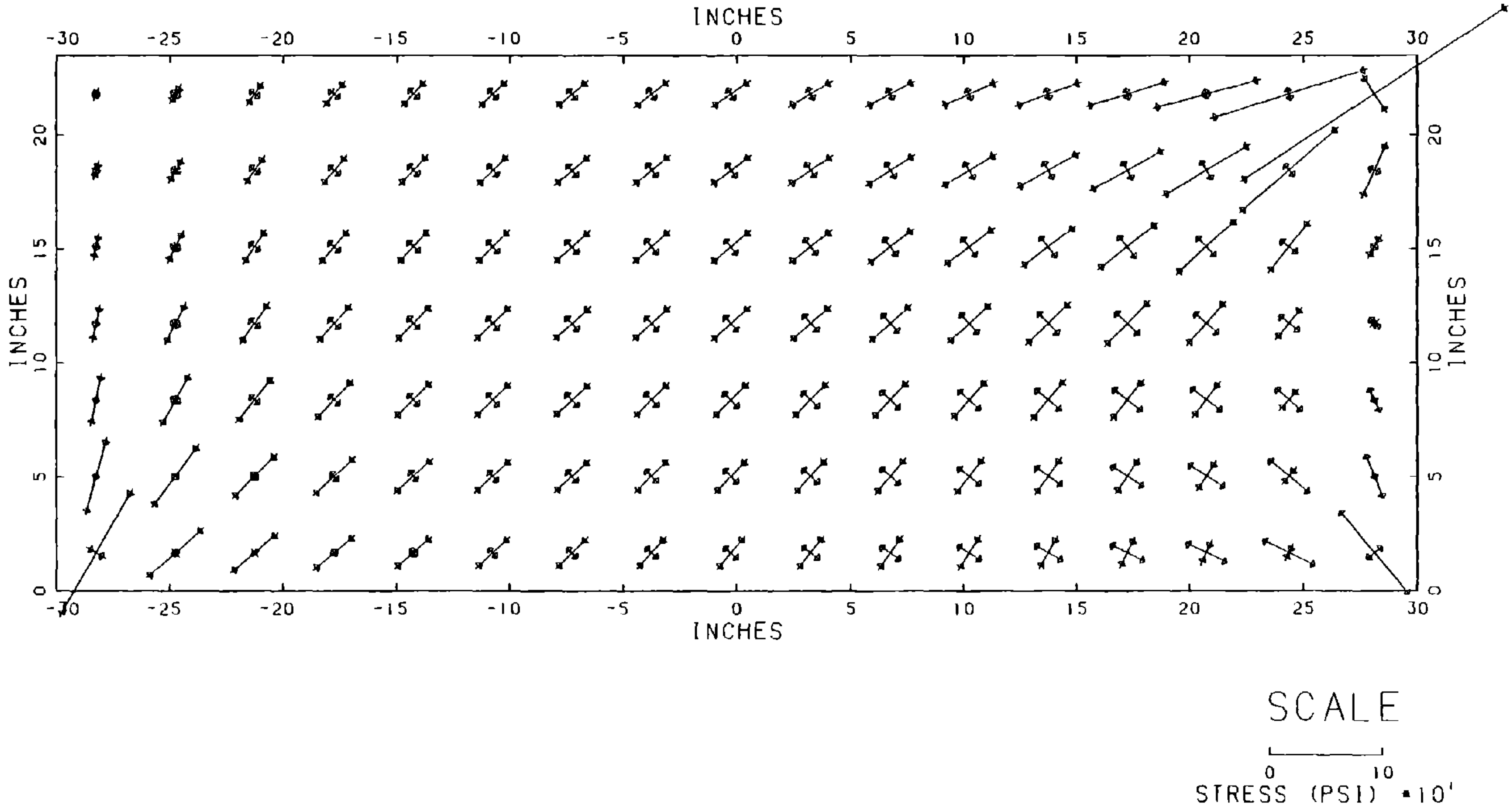


FIGURE D-15. PRINCIPAL STRESSES IN PIER 3

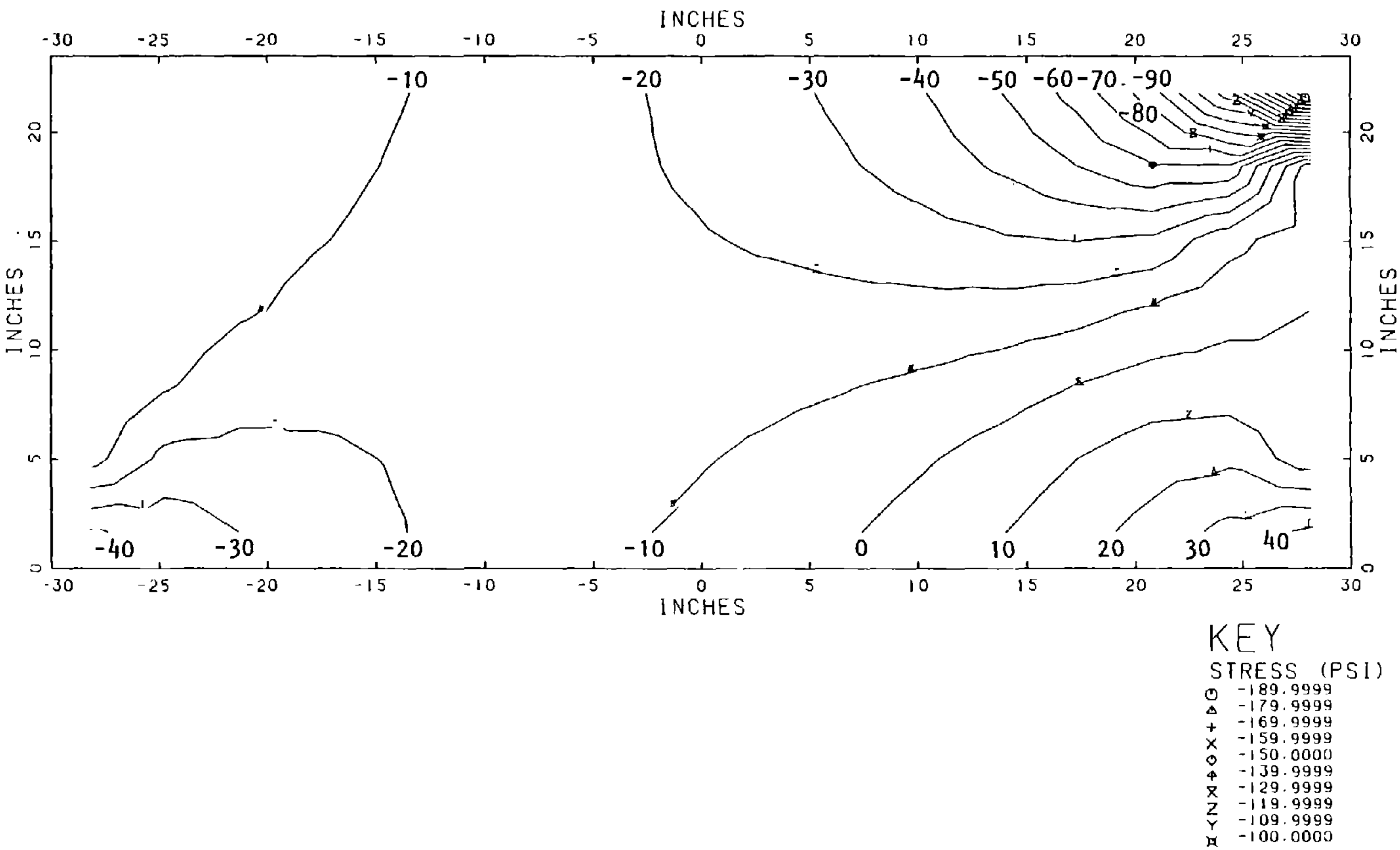


