CRYSTAL RIVER UNIT #3

MASONRY WALL RE-EVALUATION PROGRAM

INCLUDING RESPONSE TO THE NRC QUESTIONS

04-5079-204

REVISION 0

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1.0 INTRODUCTION

1.1 PURPOSE

In response to USNRC IE Bulletin No. 80-11 Masonry Wall Design, dated May 8, 1980, Crystal River Unit #3 Nuclear Safety Related masonry walls were identified and re-evaluated. A report of the analysis and results (reference 1) was submitted to the NRC for review, as required by Item 2b of Bulletin 80-11.

Additional information about the re-evaluation program is required to complete the review (reference 2). Specifically, the NRC has listed thirteen (13) items which require responses.

This report addresses the thirteen items. Included in the discussion are the five (5) Control Complex masonry walls examined in the original report, and the Turbine Building masonry walls examined in subsequent work. The responses in Section 2 clarify the original analysis by providing additional documentation of existing wall conditions, discussing analytical assumptions, and justifying evaluation criteria. A revised evaluation including reanalysis of the Control Complex walls is presented in Section 3. Section 4 presents conclusions about masonry wall adequacy.

1.2 EXISTING CONDITIONS OF MASONRY WALLS

1.2.1 Locations and Descriptions

As described in the original report, five masonry walls were identified whose collapse would endanger Nuclear Safety Related equipment. All five are interior, non-load bearing, hollow single-wythe walls located in the Control Complex. Walls 1, 2, and 3, located at elevation 145'-0", are identified in Figure 2. Walls 4 and 5, located at elevation 95'-0", are shown in

Figure 3. Subsequent to the initial report wall 1 was modified to eliminate the safety concerns identified in the initial response to IE 80-11.

As a result of the EFW system upgrade, the Turbine Building Instrument Room walls and the Turbine Building air shaft walls also were evaluated. The Instrument Room walls, located at elevation 95'-0", are interior, hollow single-wythe walls which support a concrete deck. The air shaft walls are also interior, hollow single-wythe walls which extend from elevation 95'-0" to a concrete roof at elevation 182'-2".

The Control Complex is a Nuclear Safety Related reinforced concrete structure. As such, it has been designed to withstand such loads as tornado, OBE, and SSE. The Turbine Building is a Non-Nuclear Safety Related structure. The locations of both buildings are shown in Figure 1.

1.2.2 Drawing and Specification Review

A review of architectural drawings and masonry wall specifications provided limited information about the wall construction. The masonry units were specified as ASTM C 90-66T, Grade A (G-II). The mortar used for the masonry wall construction was designated ASTM C 270-64T, Type N. There is no evidence of significant wall reinforcing, or of shear anchors or dowels at wall boundaries.

1.2.3 Field Investigation

Additional field investigation was required to confirm wall conditions and to address the items posed by the NRC. Sketches of existing conditions of the five Control Complex walls were prepared from field information. These sketches are given in Figures 4 to 8. Sketches of the Instrument Room walls are given in Figures 9 and 10. As seen in Figure 4, a portion of wall 1 has been removed since the original evaluation. This modification was implemented in accordance with the initial IE 80-11 response.

Of primary interest is the condition of the masonry wall boundaries. With the exception of the corner shared by walls 1 and 2, all masonry corners are integral and continuous. Between walls 1 and 2 a vertical joint, shown in Figure 5, exists at the upper portion of the corner.

The wall boundaries along concrete structural elements could not be examined in detail. No destructive inspection was performed. Although inconclusive, there is no evidence of reinforcing or shear anchors at the boundaries.

All walls were constructed with running bond. No bond beams were found in the Control Complex or Instrument Room walls.

2.0 RESPONSES TO ITEMS

2.1 ITEM 1:

Provide and justify the reasons for not considering tornado loads in the analysis. Indicate if walls are subject to missile impact (both internal and external). If so, provide sample calculations (and, if necessary, provide explanations to make the calculations understandable).

