OCONEE NUCLEAR STATION

MASONRY WALL CONFIRMATORY TEST PROGRAM

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the analytical results are non-conservative but the stability of the walls is adequate. the analytical methodology will be modified.

2 OVERVIEW OF TEST PROGRAM

The objective of the confirmatory test program is to demonstrate the validity of the arching action analysis methodology used by Duke Power Company for the structural analysis of the masonry walls at the Oconee Nuclear Station. Within this framework, all the major variables of concern to the NRC have been included in the total of twelve tests. A summary of the major issues involved in developing the test program are presented in the following subsections.

2.1 Test Set-Up

The test set-up proposed for the twelve tests is the unidirectional "shake table" shown in Figures 1 and 2. This set-up is similar to that used for a previous test program involving out-of-plane testing of reinforced masonry walls. It has the capability to apply at the top and bottom of the wall motions whose spectra closely match floor response spectra at these locations. In addition, it incorporates the appropriate differential displacement at the top and bottom of the wall.

Based on our previous experience with a similar test set-up the proposed single point load at the top of the test specimens will provide reasonably uniform input across the wall width provided that the test specimens are symmetric (openings, added masses, etc.). Accelerations will be measured across the top of each test specimen for verification of the load distribution.

As shown in Figure 1 the test set-up includes a moment resisting steel frame which provides restraint for the wall in order for the arching to take place. The steel frame will have hinges at the base of the test specimens. During the excitation of the wall the top beam will experience two different types of rotations. One rotational component is due to the differential displacement between the top and bottom of the wall where the top of wall is displaced more than the bottom. The other rotational component comes from the eccentric thrust force of the arching mechanism which introduces torque along the top beam. Figure 3 gives the estimated rotation of the top beam and shows it to be away from the compressive edge of the top mortar joint thus ensuring the conservatism in the test setup.

The alternate to this set-up is the shake table set-up shown in Figure 4. The major drawback with this set-up is the lack of control over the input motion at the top of the frame. It is our opinion that the resulting response spectra at the top of the frame will be significantly different from that desired, and as a consequence, the unidirectional set-up was selected for use in the test program.

2.2 Boundary Conditions

The test set-up shown in Figures 1 and 2 incorporates top and bottom boundary conditions typical of those at the plant. The effect of side boundaries was not included as these will significantly delay the formation of the arching action mechanism and excluding their effect will ensure that conservative results are obtained. A discussion of this is provided in Appendix A and it should be noted that it is consistent with the analysis methodology used.

The walls will be constructed on top of concrete foundations which are considered part of the test specimens. Figure 5 shows the top connection detail. The concrete beam at the top of each wall (shown in Figure 5) is placed in its location during wall construction and allowed to cure with the wall. Therefore it is considered a part of the test specimen. The steel beam shown in Figure 5 is on the other hand a part of the test frame.

2.3 Wall Dimensions

All test panels are 14'-8" high. This equals or exceeds the height of 89% of all walls qualified by arching action methodology at the plant. The lowest factor of safety obtained from the analytical assessment of the walls was for walls equal to or less than the height of the test specimens. The chosen test panel height is therefore representative of the most critical walls at Oconee Nuclear Station.

The wall width of 10 - 12 feet will be determined by the restrictions of space in the test laboratory.

All the test panels will be single wythe. The reason is that less than 3% of all the concrete block walls qualified by arching action at the Oconee Nuclear Station are multiwythe. Considering that fact and that the purpose of the test is to validate an analysis methodology this parameter is not included in the test program. Furthermore, the very limited number of multiwythe walls that were qualified by arching action were qualified as individual walls (each wythe separately) and in addition, they would also qualify on a stress basis (as multiwythe) with a very low (less than 3 psi) collar joint shear stress.

2.4 Materials

The walls will be constructed with materials (blocks and mortar) as close as possible to those at the plant. Construction techniques will also be as similar as possible to those used at the plant.

2.5 Test Panels

It is proposed to use four panels to perform the twelve tests, two of each type described in Section 3.2.

If more than one test can be performed on a test panel, then a significantly higher degree of confidence will be obtained in the arching action mechanism since it will have been demonstrated that the walls can resist more than one of the conservative input motions. Additional panels may be tested if mortar joint deterioration is observed. If these have not deteriorated during a particular test then the wall will be capable of being used for other tests.

It should be noted that during the recently completed shaking table tests on masonry houses performed at the Earthquake Engineering Research Center that seven large amplitude tests were performed on a pier that was deflecting significantly in an arching type behavior. There was no deterioration at the mid-height horizontal joint for the vertical arch; consequently, each preceding test did not have any impact on the results of each succeeding test.

2.6 Openings

Of the twelve tests proposed, six will be performed on test walls that incorporate an opening typical of those found at the plant. The height of the openings (7'-4") represents the maximum opening height found in all but one wall at the Oconee Nuclear Station.

2.7 Attachments

There are three typical types of attachments to the masonry walls at the plant. These include horizontally spanning cable trays, vertically spanning cable trays and cabinet-type attachments. The vertically spanning cable trays will significantly enhance the strength of the wall in that they will delay or prevent the formation of the arching action phenomenon. Consequently, to ensure conservatism in the test program, these are not included as a test parameter. However, both the horizontally spanning cable trays and the cabinet-type attachments are included.

2.8 Input Motions

The response spectra that will form a basis for the time history input motions will be obtained by enveloping the floor response spectra at all elevations containing masonry walls in both the North-South and East-West directions. This will be done for the respective response spectra at both the top and bottom of the walls. Thus the response spectra of the time history input motions at the top and bottom of the test specimens will exceed by a minimum of 10% the respective enveloped SSE floor response spectra. A value of 10% is considered adequate and conservative because of the enveloping procedures used and several other factors that would increase the capacity of the walls have been neglected. These include:

a. Neglecting the positive influence of the horizontal spanning of the walls.

b. Neglecting the positive influence of the many vertically spanning cable trays.

c. The test time histories will envelop all the SSE floor spectra over the full frequency range (0.15 - 100.0 Hz) and thus are very conservative when compared to an expected site time history at a given floor elevation.

d. All tests will be performed such that the test time histories will exceed the envelope of all SSE floor spectra by 10%.

Since each of the twelve tests i nd of themselves will contain a significant degree of conservatism, it is our elief that these eight tests combined with the four tests to failure will pro ide a high degree of confidence in the inherent factor of safety of the walls.

Since the arching action phenomenon is a nonlinear problem, two different types of time histories that envelop the respective floor spectra will be developed and used with each variation of openings and attachments. Selection criteria for the two different time histories will ensure they have different velocity characteristics.

The low level frequency range of the input motions will be included but certain limitations are imposed by the physical limitations of the test equipment (the actuators). Because of the large displacements inherent in the low frequency range it is necessary to filter out the very lowest of these low frequencies. It has been verified that the input motions will be able to envelop the envelope floor spectra down to the frequency range of 0.10 - 0.15 Hz. (or period range 10.0 - 7.5 Sec.).

On four test specimens the level of the input motion will be gradually increased until instability of the wall is obtained. These results will provide a basis for evaluating the overall factor of safety of the analytical methodology.

3 TEST PARAMETERS

Section 2 provided an overview of the test program and the options considered in developing the test parameters. This section provides the specific parameters of the twelve tests. A summary of the parameters is given in Tables 1 and 2 and a detailed description follows.

3.1 Materials

The test specimens will all be constructed from 8" hollow concrete masonry blocks (8" x 8" x 16" units) and be ungrouted. The specimens will be single wythe in a running bond type construction and have joint reinforcement in every other bedjoint. Samples of both prisms and blocks have been extracted from the walls at the plant. The average compressive strength of the blocks was 3135 psi and that of the prisms was 1863 psi. These will be duplicated as closely as possible in the test specimens.

3.2 Panel Dimensions

There are two different panel types. Panel Type 1 to be used for Test Nos. 1, 2, 3, 4, 5 and 6, has no opening, whereas Panel Type 2 to be used for Test Nos. 7, 8, 9, 10, 11 and 12, incorporates an opening.

The width of each panel will be 10 - 12 feet. This dimension will be determined by space constraints at the laboratory.

Panel Type 1: This panel is to be 14'-8" high (22 block courses), single wythe, ungrouted and unreinforced. The specimen will have no openings. A sketch of the panel is shown in Figure 6.

Panel Type 2: This panel type is to be identical to panel type 1 in all respects except it will have a centrally placed door opening 4'-0" wide and 7'-4" high. A sketch of the panel is shown in Figure 7.

3.3 Attachments

The effect of attachments will be assessed by comparing the results of Tests 1 and 4, 2 and 5, 7 and 10, 8 and 11 (Refer to Tables 1 and 2).

The attachment to be used for Panel Type 1 in Test Nos. 4, 5 and 6 is the cabinet type attachment shown in Figure 8. This will be centrally located in the lower half of the wall. The attachments to be used for Panel Type 2 in Test Nos. 10, 11 and 12 are the horizontally spanning cable trays shown in Figure 9. These will be located along the length of the wall in the upper half of the wall.

3.4 Input Motions

A typical floor response spectrum for the Auxiliary Building is shown in Figure 10. Envelop response spectra for the various elevations at the top and bottom of the masonry walls will be developed. This will ensure that the peak spectral values at all elevations will be incorporated in the envelop spectra.

These enveloped SSE floor spectra will form the basis for developing the test time history input motions. The response spectra of the time histories to be developed will envelop these floor spectra with as close a match as possible over the full frequency range. The only physical limitation on the time histories to be developed is that the velocity and displacement demand on the actuators must stay within the limits described in the following section.

Several sets (top and bottom) of time histories of 30 seconds duration will be developed From these two sets (i.e., two top and two bottom) of time histories will be selected for use. The selection criteria for the two sets will ensure that they have different velocity characteristics.

The intensity of the test signals will be amplified such that the target envelop floor response spectra will be exceeded by at least 10% over the full frequency range. For Tests No. 3, 6, 9 and 12 the level of the input motion will be gradually increased until instability of the wall is obtained.

4 DESCRIPTION OF TEST FACILITY

The tests will be performed at the structural laboratories of the Earthquake Engineering Research Center. University of California. Berkeley. The test setup will consist of four reaction frames (A-frames) and two MTS actuators located towards the top and bottom of the reaction frames.

The walls will be placed on an existing unidirectional "shake table" which will be slightly modified to accommodate the 10 - 12 feet width of the test specimens.

The "shake table" is a 4' x 8' x 1" steel plate set on top of four Thomson Dual Roundway Bearings allowing the table to move freely with minimal friction. A second set of bearings is placed such that uplift of the wall base is prevented. The bearings have a specified coefficient of friction equal to 0.007.

The wall assembly consists of the 14'-8" high masonry wall, two steel columns placed one on either side of the wall and a reinforced concrete beam acting as the top boundary of the wall. The steel columns will not be in contact with the wall thereby leaving the wall's side boundary unrestrained. The weight of the reinforced concrete beam will be carried by the two steel columns but the beam will be in close contact with the top of the wall thus allowing the wall to arch against it. Figures 1 and 2 show the proposed test setup.

The actuators are capable of developing a maximum dynamic load of 75 Kips using a hydraulic pressure of 3000 psi for a relatively short time. Normal use of the hydraulic system requires a hydraulic pressure of 2500 psi thus reducing the dynamic load to 62.5 Kips. The maximum stroke of the actuators is ± 6 inches, the maximum piston velocity is 30 in/sec and the flow capacity of the servovalves is 200 gal/min. The actuators are controlled by a displacement command signal and follow a prescribed displacement pattern which can be any earthquake time history, sine wave, step function, etc. This is the most common test method and will be used for this test.

The operational capabilities of the actuators are limited by the above-mentioned force, velocity and displacement capacities and also by a frequency limitation of about 5 Hz. for sinusoidal loads.

No major modifications are required to the existing reaction frames. However, for the purpose of having the four test frames perform together, two stiff cross beams will be constructed between the reaction frames just below or above the actuators.

5 DATA TO BE MEASURED

There will be three groups of tests performed, namely material property tests, low level dynamic response tests and high level dynamic response tests. The following addresses the data to be measured in each group of tests.

5.1 Material Property Tests

The objective of the material property tests is to determine the strength characteristics of units of the material from which the wall is constructed.

5.1.1 Mortar

For each panel a minimum of nine cube samples of mortar will be constructed concurrent with the construction of the panel. Three cubes of mortar will be tested at (1) seven days, (2) 28 days following panel construction and (3) within 48 hours of the dynamic test of the panel. These cube tests will conform to ASTM C91 and ASTM C270-80a.

5.1.2 Block and Prism Specimens

A minimum of three samples of block and three prisms will be tested for each panel. The three-high unit prism samples will be constructed at the same time and with the same materials as the panels they represent. The block and prism tests will take place within 48 hours of the dynamic test of the panels for which the blocks and prisms are representative. These tests will conform to ASTM E 447-74 and ASTM C 140-75.

During the masonry prism tests some of the prisms will be loaded eccentrically with varying eccentricity until the cracking of bedjoints occurs. After cracking of the bedjoint(s) the prisms will be loaded and unloaded without the eccentricity, gradually increasing the load until a failure occurs.

During the testing of the block and prism specimens a number of DCDT's will be attached to the specimens and data from these will be acquired throughout each test. This data will allow determination of the nonlinear compression stress - strain relationship of the individual masonry block units, the mortar joints and the combined prisms. The data will also allow determination of strain distribution in the specimens which is especially important in the eccentrically loaded prisms. The compression stress - strain characteristics thus determined (specifically the crushing stress and strain of the masonry material) will then be used in the correlation studies between the test results and the analytical arching action methodology.

5.1.3 Flexural Specimens

A minimum of three flexural specimens 5 units high will be tested for each panel to determine the modulus of rupture of the specimens. The flexural samples will be constructed at the same time and with the same materials as the panels they represent. The tests will take place within 48 hours of the dynamic test of the panels for which the specimens are representative.

During the testing of the flexural prisms a number of DCDT's will be attached to each specimen and data acquired continously throughout each test to failure. This data will allow determination of the tensile stress - strain relationship of the masonry material (block, mortar and combined). This is useful when the uncracked capacity of the test panels is evaluated.

5.1.4 Web-shear Tests.

A minimum of three masonry block units will be tested for each test specimen. The tests will be performed similarly to the ones performed by Gabrielson, et. al. (Ref. 3).

5.2 Low Level Dynamic Response Tests

The purpose of these tests is to evaluate the low amplitude damping exhibited by the test specimens.

Pull-back testing will be used to evaluate the damping corresponding to the uncracked and the cracked states of the test panels. This will involve statically displacing the panels, releasing them suddenly and allowing them to vibrate freely while measuring the panel deflections and accelerations at the center of the panel. Damping will then be determined by the logarithmic decrement method following standard procedures using the accelerometers and potentiometers at the center of the wall.

Previous experience has shown that response quantities not close to the location of maximum response (in this case midheight of panel) produce unreliable and inconsistent damping values.