FIGURE D-16. HORIZONTAL STRESS (σ_{xx}) IN PIER 3

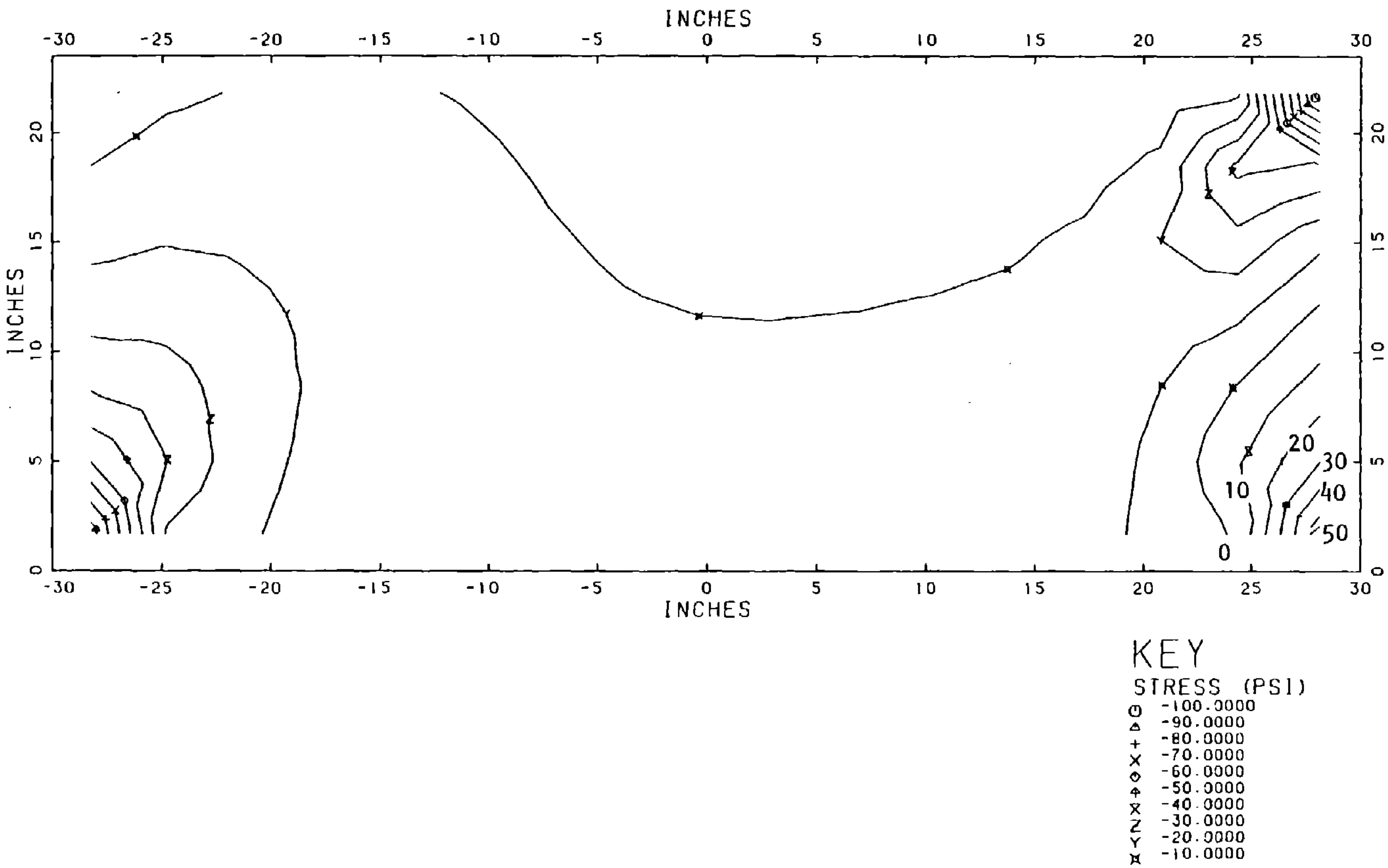


FIGURE D-17. VERTICAL STRESS (σ_{yy}) IN PIER 3