Response:

All five Control Complex masonry walls whose collapse could endanger Nuclear Safety Related equipment are interior walls. Since the Control Complex is a Nuclear Safety Related structure, its components have been designed to withstand tornado loads and tornado generated missiles. The masonry walls are therefore protected or sheltered from the effects of a tornado. The interior walls will not be subjected to tornado related loads.

The Turbine Building has not been designed specifically for tornado effects. Therefore, damage to the Instrument Room walls due to tornado loads on the exposed perimeter of the Turbine Building is possible. However, an alternate emergency feedwater source protected from tornado effects is being designed as part of the EFIC upgrade program. Evaluation of the Instrument Room for tornado loading is then unnecessary because protection against tornado induced failure of the emergency feedwater system components near these walls is no longer required. Since a seismic anal sis of the Instrument Room walls was done prior to the EFIC upgrade program, these results will be included in this report for general information.

2.2 ITEM 2:

Justify the use of an allowable stress increase factor of 1.67 for load combinations containing accident pressures for SSE loads. This is in excess of several factors permitted by the SGEB criteria (1); they are listed below by type of stress:

masonry shear in flexural members	1.3
masonry shear in unreinforced shear walls	1.3
reinforcement takes entire shear	1.5
tension normal to bed joint	1.3
tension parallel to bed joint	1.5

If any existing test data will be used to justify this increase factor, discuss the applicability of these tests to the walls at the Crystal River plant with particular emphasis on the following:

- boundary conditions
- nature of loads
- size of test walls
- type of masonry construction (block or mortar type grouted or ungrouted)

The Licensee is also requested to identify walls that would not be qualified if the SGEB criteria were to be used and to specify the percentage of exceedance. The Licensee is advised to explain all conservative measures (if any) used in the analysis to justify this increase factor.

Response:

According to current criteria, a stress increase factor of 1.67 cannot be justified without adequate test data. As explained in reference 1, the 1.67 factor was obtained as the working stress equivalent to the ACI 318-63 ultimate strength requirement. The stress increase factors provided by the SGEB criteria (reference 4) will be used for the re-evaluation contained in Section 3.0 of this report.

2.3 ITEM 3:

With reference to the reinforcement in masonry walls, the ACI 531-79 Code specifies that the minimum area of reinforcement in a wall in either direction, vertical or horizontal, shall be 0.0007 (0.07%) times the gross cross-sectional area of the wall and that the minimum total area of steel, vertical and horizontal, shall not be less than 0.002 (0.2%) times the gross cross-sectional area. It should be noted that the horizontal reinforcement is installed to satisfy the minimum reinforcement requirement for a reinforced wall.

With reference to the joint reinforcement, identify the walls qualified by the tensile strength of joint reinforcement and indicate the type and spacing of the joint reinforcement for each wall.

Based on the review of existing codes and published literature, the NRC does not, at present, approve the use of joint reinforcement as a structural element.

Response:

The masonry specifications and plant drawings provide no information about wall reinforcing. At most the walls contain "DUR-O-WALL" type joint reinforcing, installed in horizontal joints at 16" on center vertically. The walls clearly do not meet the minimum reinforcing requirements for reinforced masonry as given in ACI 531-79. All masonry wall evaluations must be based on unreinforced masonry requirements.

The original re-evaluation considered the walls as unreinforced masonry. They were not qualified by the tensile strength of joint reinforcement.

2.4 ITEM 4:

Indicate the boundary conditions used in the analysis and verify that they resemble the real physical conditions. Identify all of the mechanisms used to transfer shear and moment (if any). If any doubt exists (i.e., whether simply supported or fixed-end conditions should be assumed), verify that the assumed boundary conditions will produce conservative results.