5.3 High Level Dynamic Response Tests

Response measurements to be recorded during the dynamic tests will obtain as much data as is practicable to validate the arching action analysis methodology. Data of interest is the relative deflection of the wall at various locations, compressive stresses in the masonry face shells at the hinged locations and the force developed at the top of the wall. In addition, the input at the top and bottom of the wall will be measured and compared to that required. A discussion of each of the parameters to be measured follows.

5.3.1 Deflection of the Test Panels

An important parameter of interest is the relative deflection profile of the wall. Potentiometers will be used to measure the out-of-plane deflections of the panels during the testing. The potentiometers will be placed over the height of the wall as shown Figures 11 and 12.

5.3.2 Faceshell Strain and Curvature

The compressive stress in the masonry face shells is an important variable for the validation of the analysis methodology. The compressive stress in the masonry face shells can only be measured indirectly from compressive strain measurements.

The curvature and faceshell strain will be measured by the change in distance between a series of small studs attached to the block face of the panels symmetrically on each side. These studs will be oriented in a vertical line such that a Direct Current Displacement Transducer (DCDT) may be oriented vertically when attached to two studs. Each DCDT will then measure vertical displacements between two stud locations. A maximum of 36 DCDT's will be attached to the walls, 18 on each side. Typical locations are shown in Figures 13 and 14. The curvature and faceshell strain will then be calculated from these relative displacements. The compressive stress in the face shell will then have to be deduced from the compressive strain measurements. This will be performed by using the stress-strain curves obtained from the prism tests.

Direct measurement of the longitudinal strain in the face shell of a masonry unit in a dynamic test of this nature is extremely difficult. The proposed method is to use DCDT's. An alternate is to use a strain gage of sufficient length (4-6 inches) to measure the compression or tension strain over a reasonable length of the unit. However, it is very difficult to ensure that an adequate bond exists over the full length of the strain gage when bonding to a surface with the roughness of a masonry unit. If bond is lost along any part of the length of the strain gage due to local microcracking or other causes the strain gage tends to buckle and thus record tension rather than compression measurements. These problems have been observed in many previous test programs and as a result strain gage measurements have not been utilized for masonry units in any recent masonry research programs and are not included in this test program.

5.3.3 Acceleration Response of the Test Panels

Accelerometers will be used to measure the acceleration response of the panels. The location of these accelerometers is shown in Figures 15 and 16. The accelerometers at the top and bottom of the walls will be used to measure and validate the input motion to the wall. The other accelerometers will provide additional data on the frequency response of the wall.

5.3.4 Force Developed at Top of the Walls

Load cells will be inserted in the two steel columns to measure the force developed at the top hinge. The load generated in these load cells will be the reaction force of the wall against its top boundary.

6 TEST REPORTS

There will be three reports on the various phases of the test program. The contents of the reports are described below.

6.1 Pre-Test Analysis

Prior to the commencement of the testing each test specimen will be analyzed using Duke Power Company's arching action analysis methodology to provide a best estimate of the expected test results.

Material properties will be based on the actual field tests previously performed since these will be the target values for the test specimens. The time history to be used for the analysis will also be the same as the test time history.

6.2 Report on Reduction and Evaluation of Test Data

This report will present the final results and the manner in which the raw data was reduced. Included will be the following.

a. Detailed description of the test setup and its characteristics.

b. Development of the desired test inputs and a comparison of the desired and achieved input motions.

c. Strength characteristics of the mortar for each panel.

d. Strength and stress-strain characteristics of the block and prism specimens for each panel.

e. Strength and stress-strain characteristics (modulus of rupture) of the flexural specimens for each panel.

f. Full intensity test data as directly measured. During the full scale dynamic testing of the test panels there will be 64 channels of measured data. For each channel the maximum positive and negative values will be determined together with the time of occurrence. The data discussed in Sec. 6.2.1 will be obtained from the four different measuring devices:

6.2.1 Detailed Test Data

a. Deflections

Deflection profiles of the wall will be shown for the time periods during the testing when the maximum positive and negative displacements at the center of the wall are reached.

Selected time histories of each displacement channel will be plotted.

Relative time histories up the height of the wall with respect to the top and base of the wall will be plotted.

b. **Faceshell Strain and Curvature**

The faceshell strain at the DCDT locations will be calculated. In addition the curvature will be calculated.

Time history curvature and faceshell strain plots at selected locations will be presented.

c. **Accelerations**

Response spectra will be plotted for each of the accelerometers. Time history plots of each accelerometer will be presented.

6.3 Report on Validity of Arching Action Analysis Methodology

The main objective of the confirmatory test program is to validate the arching action analysis methodology used to assess the stability of the masonry walls at the Oconee Nuclear Station. In order to achieve this objective several key parameters from both test and analytical results will be compared.

The arching action theory for masonry walls was initially developed by McDowell, et. al. (Reference 1) and presented in his paper as a family of curves. The curves are normalized to dimensionless variables of thrust ratios as a function of midspan deflection on one hand and moment ratio as a function of midspan deflection on the other. These curves are reproduced in Figure 17 of this proposal, and Figure 18 provides a definition of the variables. In addition, e_c defines the strain at which maximum stress S_c is attained in the masonry stress - strain relationship.

The main parameter that defines the particular force - deflection curve for a wall is the variable R. This variable therefore defines the path that the thrust or moment ratios vs midspan deflection follows as the wall deflects from zero to snap-through. The variable R depends on the wall geometry and the crushing strain e_c of the masonry material. Thus once the crushing strain has been determined from the prism tests, R and then the force-deflection curve can be determined

During the test program all the necessary data to correlate tests with analysis will be measured. These include the stress - strain relationship of the masonry material, the thrust force and the midspan (or crack) deflection. Using that data the following correlation procedure between analysis and tests is anticipated.

a. During the testing of the compressive prisms the stress-strain relationship for the test panels will be established (Section 5.1). From that relationship e_c and S_c will be determined.

b. Using the e_C value with the known test panel geometry the R value for the panels will be calculated. Similarly using the S_C value the analytical thrust force and midspan deflection curve will be calculated.

c. During the testing of the walls the thrust force and midspan (or crack) deflection will be directly measured. Using these values the actual thrust force vs. midspan deflection relationship will be established.

d. The analytical curve and the actual curve will be compared and correlated.

In addition to the above steps one of the most important comparisons between the analysis and test results will be the maximum deflection at the center of the wall and the ability of the wall to withstand the compressive strains induced by the maximum deflection. If the compressive masonry strains are excessive, face shell spalling along the entire length of a bed joint will be visually apparent from the tests.

The means and criteria to explain significant differences between test results and analysis will depend upon what, if any differences occur between the key results from the analysis and those obtained from the testing. Two scenarios are possible:

1. The key test results are more conservative than those from the analysis. In this case the analysis methodology will be considered conservative and therefore acceptable. The definition of conservative is that the test force-deflection results lie at or above the theoretical curve.

2. The key test results are less conservative than those from the analysis. In this case the analysis methodology will be assessed and modified or a new analysis criteria developed in light of the behavior of the test specimens. This may require reanalysis of some of the walls.

7 TEST SCHEDULE

It is anticipated that the testing will take about 22 weeks including the construction and curing of the masonry walls, set-up and instrumentation and the actual testing of the wall. Compilation, reduction and preparation of the test reports is estimated to take about 17 weeks and be performed partially concurrently with the testing of the walls. The overall test schedule · presented in Figure 19.

This represents a relative schedule pending agreement with the NRC on definition of the test program and availability of the test facility.

8 REFERENCES

1. McDowell, E., McKee, K. and Sevin, E., "Arching Action Theory of Masonry Walls," Journal of the Structural Division, Proceedings of ASCE, ST2, Paper 915, March 1956.

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4. Gulkan, P., Mayes, R. and Clough, R., "Shaking Table Study of Single Story Houses, Volume 2 : Test Structures 3 and 4," Earthquake Engineering Research Center, Report No. UCB/EERC-79/24, College of Engineering, University of California, Berkeley, CA, September 1979.

5. Gulkan, P., Mayes, R. and Clough, R., "Shaking Table Study of Single Story Masonry Houses, Volume 1 : Test Structures 1 and 2," EERC Report No. UCB/EERC-79/23, College of Engineering, University of California, Berkeley, CA, September 1979.

6. Gulkan, P. and Mayes, R., and Clough, R., "Strength of Timber Roof Connections Subjected to Cyclic Loads," EERC Report No. UCB/EERC 78/17, College of Engineering, University of California, Berkeley, CA, September 1978.

TABLE 1: TEST PARAMETERS - PANEL TYPE 1

PARAMETER \ TEST NO.	1	2	3	4	5	6
Wall Height	14'-8"	14'-8"	14'-8"	14'-8"	14'-8"	14'-8"
Panel Geometry	Type 1	Type 1	Type 1	Type 1	Type 1	Type 1
Opening	No	No	No	No	No	No
Type of Attachment	None	None	None	Cabinet in Lower Half	Cabinet in Lower Half	Cabinet in Lower Half
Excitation Time History	Input 1	Input 2	Increasing Magnitude of Input 1	Input 1	Input 2	Increasing Magnitude of Input 1

TABLE 2: TEST PARAMETERS - PANEL TYPE 2

PARAMETER \ TEST NO.	7	8	9	10	11	12
Wall Height	14'-8"	14'-8"	14'-8"	14'-8"	14'-8"	14'-8"
Panel Geometry	Type 2	Type 2	Type 2	Type 2	Type 2	Type 2
Opening	Yes	Yes	Yes	Yes	Yes	Yes
Type of Attachment	None	None	None	Horiz. Cable Tray in Top Half	Horiz. Cable Tray in Top Half	Horiz. Cable Tray in Top Half
Excitation Time History	Input 1	Input 2	Increasing Magnitude of Input 1	Input 1	Input 2	Increasing Magnitude of Input 1

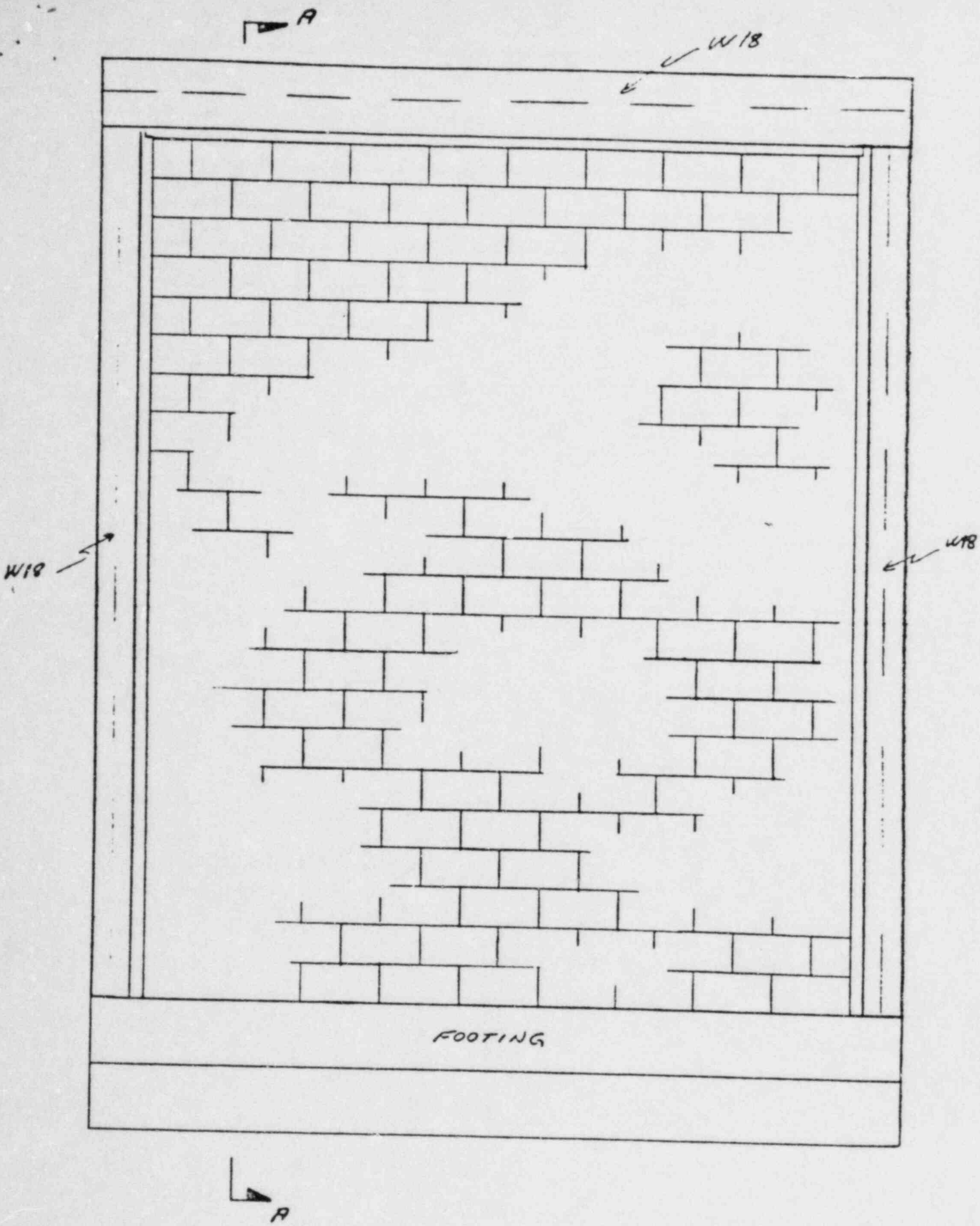


Figure 1: Test Set-Up - Frontal View.