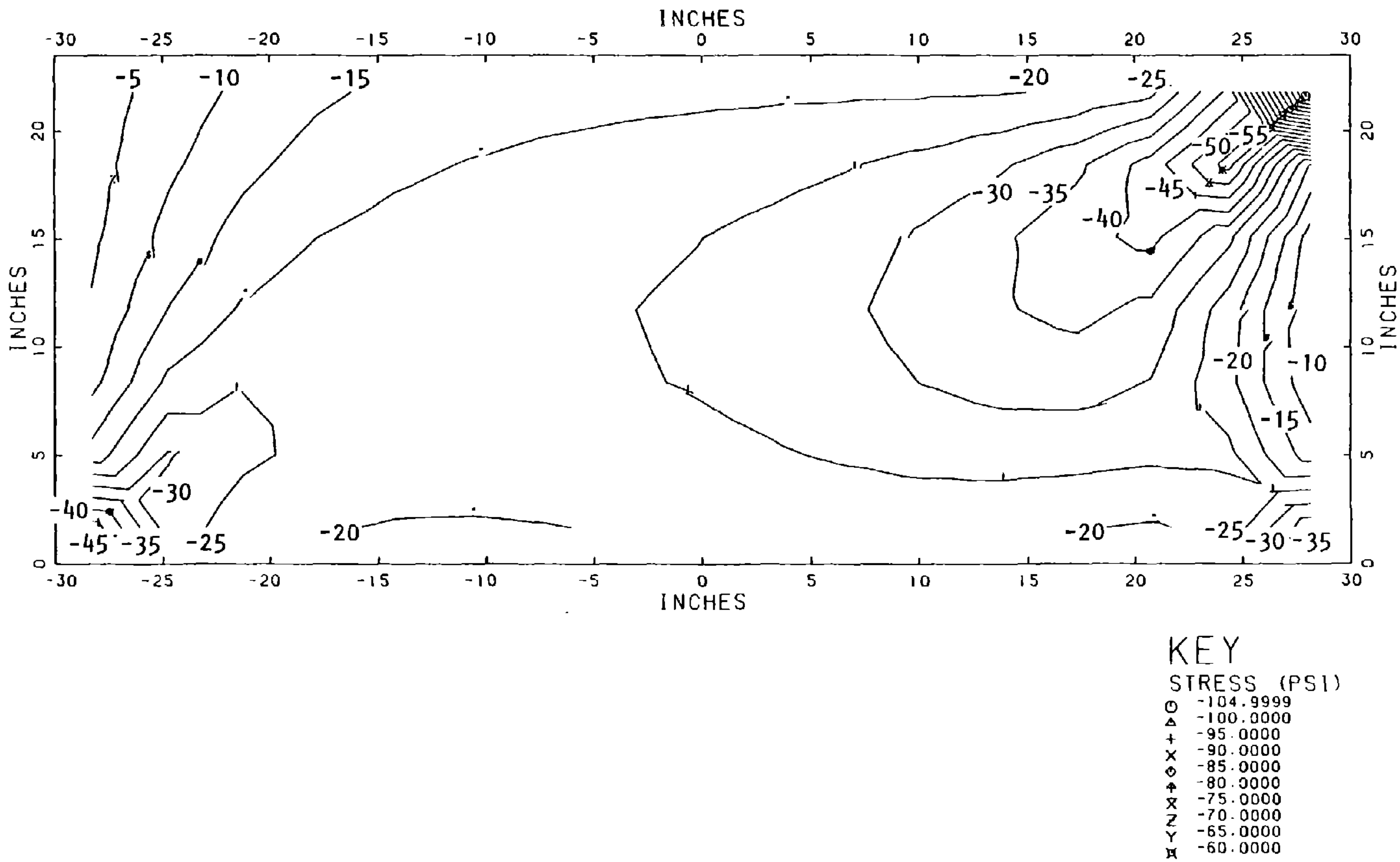


FIGURE D-18. SHEAR STRESS (τ_{xy}) IN PIER 3