Response:

Boundary conditions used in the original analysis considered wedging action (arching) as a mechanism for shear transfer across a boundary. Subsequent research has provided limitations on application of the wedging effect. Studies indicate that for wedging to occur in an unreinforced masonry wall rigid confinement must occur at the edges so that in-plane loads can be developed. Boundary stiffness must be extremely high in both the normal and rotational directions. Current industry guidelines generally require slabs and wall systems of 2'-0" thickness or greater as boundaries for justification of wedging. In addition, questions have arisen as to whether magnitudes of seismic displacements are sufficient to cause wedging.

Based upon the actual dimensions of the structural boundary elements and the question of seismic applicability to wedging action, the boundary conditions have been modified. The revised conditions are conservative, assume no wedging action, and are more easily justified.

<u>Boundary:</u> **Base (See Figure 11 for notation)**

Previously Modelled Condition: $\Delta x = \Delta y = \Delta z = \Theta y = \Theta z = 0$; "knife edge" support (simply supported)

Modified Condition: $\Delta x = \Delta y = \Delta z = \Theta x = \Theta y = \Theta z = 0$; "fixed" support

Actual Condition: No shear anchors or dowels exist to transfer shear or moment across the boundary. Shear must be transferred by mortar bond, friction, and grain interlock. These mechanisms are reliable since a net vertical compressive force exists at the boundary due to wall dead weight. Therefore, translation restraint is expected. Additionally, mortar tension and dead weight pressure distribution provide Θx restraint.

<u>Boundary:</u> Side edge along concrete structure

Previously Modelled Condition: $\Delta x = \Delta z = \Theta x = \Theta z = 0$; "knife edge" support with free vertical translation

Modified Condition: No restraint; "free" edge

Actual Condition: No shear anchors or dowels exist to transfer shear or moment across the boundary. Mortar at the boundary is subject to significant cracking due to shrinkage and relative displacement. Consequently, mortar bond cannot be assumed. Grain interlock and friction are questionable since no reliable compressive normal force exists across the boundary. Although some shear development can be expected, its magnitude is questionable. Mortar cracking at the boundary also prevents reliable Δx and Θy resistance.

Boundary: Side edge at continuous masonry corner

Previously Modelled Condition: $\Delta z = \Theta x = \Theta z = 0$; "knife edge" support with free in-plane translation

Modified Condition: $\Delta z = \Theta x = \Theta y = \Theta z = 0$; "fixed" support with free in-plane translation

Actual Condition: The corner is integral and continuous with both intersecting masonry walls. No vertical mortar joint is formed. Shear and moment transfer rely upon the integrity and load capacity of the masonry units.

Boundary: Top edge along concrete structure

Previously Modelled Condition: $\Delta z = \Theta y = \Theta z = 0$; "knife edge" support with free in-plane translation

Modified Condition: No restraint; "free" edge

Actual Condition: No shear anchors or dowels exist to transfer shear or moment across the boundary. Possible mortar cracking and lack of a dependable normal compressive force eliminate reliable shear development. Similarly, resistance to Θx cannot be assumed.

Boundary: Top edge along supported slab

Modelled Condition: $\Delta x = \Delta z = \Theta y = \Theta z = 0$; "knife edge" support with free vertical translation

Alternate Condition: $\Delta z = \Theta x = \Theta y = \Theta z = 0$; "fixed" support with free in-plane translation

Actual Condition: Although no shear anchors or dowels exist to transfer shear or moment across the boundary, the cast-in-place condition of the supported slab results in significant bond and interlock. Additionally, the dead load of the supported slab provides frictional resistance at the boundary. With the supported slab acting as a diaphragm, a condition similar to a wall base results. Translation restraint and Θx restraint is expected.

Boundary: Side edge along steel structure

Modelled Condition: $\Delta x = \Theta z = 0$; "free" edge

Alternate Condition: No restraint; "free" edge

Actual Condition: The masonry wall is not mechanically tied to the steel structure. No mechanism is available to transfer shear or moment across the boundary. Restraint is not available.