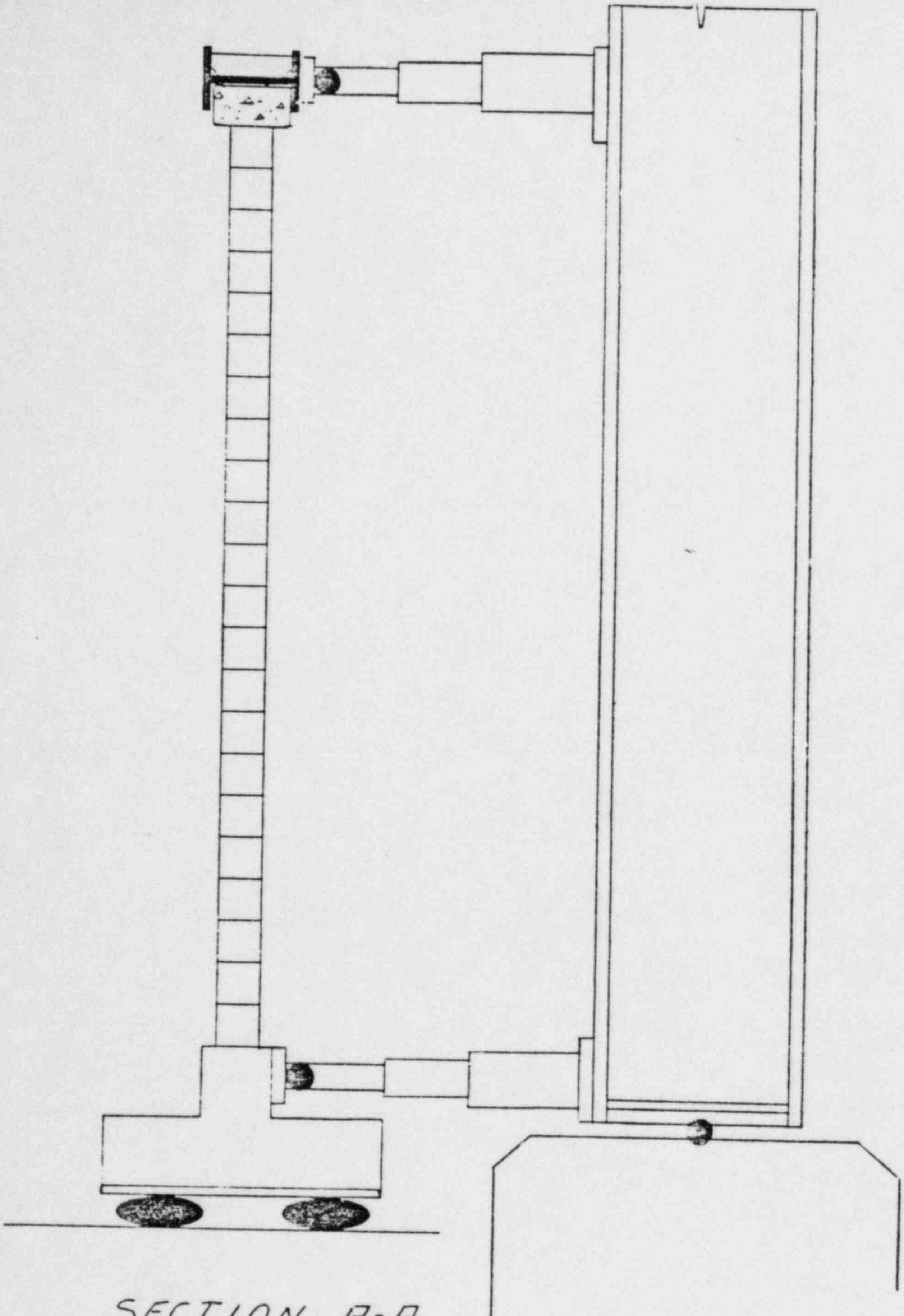


Figure 2: Test Set-Up - Side View.

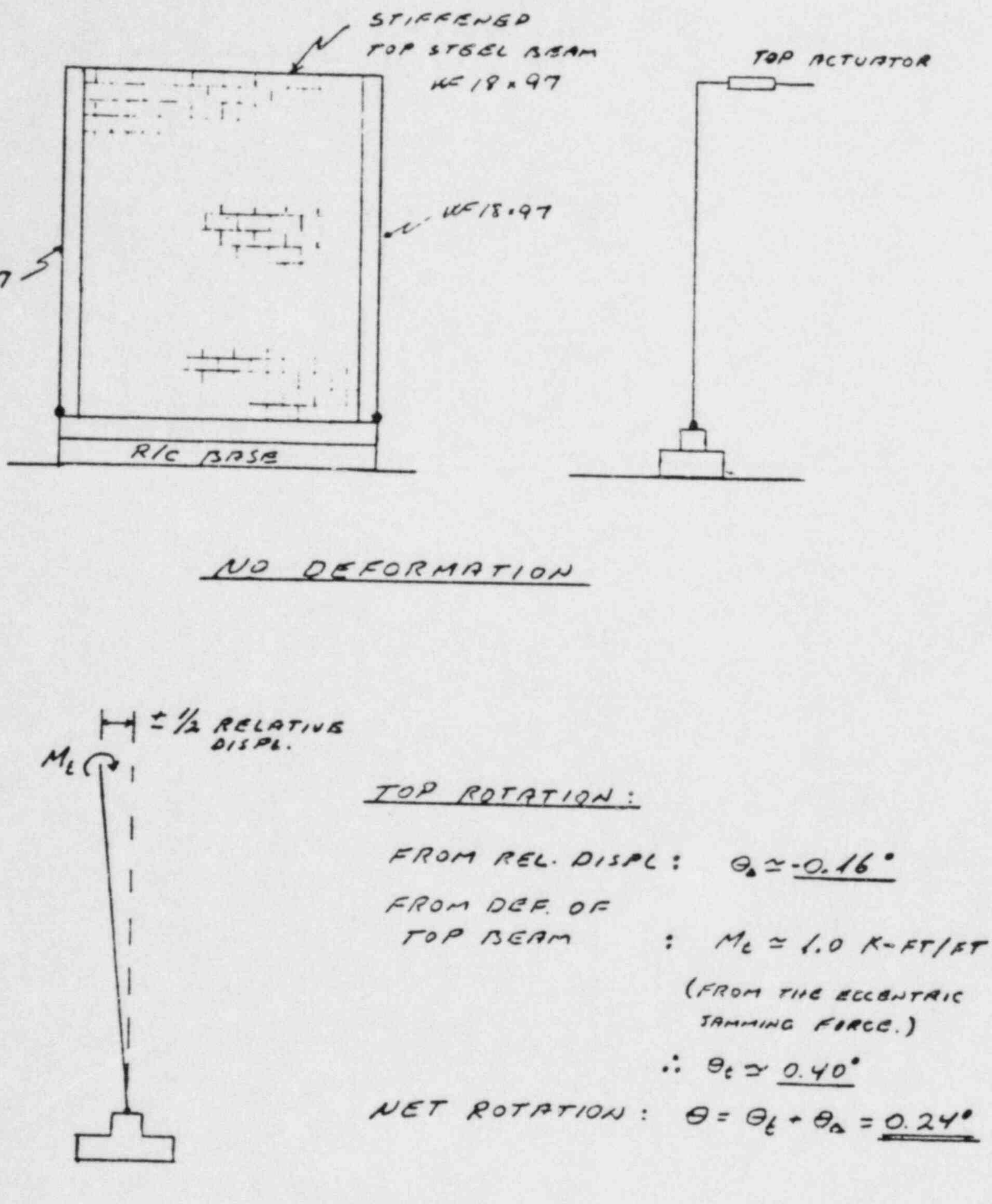


Figure 3: Estimated Rotation at Top Boundary.

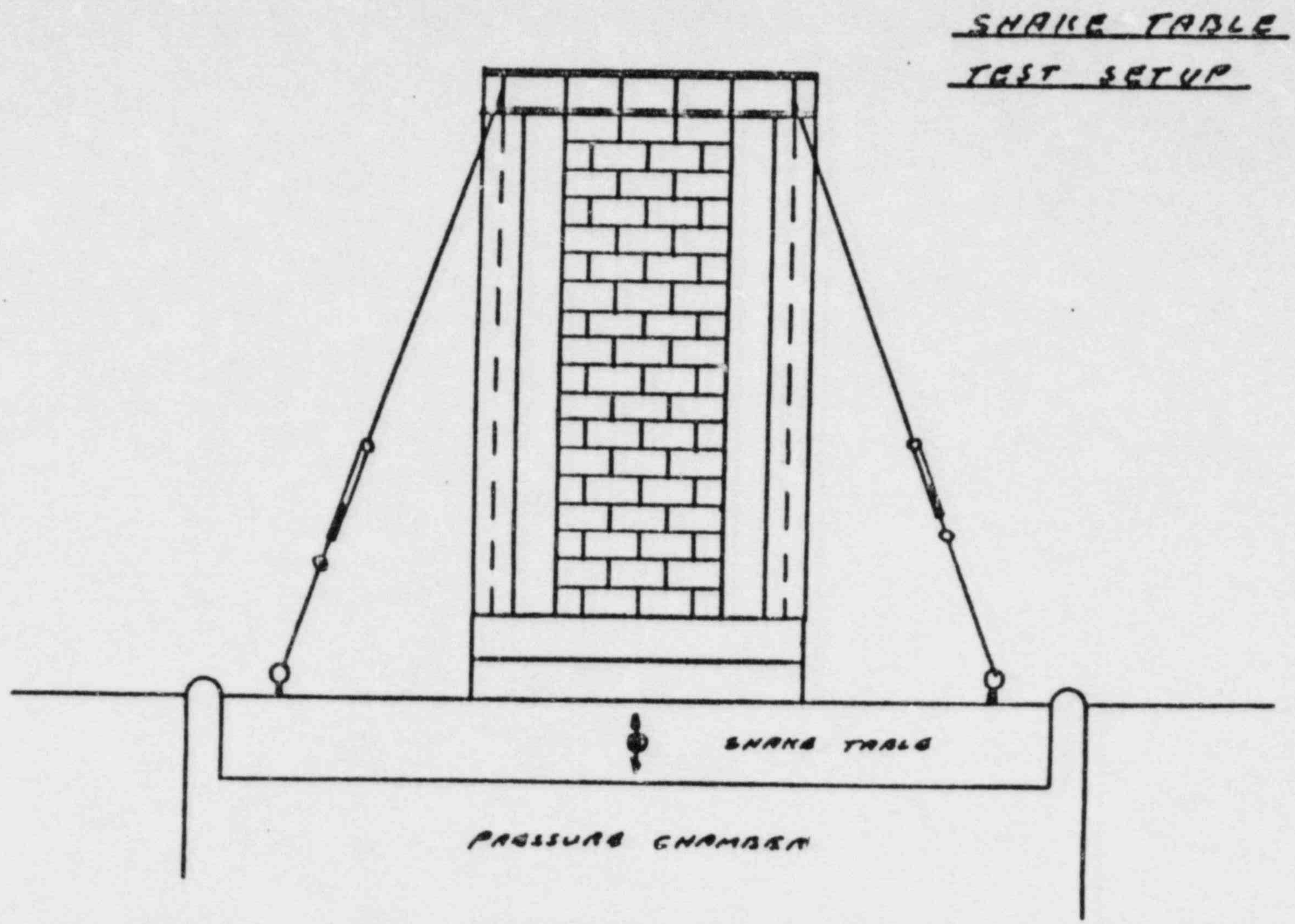


Figure 4: Alternate Shake Table Test Set-Up.

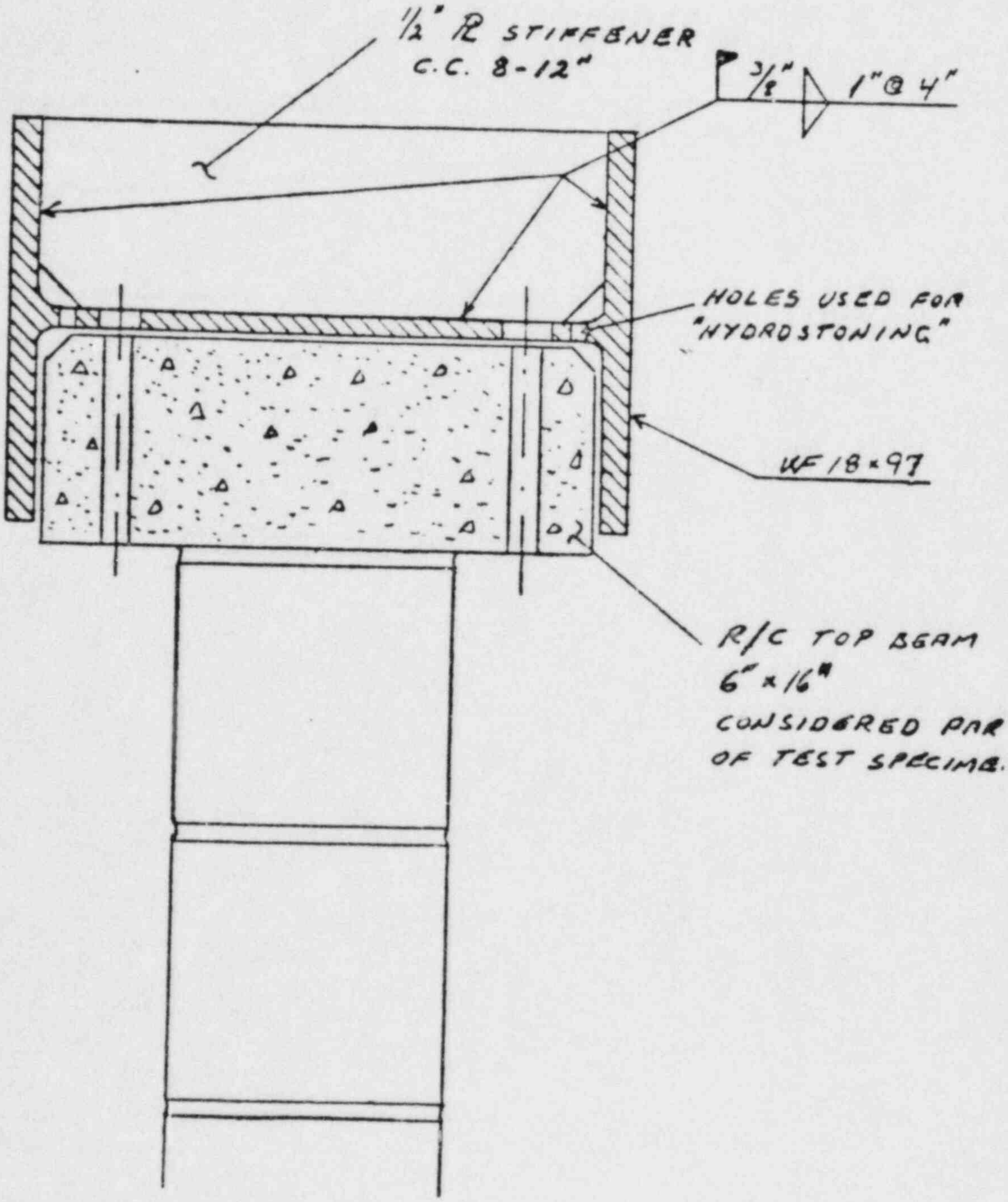


Figure 5: Top Boundary Detail.