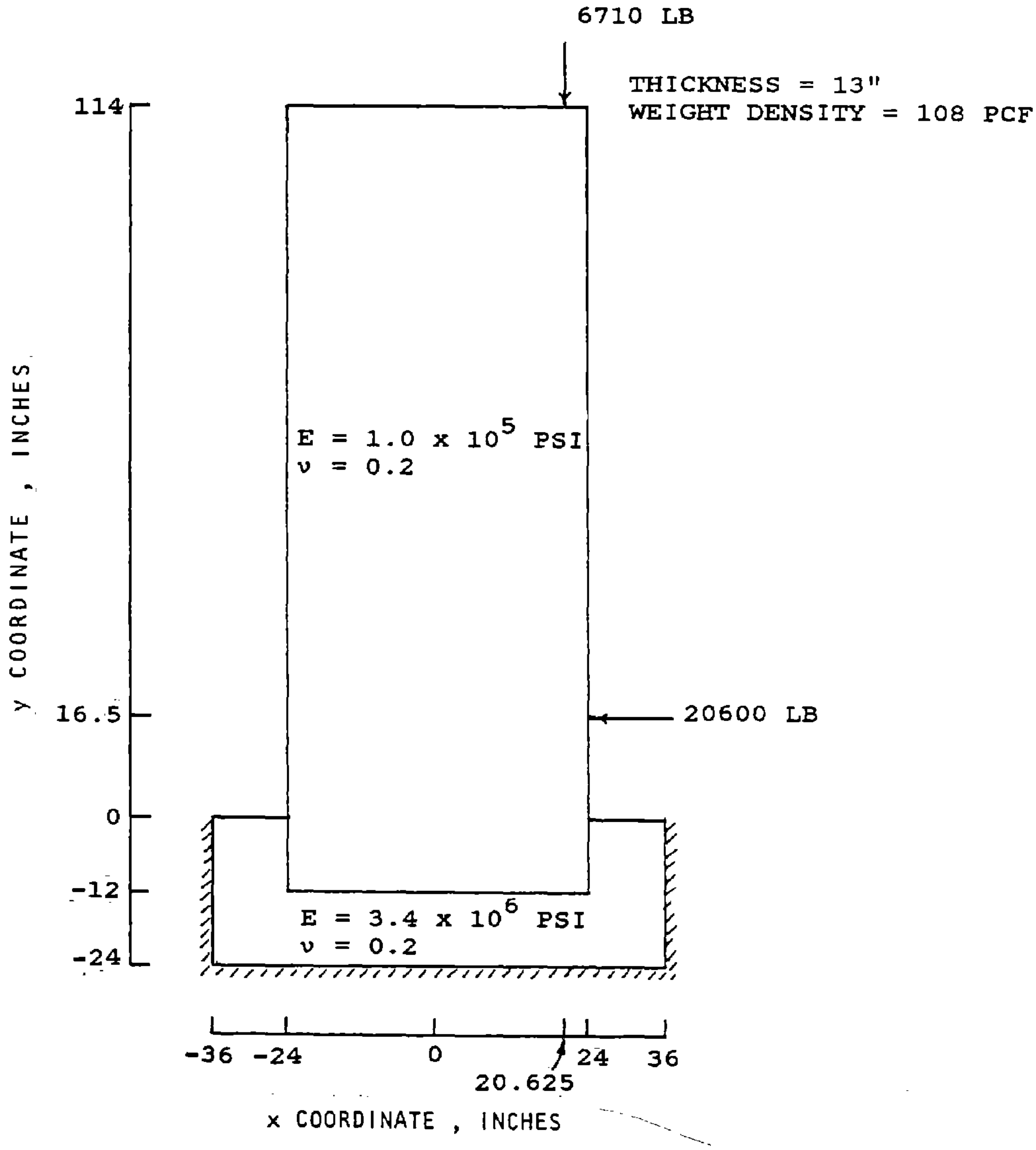


FIGURE D-19. MODEL FOR PIER 4

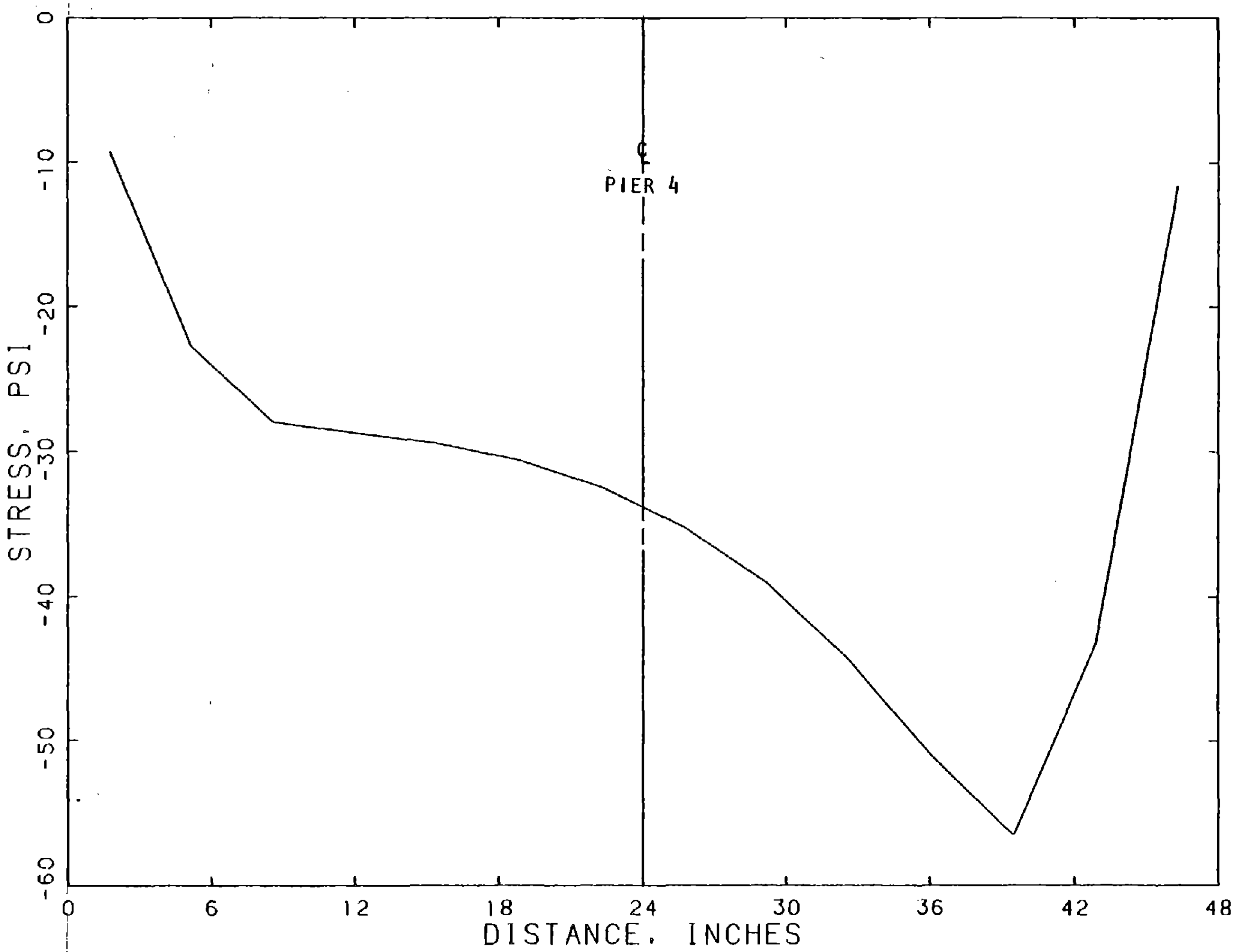


FIGURE D-20. SHEAR STRESS (τ_{xy}) IN PIER 4 AT 8.25" ABOVE BASE

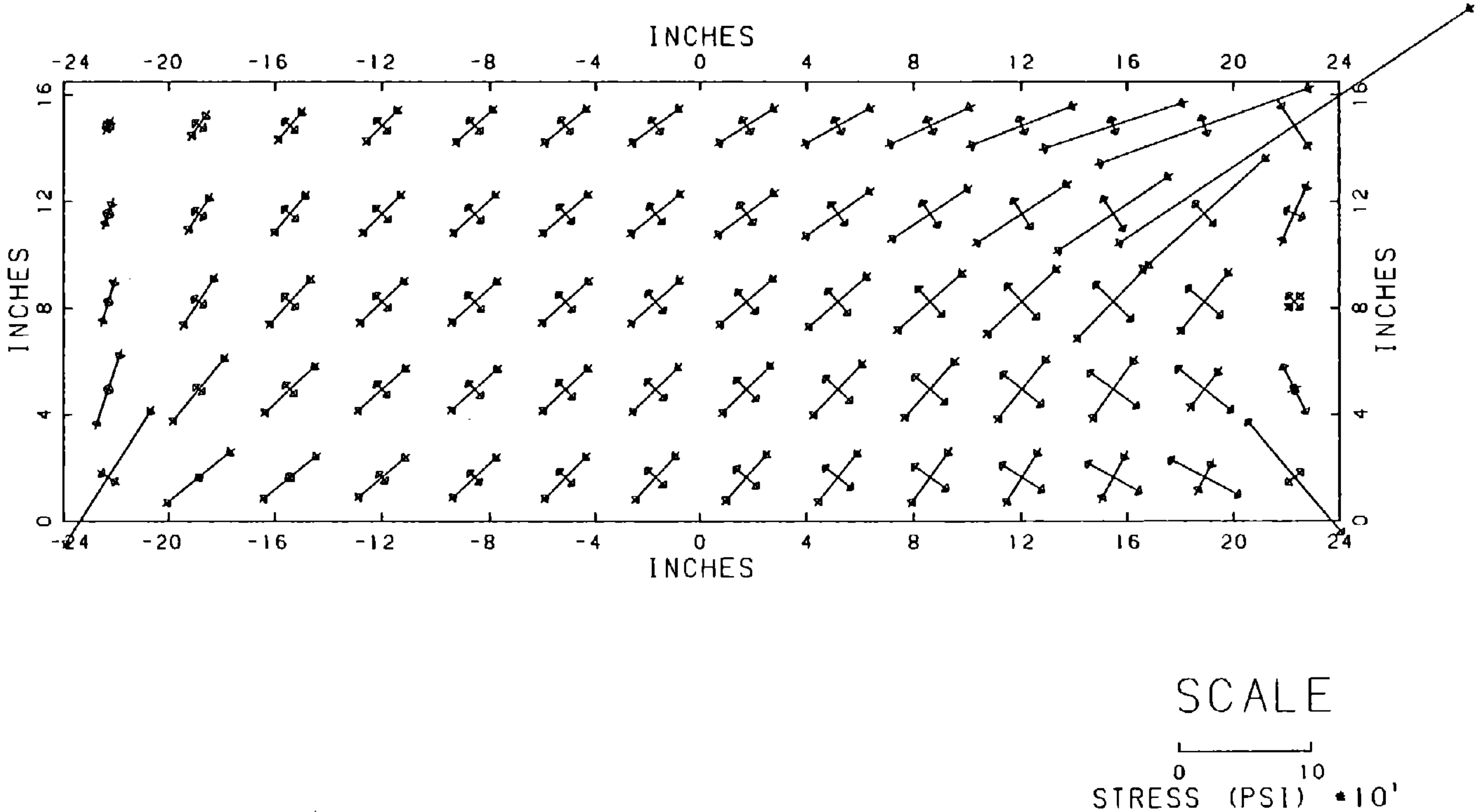


FIGURE D-21. PRINCIPAL STRESSES IN PIER 4