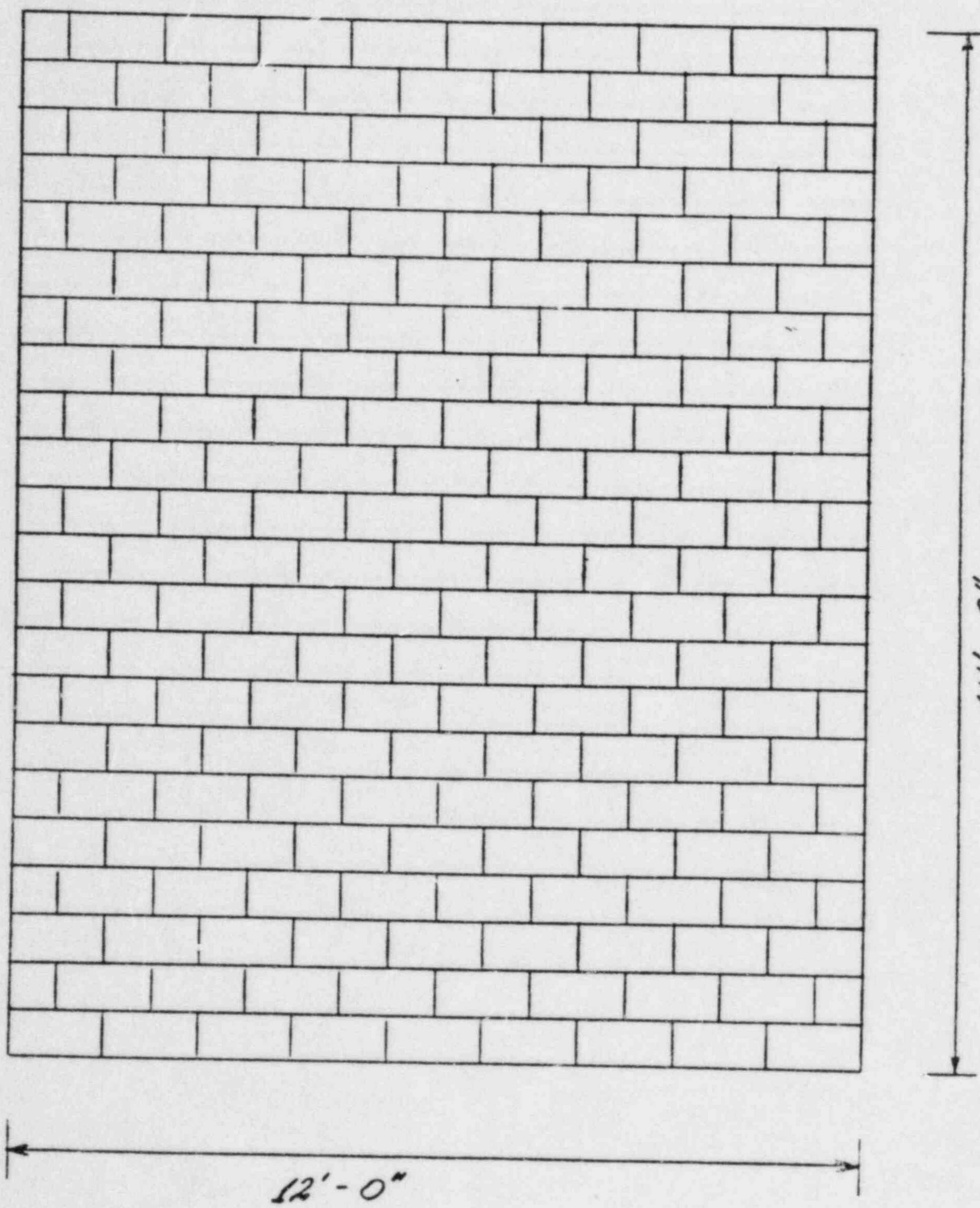


Figure 6: Sketch of Panel Type 1.

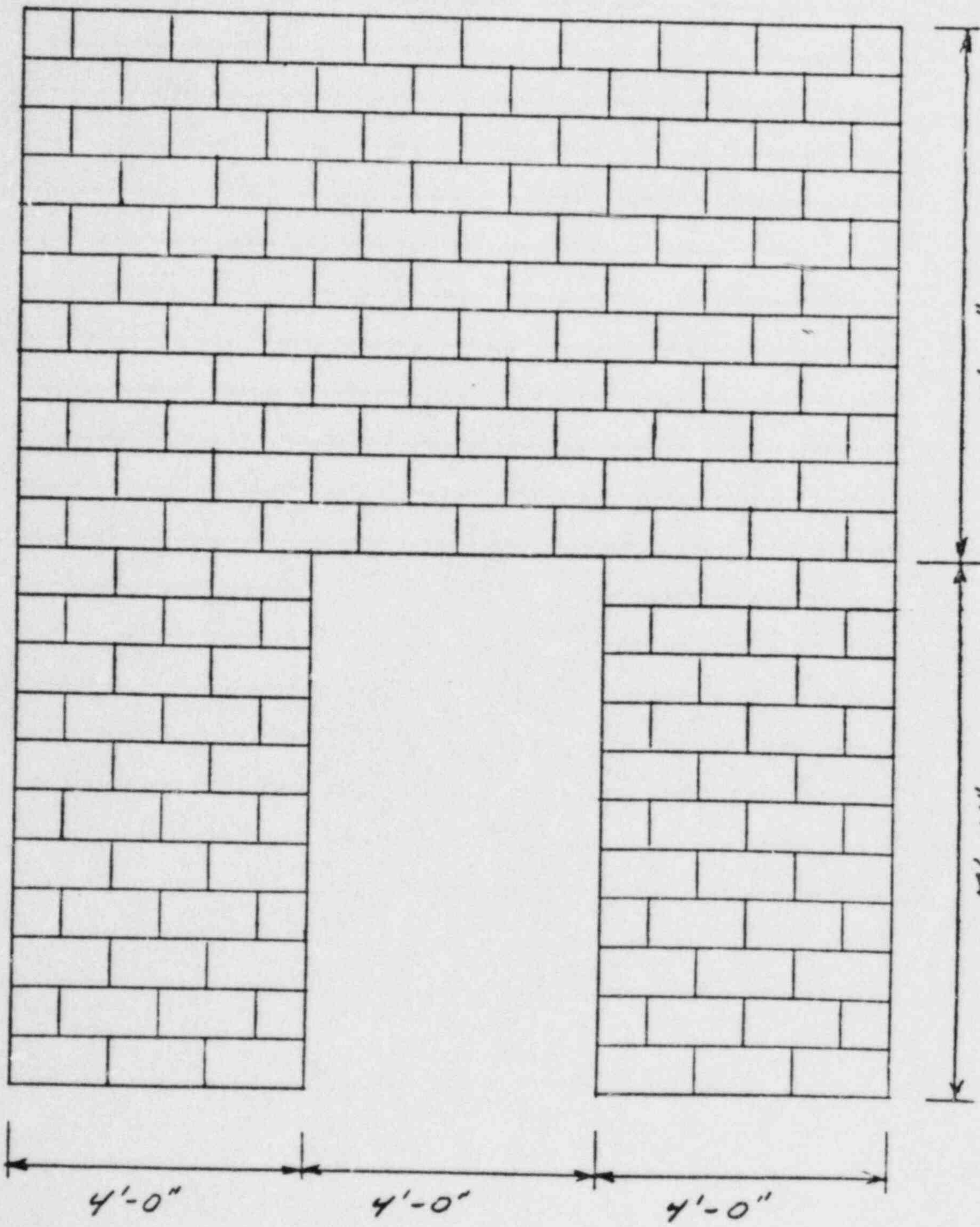
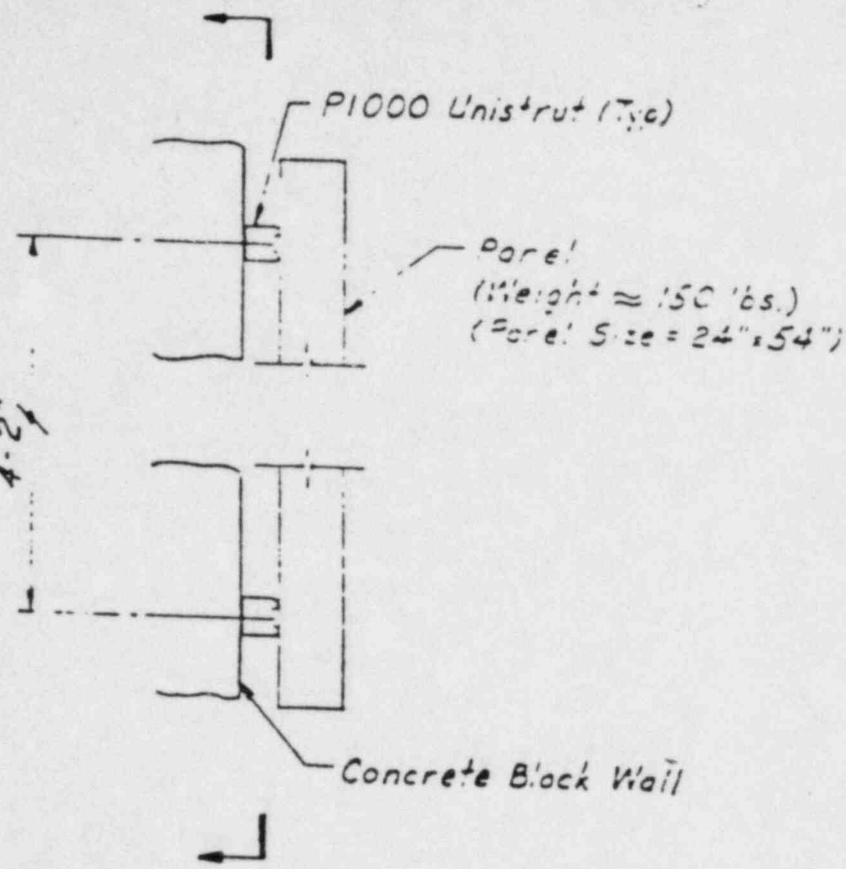
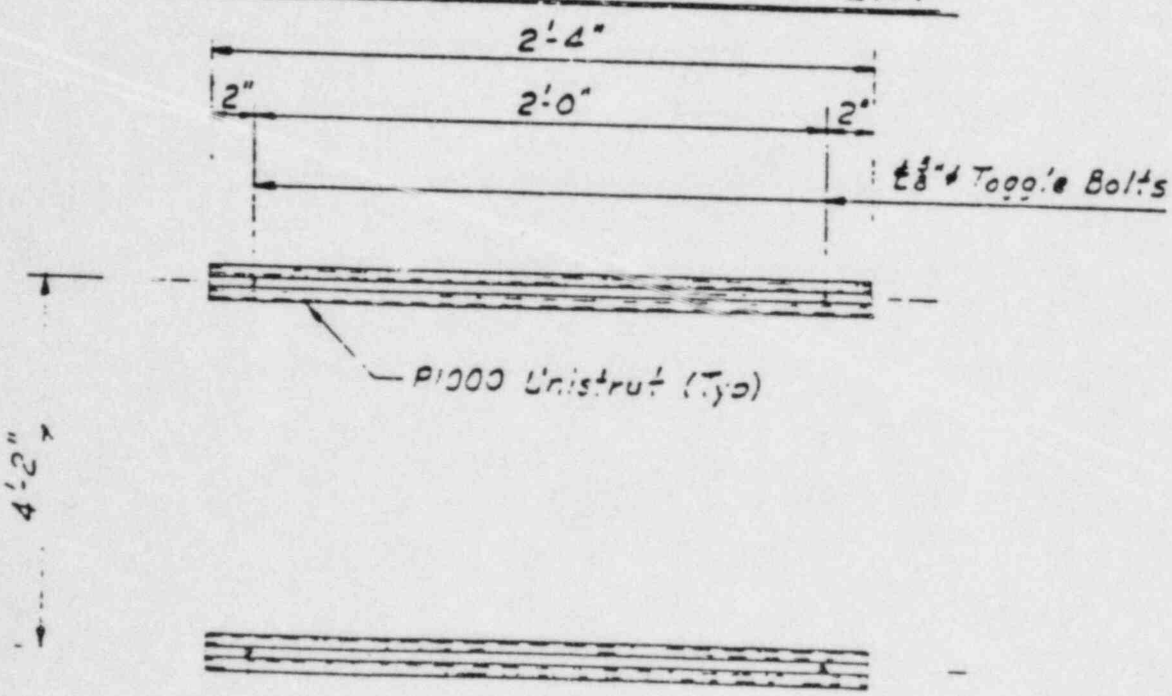


Figure 7: Sketch of Panel Type 2.

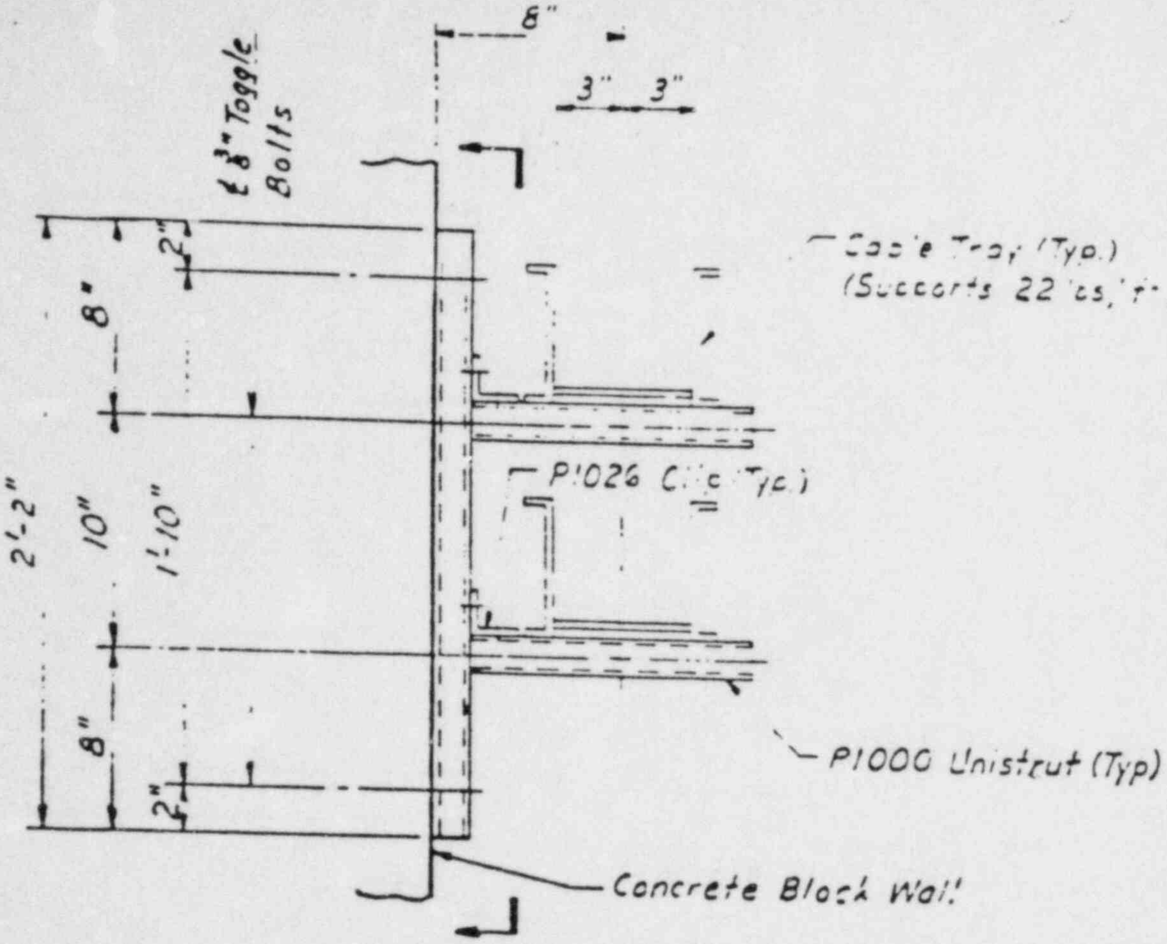


TYPICAL PANEL ATTACHMENT



SECTION

Figure 8: Typical Instrument Panel Attachment.