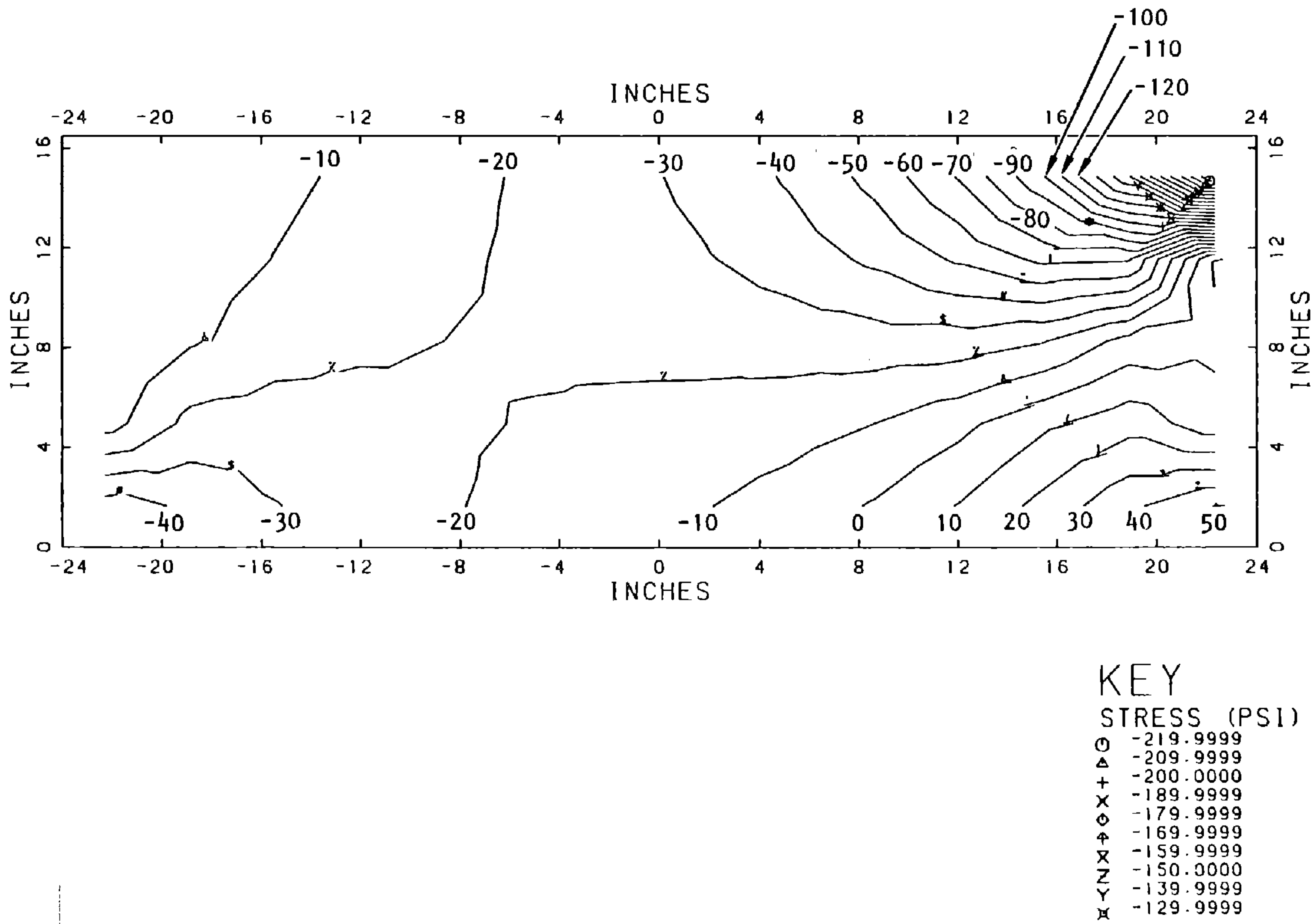


FIGURE D-22. HORIZONTAL STRESS (σ_{xx}) IN PIER 4

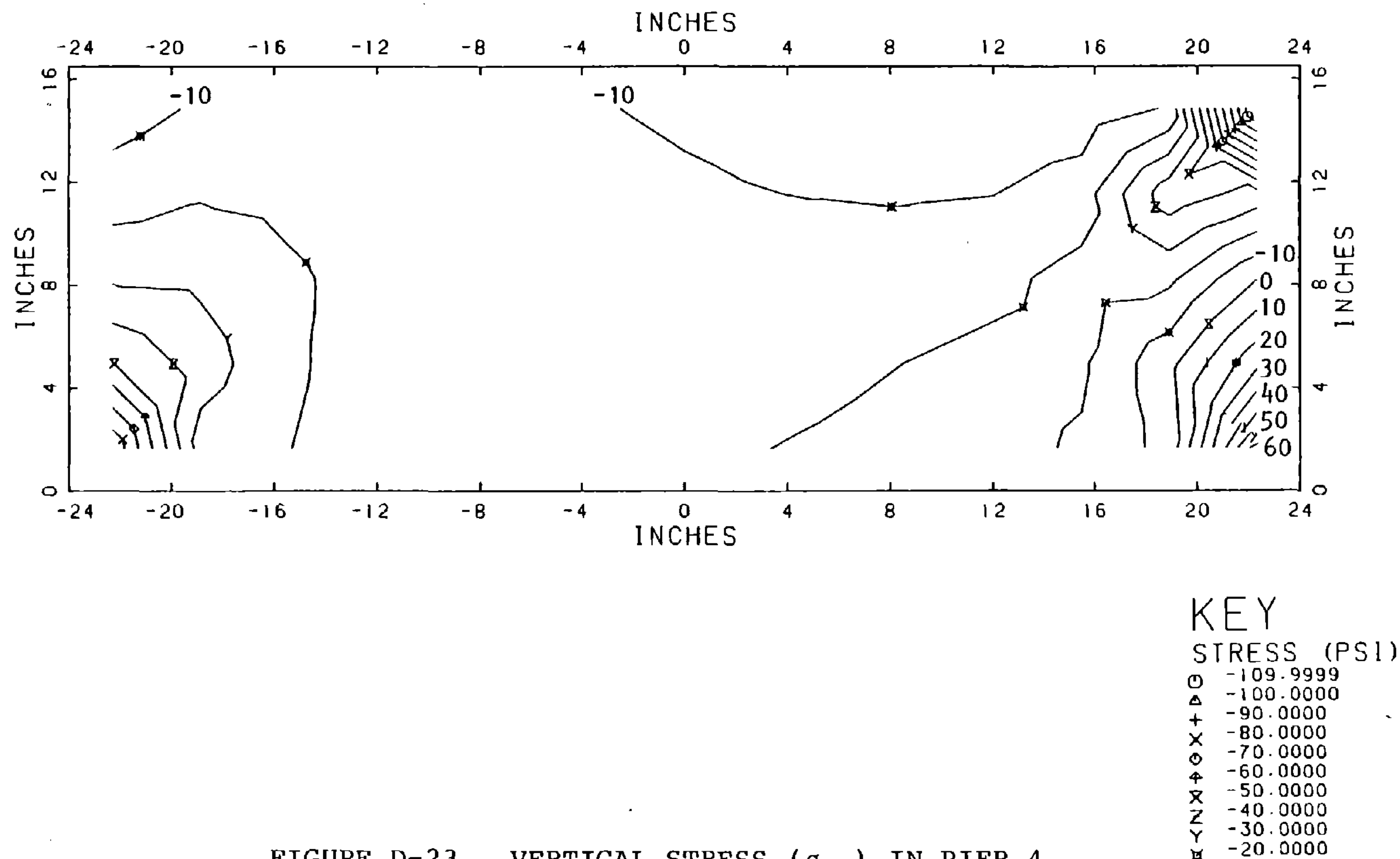