TYPICAL CABLE TRAY ATTACHMENT

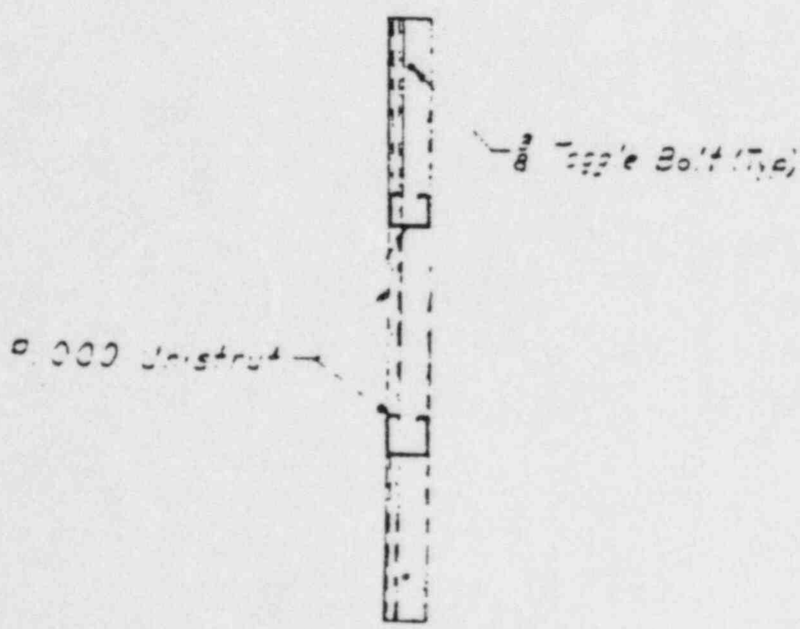


Figure 9: Typical Cable Tray Attachment.

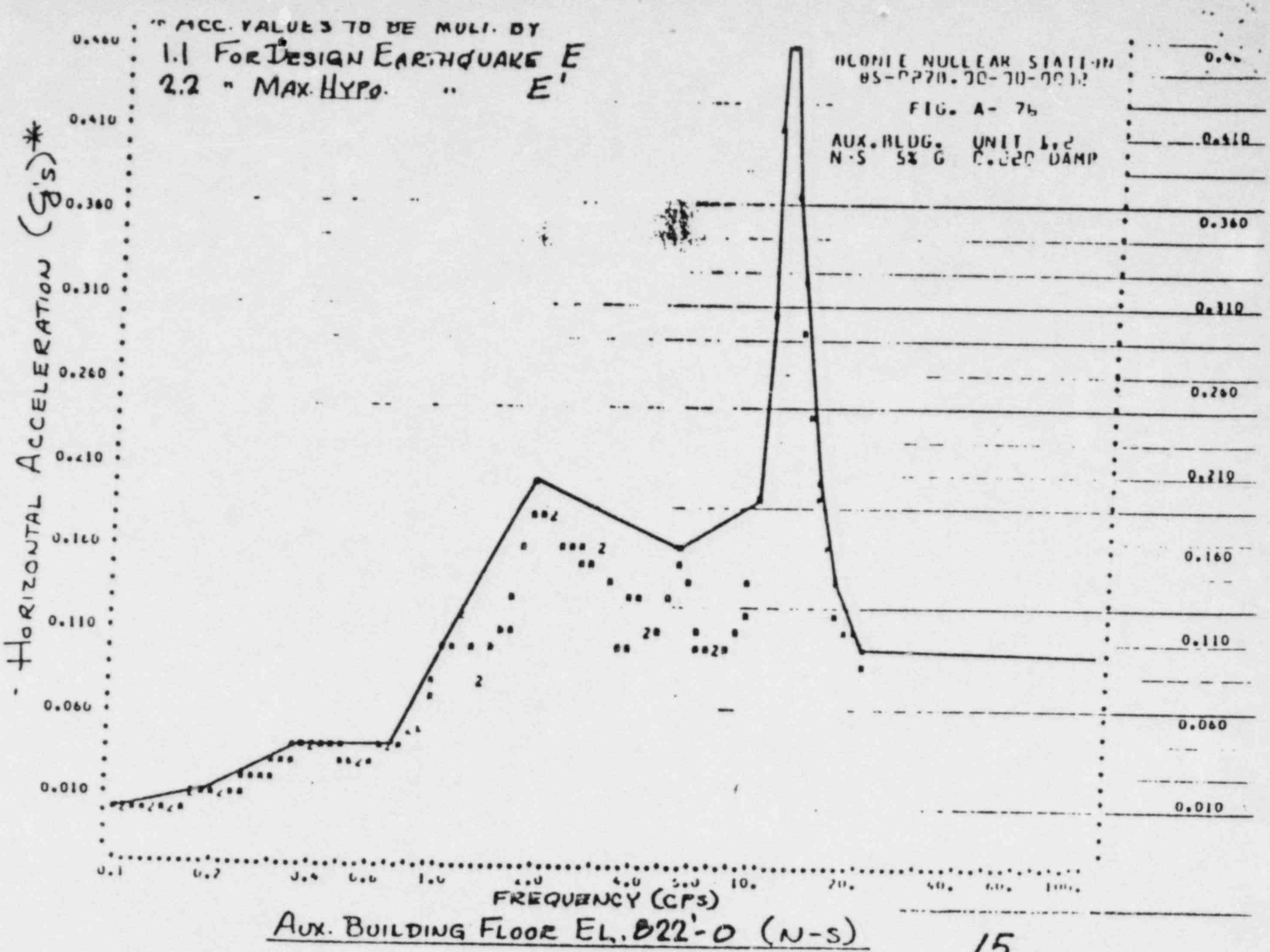


Figure 10: Typical Floor Response Spectrum.

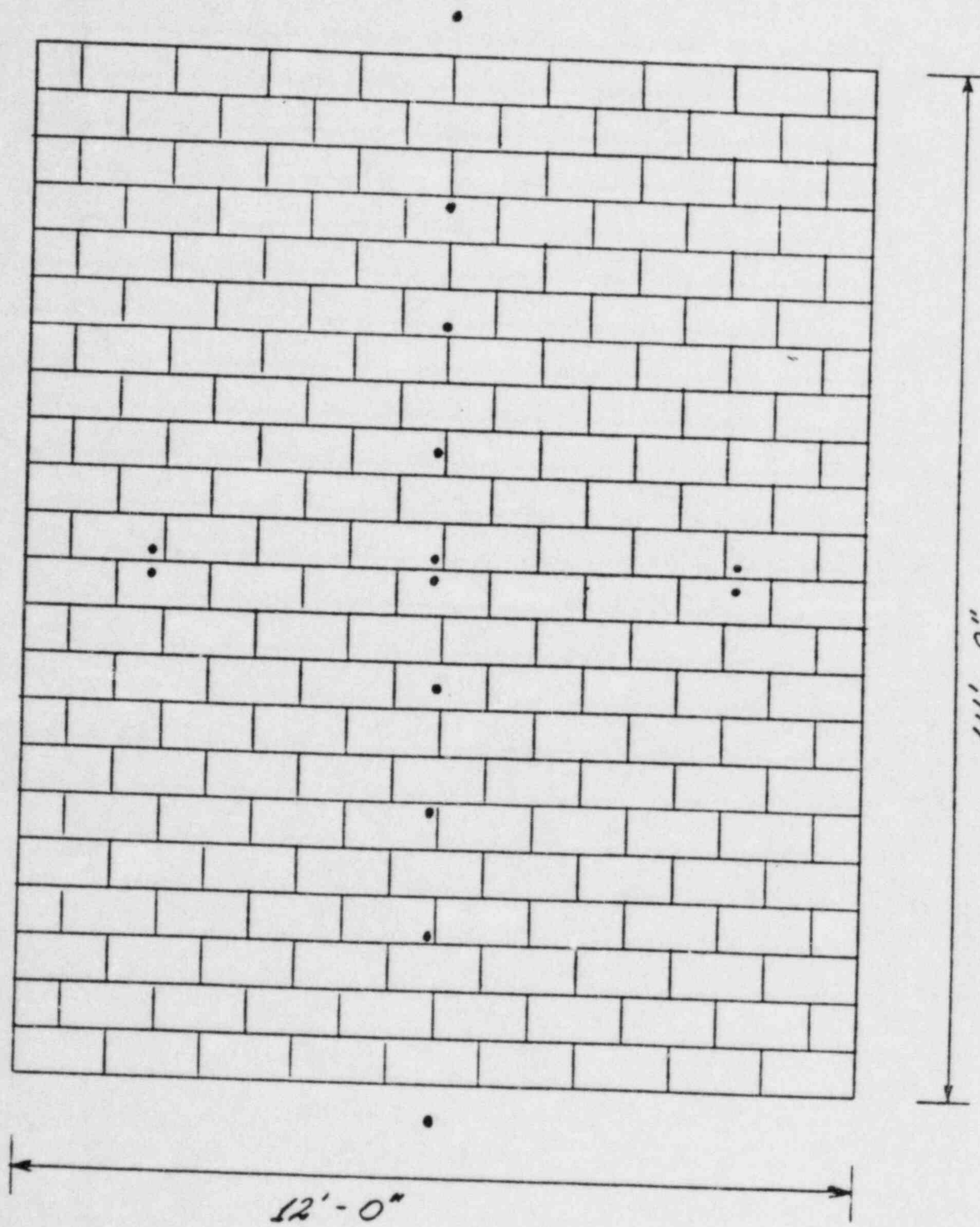


Figure 11: Location of Wirepotentiometers - Panel Type 1.

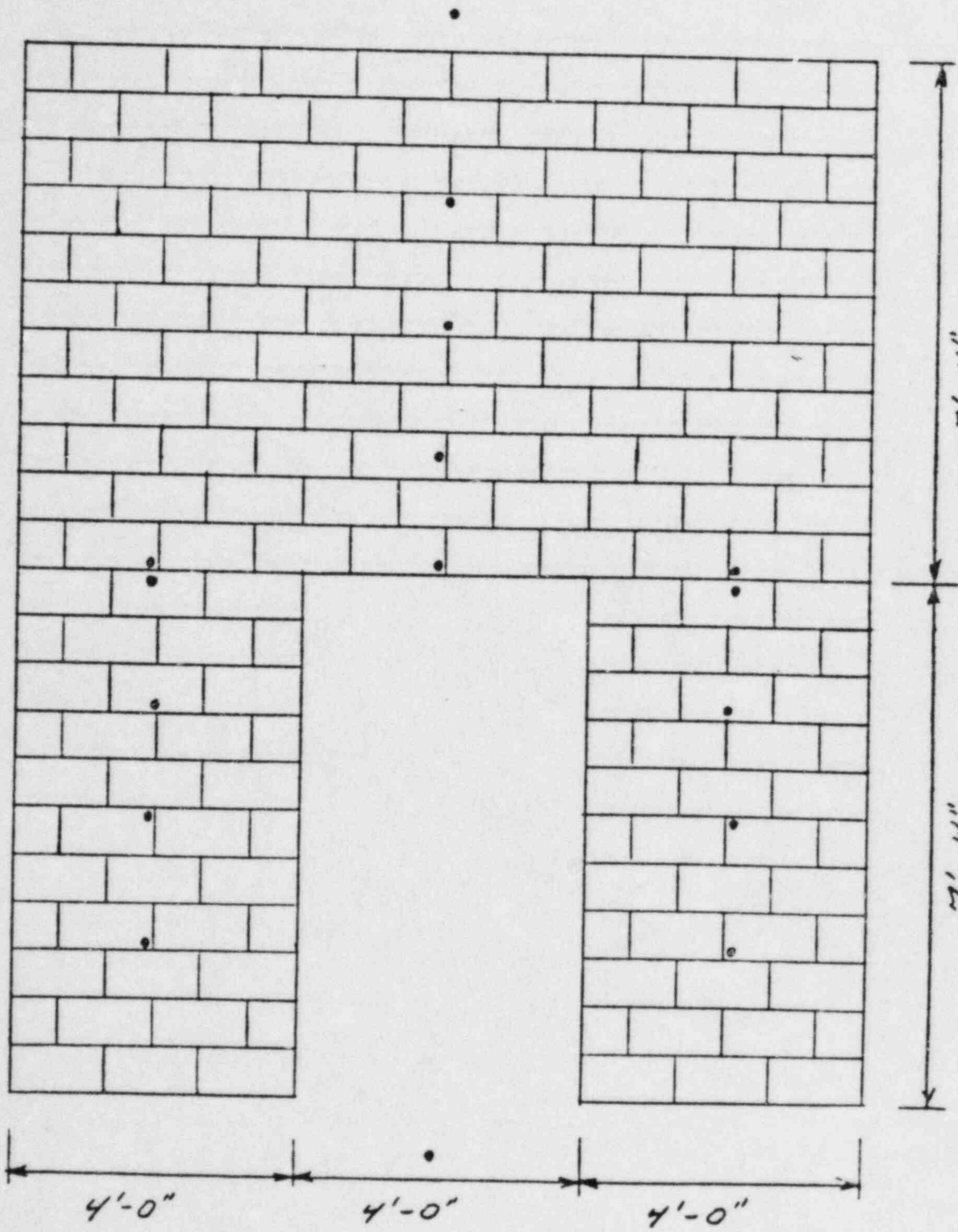


Figure 12: Location of Wirepotentiometers - Panel Type 2.

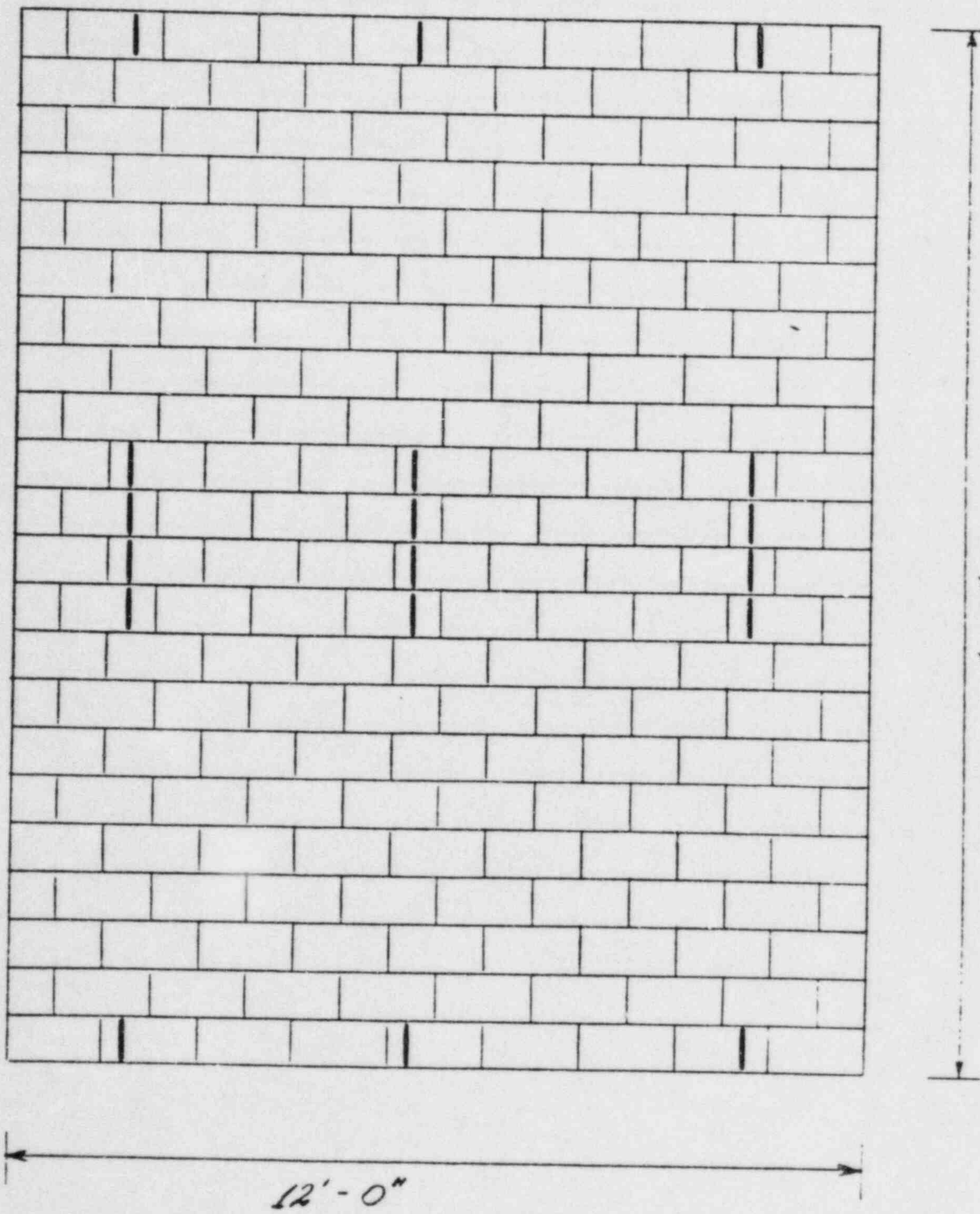


Figure 13: Location of DCDT's - Panel Type 1.

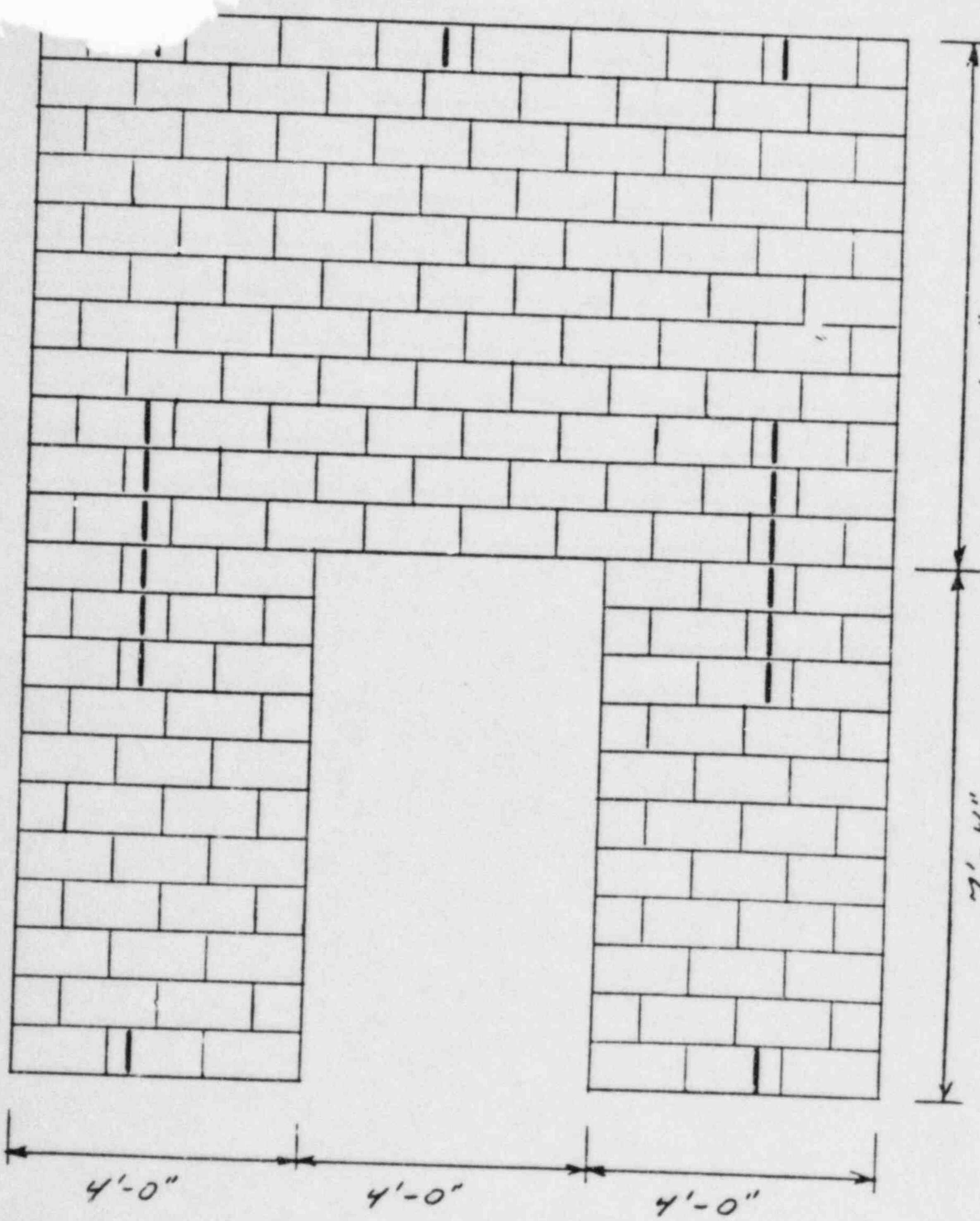


Figure 14: Location of DCDT's - Panel Type 2.

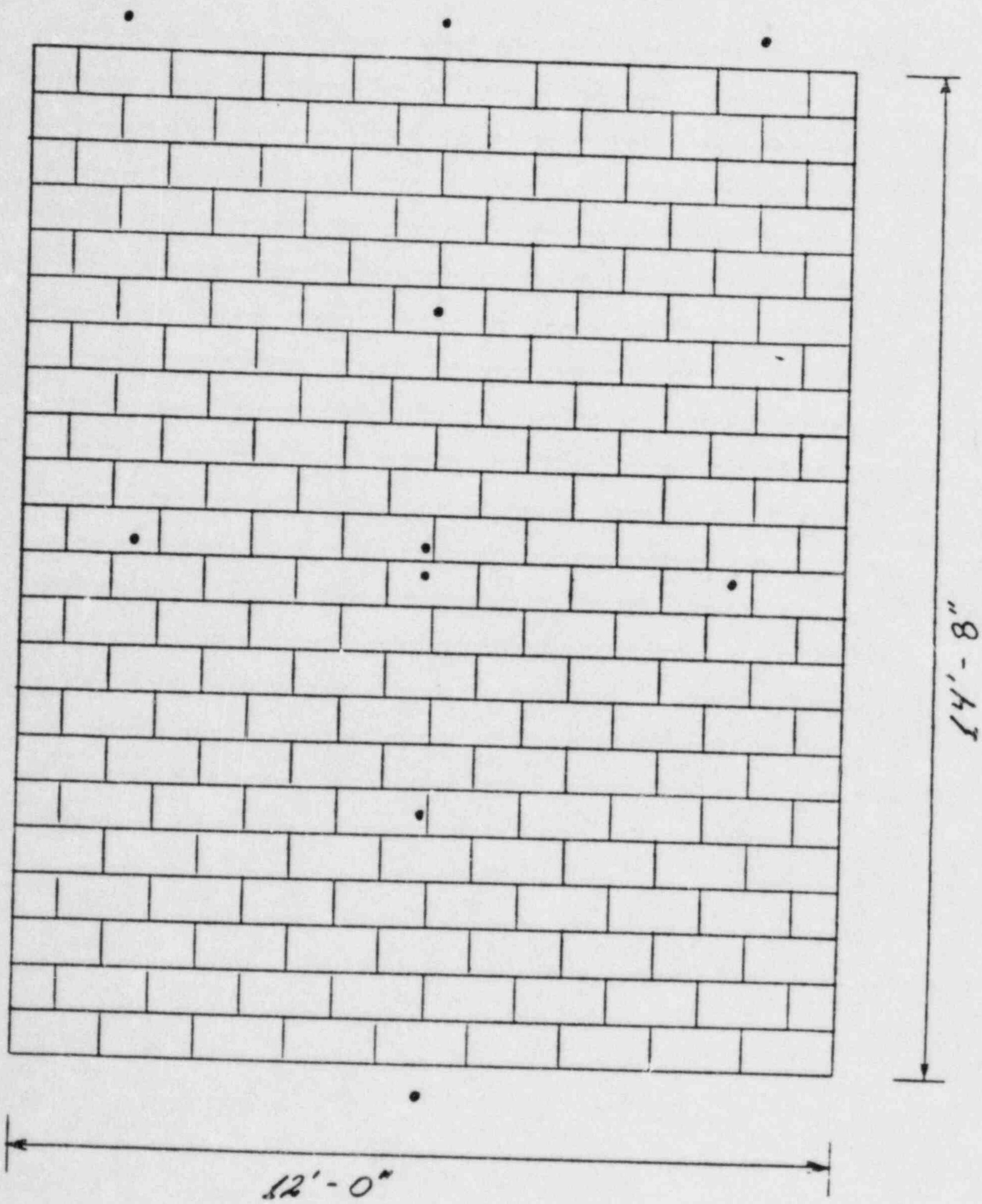


Figure 15: Location of Accelerometers - Panel Type 1.

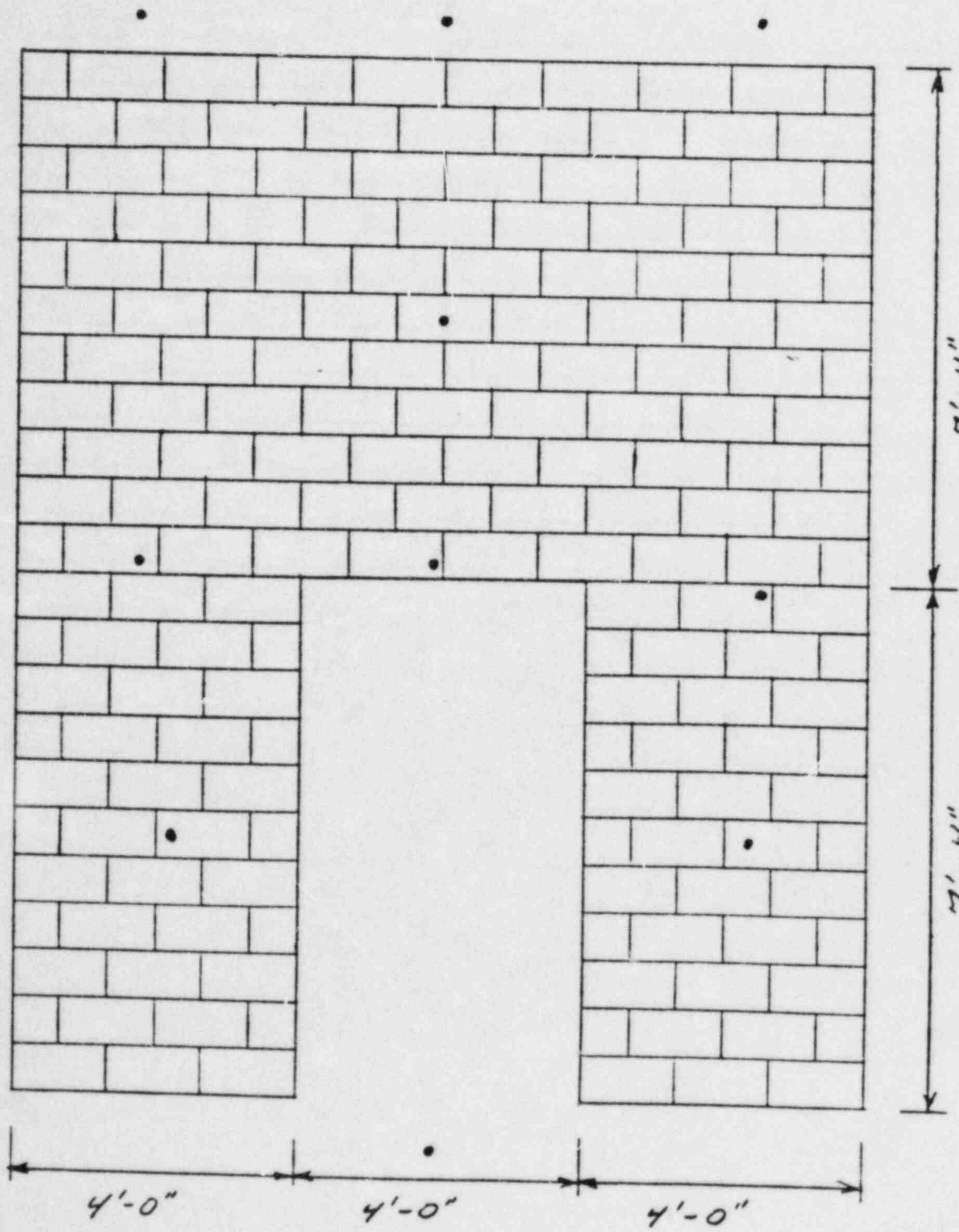


Figure 16: Location of Accelerometers - Panel Type 2.