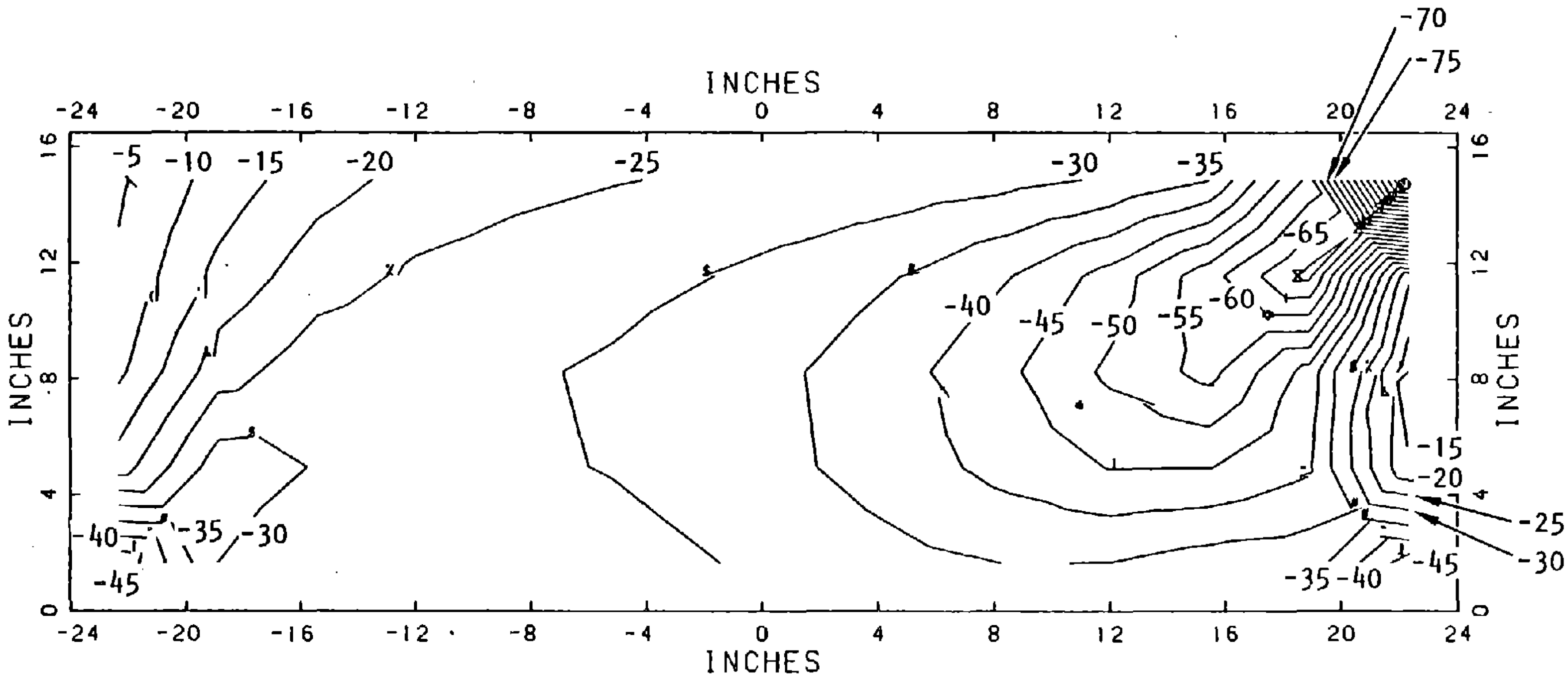


FIGURE D-23. VERTICAL STRESS (σ_{yy}) IN PIER 4



KEY
STRESS (PSI)
-125.0000
-119.9999
-114.9999
-109.9999
-104.9999
-100.0000
-95.0000
-90.0000
-85.0000
-80.0000

FIGURE D-24. SHEAR STRESS (τ_{xy}) IN PIER 4