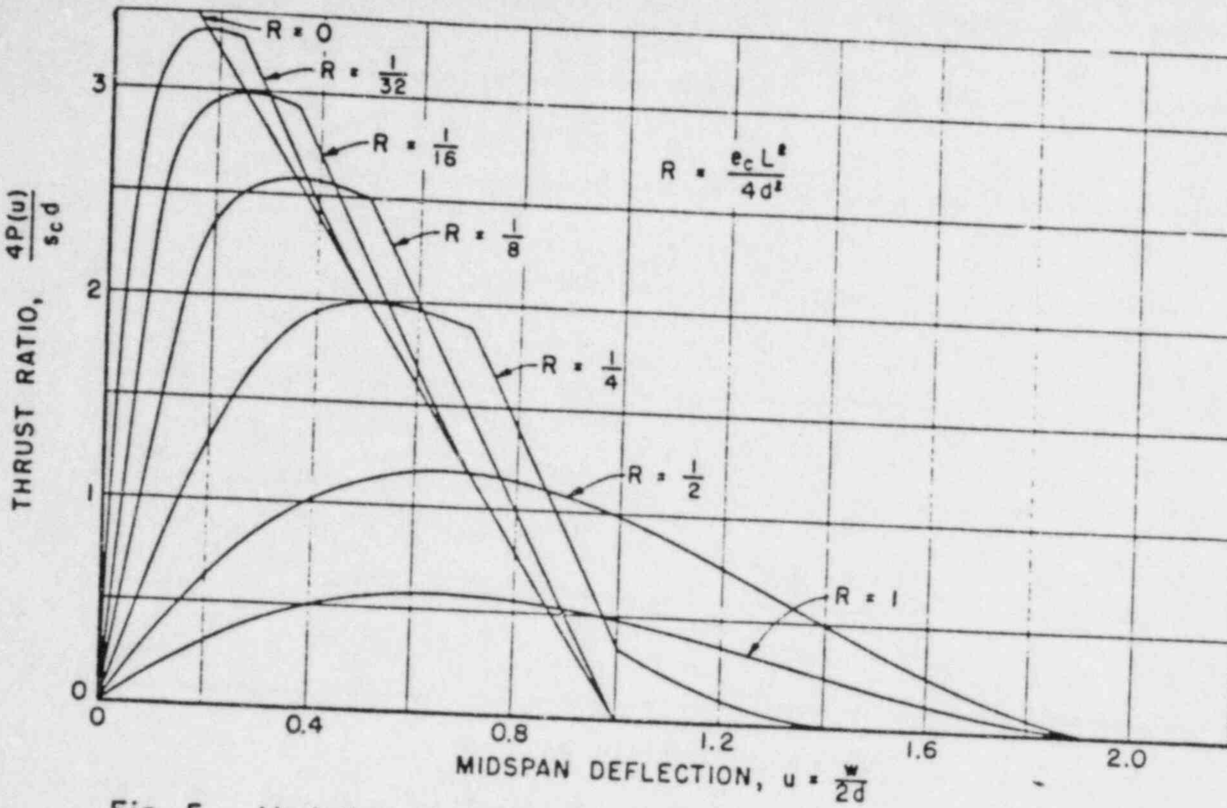


Fig. 5 Variation of Thrust Force, P(u), with Midspan Deflection

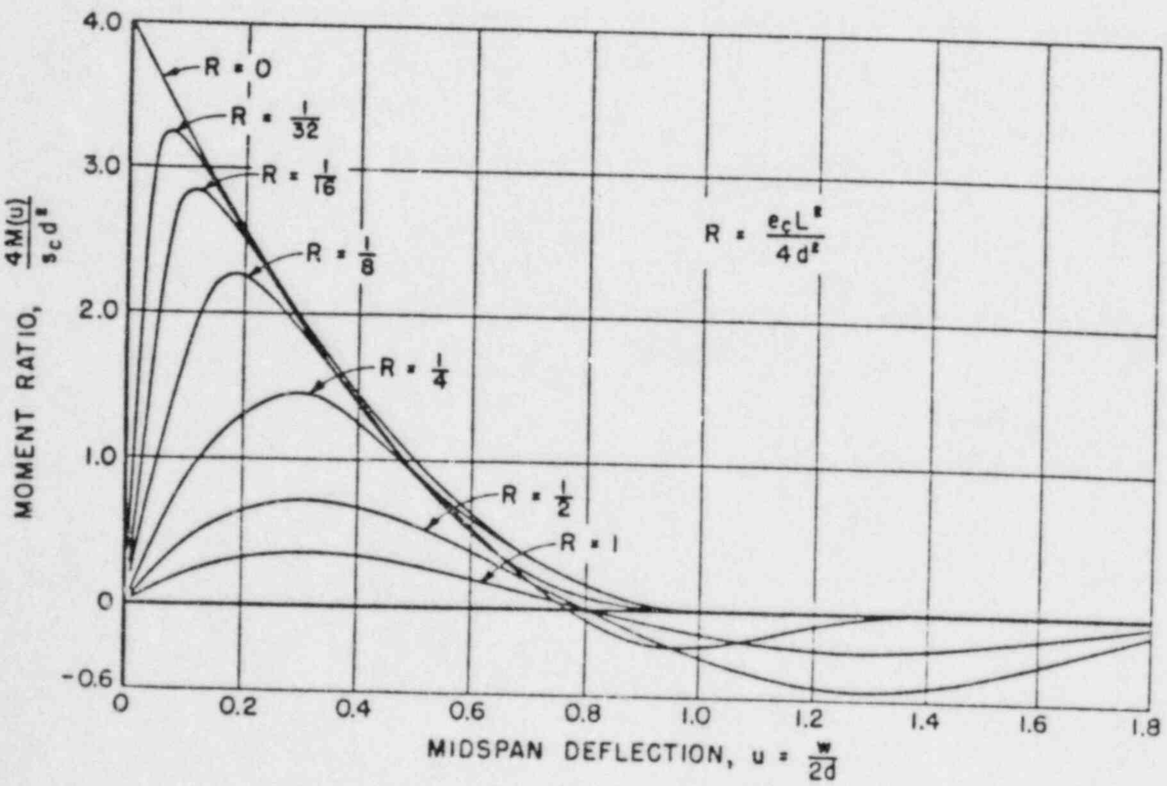


Fig. 6 Variation of Resisting Moment, M(u), with Midspan Deflection

Figure 17: Arching Action Curves - From Ref. 1.

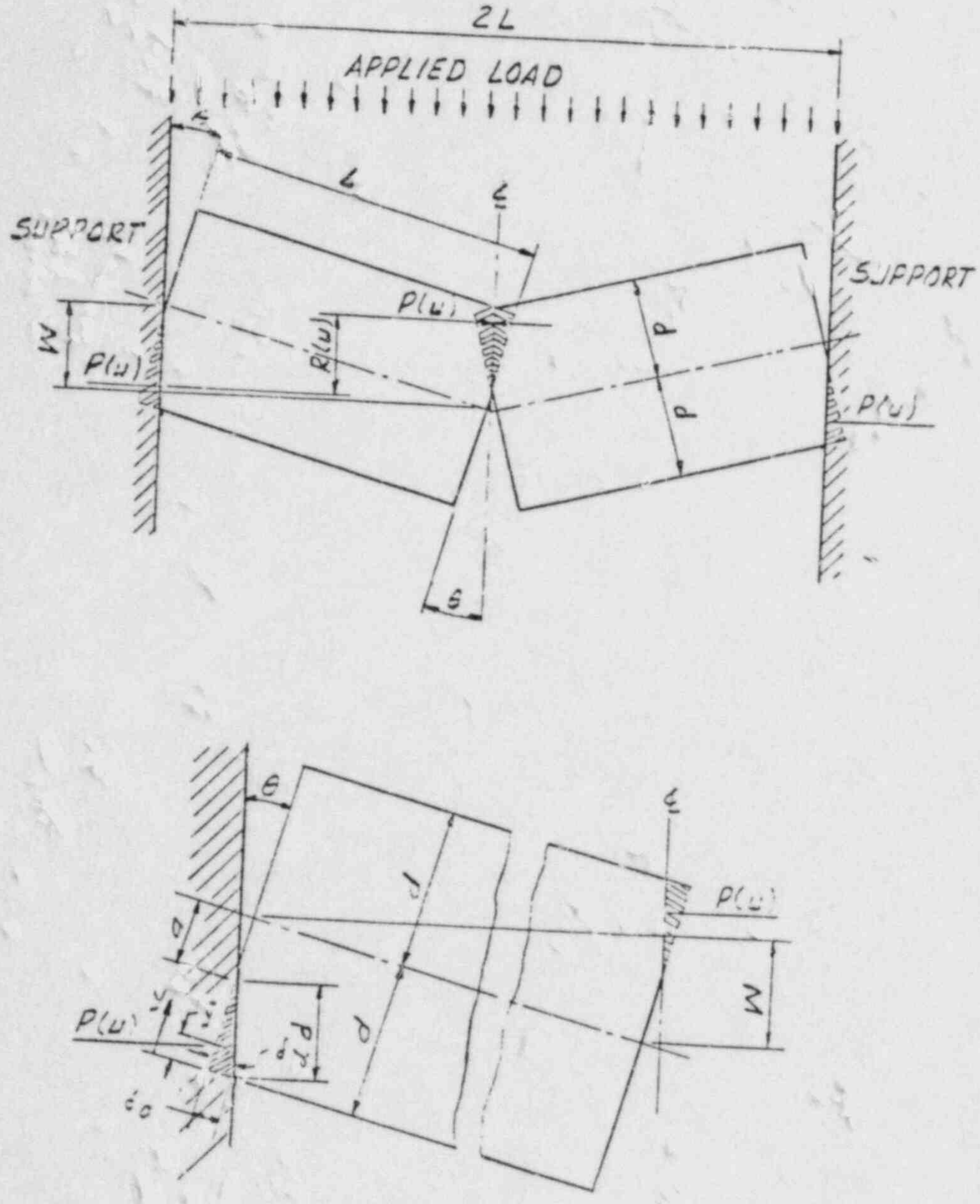


Figure 13: Geometry Arch

DUKE POWER COMPANY, CHARLOTTE, NORTH CAROLINA.

SCHEDULE: Masonry Wall Test Program.

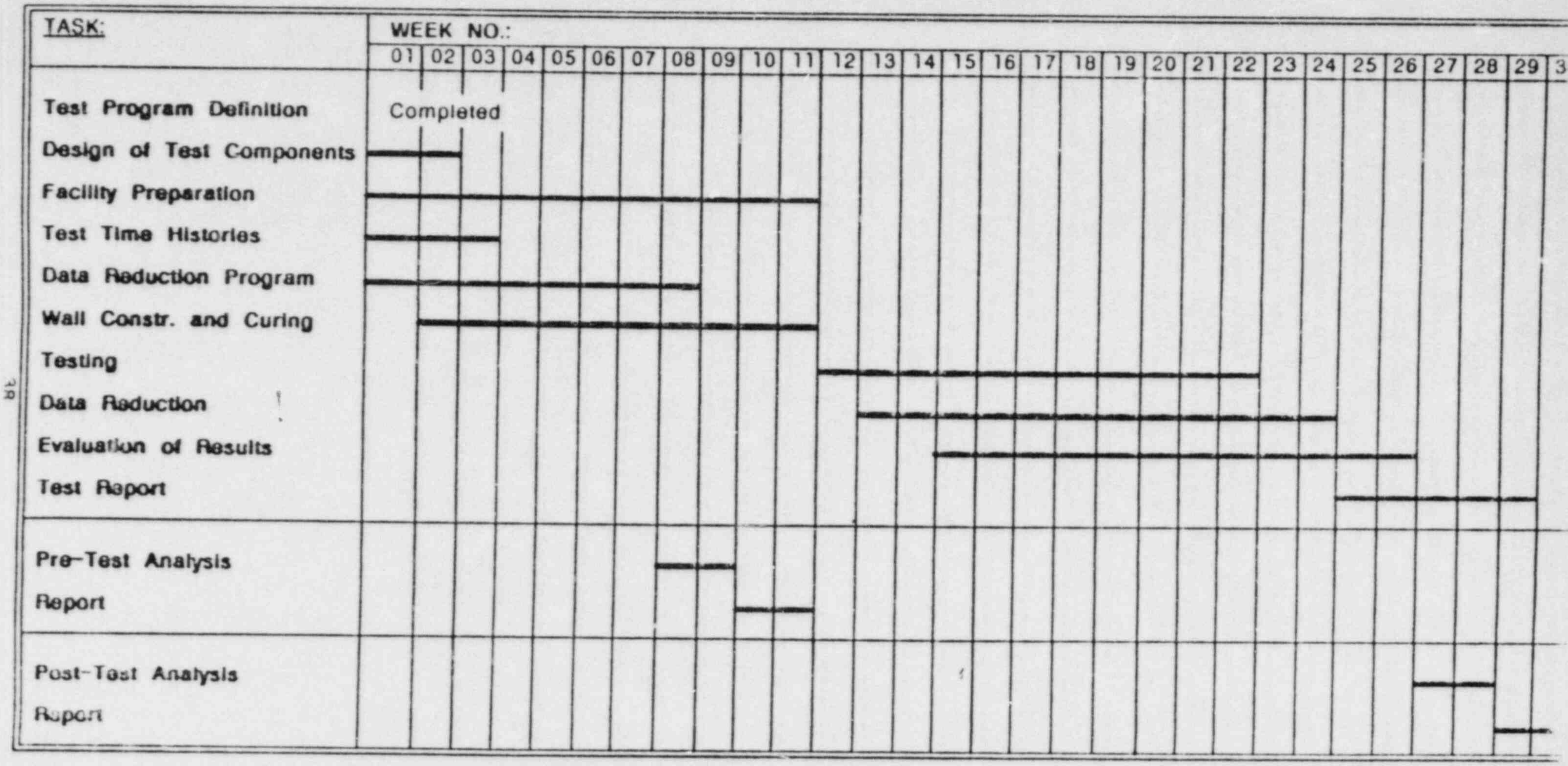


Figure 19: Overall Schedule.

APPENDIX A

EFFECT OF NOT INCLUDING SIDE BOUNDARIES

A.1 INTRODUCTION

The test program presented herein has the objective of validating the arching action analysis methodology used for the Oconee Nuclear Station. The method assumes a vertical arching mechanism based on the fact that this will provide a conservative assessment of the stability of the wall. This vertical span assumption ignores any positive benefits that arise from the restraint provided by the side boundaries.

If restraint is provided by the side boundaries, both the elastic capacity and any other two-dimensional yield line type crack pattern that develops will exceed the capacity calculated from a vertically spanning mechanism.

The purpose of this appendix is to demonstrate the conservatism of the vertically spanning assumption. This demonstration is based on elastic analysis theory by comparing frequencies (and thus spectral accelerations from the Oconee floor spectra) and maximum moments for beams and simply supported plates of various aspect ratios.

In Garielson's [1] blast load tests there were two tests that compared the post cracking behavior of a one- and two-way spanning wall. The wall was 8 ft high and 12 ft wide, constructed of single wythe 4-inch brick units. For the one-way spanning arch the test pressure that caused cracking and arching was 3 psi. For the two-way spanning wall the crack pattern was similar to a slab yield line and the pressure that caused cracking was 4.2 psi. This was a 40% increase in capacity and is similar to that predicted by elastic theory.

The discussion on the elastic capacity starts with a detailed analysis of a beam strip and then a similar treatment of a plate simply supported on four sides. The Oconee floor response spectra are then analyzed with respect to the frequencies of the above beam and plate systems and the maximum moments compared.

A.2 SIMPLE BEAM

For a uniform beam the differential equation of motion is given as (ignoring shear deformations):

$$EI \frac{\partial^4 y}{\partial x^4} + \bar{m}\ddot{y} = q(t,x) \tag{A1}$$

where EI and $\bar{m}$ are the flexural rigidity and mass, respectively. Both are uniformly distributed along the span of the beam. The function q(t,x) is the external load acting on the beam. y is the deflected shape of the beam.

Using separation of variables, simple (pinned) boundary conditions and support excitation, it is easy to show that the fundamental beam frequency is:

$$f_1 = \frac{\pi}{2L^2}\sqrt{\frac{EI}{\bar{m}}} \tag{A2}$$

and the maximum bending moment in the first mode:

$$M_1 = \frac{4}{\pi^3} q L^2 \tag{A3}$$

where $q = \overline{m} \cdot a$

and a = spectral acceleration at f_1

For simplicity, only the first mode is considered. This is adequate as the first mode is the major contributor to the beam moments.

A.3 PLATE - SIMPLY SUPPORTED ON FOUR SIDES

Similar to the beam, the differential equation that defines the motion of a uniform thin plate is:

$$D \left[\frac{\partial^4 w}{\partial x^4} + 2 \frac{\partial^4 w}{\partial x^2 \partial y^2} + \frac{\partial^4 w}{\partial y^4} \right] + \overline{m} \ddot{w} = q(x,y,t) \tag{A4}$$

where

$$D = \frac{Et^3}{12(1 - \nu^2)} \tag{A5}$$

and W is the deflected plate surface, $\overline{m}$ is the uniform plate mass per unit area, q is the external load, t is the plate thickness and ν is Poisson's ratio. D is defined as the flexural rigidity of the plate.

Using separation of variables, pinned boundaries and support excitation, it is relatively easy to show that the fundamental frequency is:

$$f_1 = \frac{\pi}{2a^2} (1 + a^2/b^2) \sqrt{\frac{E t^3}{12m (1 - \nu^2)}} \tag{A6}$$

and the two way maximum moments are:

$$M_{a1} = \frac{16 q a^2}{\pi^4} \cdot \frac{1 + \nu a^2/b^2}{(1 + a^2/b^2)^2} \tag{A7}$$

$$M_{b1} = \frac{16 q a^2}{\pi^4} \cdot \frac{a^2/b^2 + \nu}{(1 + a^2/b^2)^2} \tag{A8}$$

In equations A6, A7 and A8 a denotes the plate height, b is the plate length.

q is the product of mass and maximum spectral acceleration at f_1 and υ is the Poisson ratio. As with the beam, only the first mode is considered here since it is the major contributor to the plate moments and higher modes generally are of a secondary interest.

A.4 COMPARISON OF FREQUENCIES AND MOMENTS

Equations A2 and A6 provide a convenient way of comparing the fundamental frequency of a vertically spanning beam strip of height a with the fundamental frequency of a plate with height a and width b, and simply supported on all four sides. A similar comparison can be made for the first mode moments by using equations A3, A7 and A8.

Before the comparison of the moments is made, it is useful to introduce the relative moment capacity on a vertical and horizontal span (M_{a1} and M_{b1}, respectively). The Oconee masonry criteria permits twice the allowable stress on a horizontal span as it does on a vertical span. Therefore, when comparing equations A3 and A8 it is necessary to introduce this differential. Thus we introduce:

$$M^*_{b1} = 0.5\, M_{b1} \qquad \text{(A9)}$$

Therefore the frequency comparison is:

$$\frac{f_{1,\,plate}}{f_{1,\,beam}} = \left[1 + a^2/b^2\right]\sqrt{\frac{1}{1 - \nu^2}} \qquad \text{(A10)}$$

and the moment comparison:

$$\frac{M_{a1,\,plate}}{M_{1,\,beam}} = \frac{4}{\pi} \cdot \frac{1 + \nu \;\; a^2/b^2}{(1 + a^2/b^2)^2} \qquad \text{(A11)}$$

and

$$\frac{M^*_{b1,\,plate}}{M_{1,\,beam}} = \frac{2}{\pi} \cdot \frac{a^2/b^2 + \nu}{(1 + a^2/b^2)^2} \qquad \text{(A12)}$$

The masonry walls at Oconee Nuclear Station that were qualified by arching action have aspect ratios ranging from 0.443 to 2.160. Figures A1 and A2 show plots of frequency and moment ratios, respectively, for these ranges of aspect ratios. Note that for equation A10 the ratio a/b is restricted to ≤1. For values of a/b ≥ 1, interchange a and b and use b/a. This is not necessary for equations A11 and A12 because the comparison is with a vertical beam strip at all times. It is obvious from Figure A1 that the frequency of a vertically spanning beam strip of height a is always lower than the frequency of a plate of height a for all aspect ratios. From the shape of the floor spectra (see A5) and range of frequencies for all possible vertical beam strips, this fact will ensure at least equal but sometimes higher spectral accelerations for a vertically spanning beam strip when compared to a plate. It is also obvious from Figure A2 that the moments for equal inertia loading within a vertical beam strip are higher than the maximum moments within a plate of the same height for all aspect ratios at Oconee Station.

A.5 OCONEE FLOOR SPECTRA

Unfortunately, moments and frequencies cannot be considered in isolation of the floor spectra. The floor spectra for the Oconee Nuclear Station typically have a relatively narrow spectral peak in the 8-12 Hz frequency range and the ZPA level is reached at about 18 Hz.

The highest walls at Oconee are 180" hollow concrete block walls ($\overline{m}$ = 42 lb/ft^2). The frequency of such a vertically spanning beam strip (E = 10^6 psi; I = 334 in^4) of this height equals 9.3 Hz whereas a plate with the same height and an aspect ratio of 0.443 (height/length) has a frequency of 11.3 Hz. The ratio 0.443 is the lowest aspect ratio for the walls qualified by arching. Shorter walls have a higher frequency for the vertically spanning beam and this increases if plate action for the wall is considered. The spectral accelerations used in the arching action methodology are such that when the frequency of the wall is lower than the frequency for the peak of the spectra, then the peak value of the response spectra is used.

From the above discussion it is clear that any analysis using vertically spanning beam strips will give lower frequencies and thus higher spectral accelerations than an analysis using plate action for any particular wall in the Oconee Nuclear Station.

A.6 DISCUSSION

The objective of this evaluation has been to demonstrate the conservatism involved in the assumption that the safety of the walls can be assessed by assuming a vertically spanning arching mechanism.

If the side boundaries are not considered, it has been shown that the frequency of the wall decreases and that in all cases this decrease is conservative. Furthermore, it has been shown that the moments within the walls decrease, sometimes considerably, when side boundaries are considered; and this decrease in moment, when coupled with the decrease in spectral acceleration due to the frequency shift, will delay and possibly prevent the formation of cracks. This increase in capacity of a two-way spanning plate has also been demonstrated in the post cracking phase for one wall in Gabrielson's [1] blast load tests.

Therefore, if the wall is assumed to span vertically and any positive effect of the side boundary is neglected, then a conservative assessment of the wall will be obtained. Furthermore, if the proposed test specimens (aspect ratio a/b = 1.47) were to include side boundaries, then Figure A2 indicates that moments in the wall would only be 17% of the moments for the vertically spanning test specimen. In all likelihood this would prevent any cracks from forming during the tests.

A.7 CONCLUSION

In this Appendix the effect of including side boundaries has been studied. A comparison between a vertical beam strip approach (used at Oconee) and a plate approach (four supported sides) has been made. It is concluded that from both frequency and moment standpoint ignoring the side boundaries provides a conservative assessment of the walls.

A.8 REFERENCES

1. Gabrielson, et al., "Shock Tunnel Tests of Arched Wall Panels," Report No. 7030-19, URS Research Company, San Mateo, CA, July 1974.

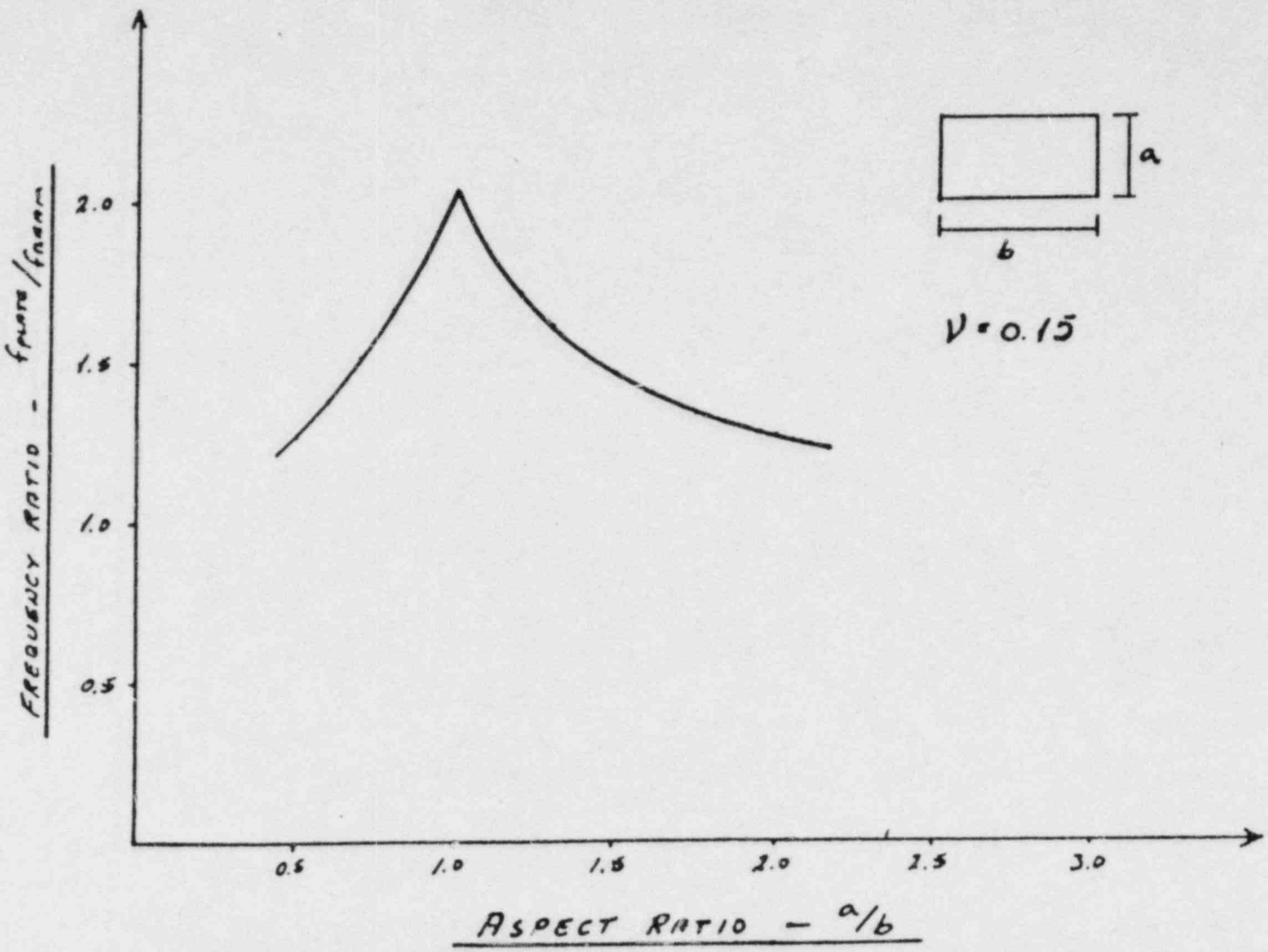


FIGURE A1: COMPARISON OF FUNDAMENTAL PLATE AND BEAM FREQUENCIES.

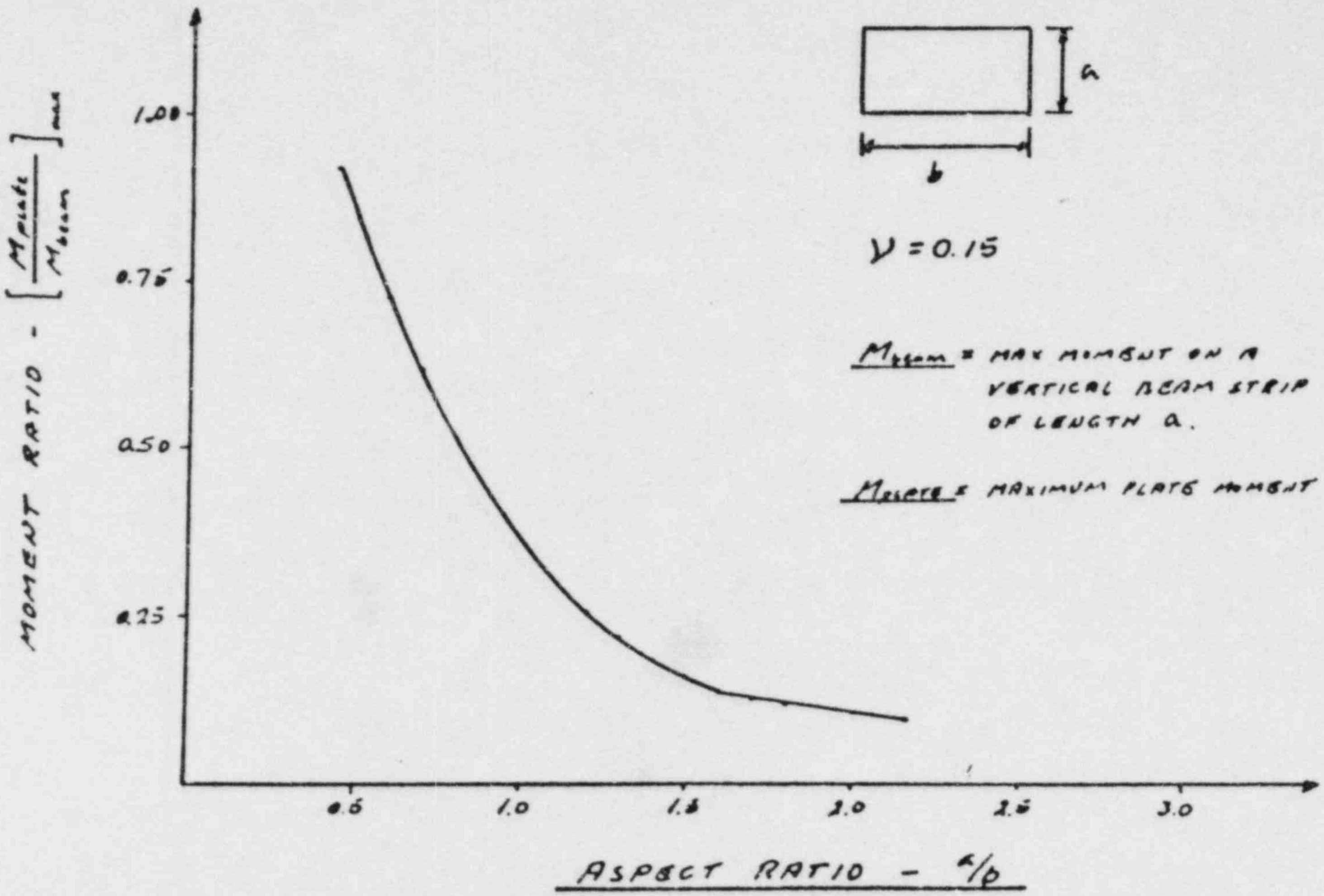


FIGURE A2: COMPARISON OF FIRST MODE MOMENTS - VERTICAL BEAM STRIP VS. SIMPLY SUPPORTED RECTANGULAR PLATE.