Boundary: Edges along various openings

Previously Modelled Condition: Either unrestrained, or "knife-edge" support

Modified Condition: No restraint; "free edge"

Actual Condition: No restraint can be supplied by ductwork or door frames.

The assumption of free boundaries where no reliable mechanisms exist to transfer shear or moment results in conservative through-thickness bending stresses adjacent to "fixed" edges. Boundaries applied in the original analysis provide two-way action, whereas the modified boundaries do not. Section 3.0 wall contains masonry wall reanalysis based on these boundary conditions.

Discussion of "alternate" conditions versus actual modelled conditions is found in Section 3.2 for the Turbine Building Instrument Room walls.

2.5 ITEM 5:

Indicate how interstory drift effects, both in-plane and out-of-plane, were considered in the analysis. Also, indicate and justify by available test data the permissible strains used for both confined and unconfined walls.

Response:

The original analysis did not consider interstory drift. Subsequent calculations (see Section 3.1.3) demonstrate that drift effects are within recommended limits.

In-plane strain criteria used to verify adequacy of the walls is discussed in reference 5. This reference suggests an allowable shear strain of 0.0001 for unconfined concrete block masonry walls. The recommended strain limit for the five Control Complex walls if considered confined is 0.001.

Out-of-plane drift effects are evaluated by determining flexural stresses developed by the relative drift displacement and comparing them to ACI allowables.

2.6 ITEM 6:

Indicate whether concrete block walls are stacked or running bond. If any stack bond wall exists, provide sample calculations for stresses in a typical wall. Also identify the number of stacked bond walls and their appropriate allowable stresses.

Response:

The masonry walls are running bond.

2.7 ITEM 7:

In Section 3.5.1 of Reference 3, it is indicated that the computed stresses are increased 5% to account for higher modes of vibration. Justify by sample calculation that 5% is an appropriate percentage of multimode effects.

Response:

Based on a parametric study, reference 5 recommends that the first mode seismic acceleration of unreinforced masonry walls be increased by a factor of 1.05 to account for higher modes of vibration. This study is based on walls having adequate perimeter support. Section 5.1 of this report presents results which consider multiple mode response of the five Control Complex masonry walls. Section 5.1 shows that in the highest stressed regions of walls 2, 4 and 5, the 1.05 factor appears reasonable. These walls, unlike walls 1 and 3 , are at least partially restrained on three sides. However, to assure higher modal participation is included, all five walls are analyzed using the first eight modes of vibration (wall 1 is re-checked using 12 modes).

2.8 ITEM 8:

Provide sample calculations (with explanations necessary to make the calculations understandable) for:

- **a single-wythe wall analysis**
- **tornado loads available (if applicable)**

Response:

Sample calculations for a single-wythe unreinforced masonry wall evaluation are included in Section 5.2. No tornado load analysis is required.

2.9 ITEM 9:

Indicate how the uncertainties due to variations in mass, material and sections properties were accounted for in the analysis.

Response:

The design responses of the Instrument Room walls, wall 4, and wall 5 were obtained from the site Design Ground Response Spectra. These curves, provided in the original report, were developed by standard methods (reference 6, Volume 1, Section 2.5.4.2) and show characteristics of the smooth, broad-band response spectra required to include uncertainties in the analytical model. Figure 12, a replot of the design curves as acceleration vs. frequency, demonstrates this clearly. For the frequencies involved in the original analysis, the lowest being 16.7 cycles/sec. for Wall 4, the response is nearly insensitive to change in frequency. Reasonable uncertainties in mass, material, and section properties will have a small effect on wall frequencies, and a negligible effect on the corresponding spectral acceleration.

The design responses of walls 1, 2 and 3 were given by the Control Complex Design Floor Response Spectrum at elevation 145'-0" for 1/2% damping. This curve, shown in Figure 13, envelopes the floor response curves of both the major and minor axis building responses obtained by dynamic analysis. The enveloping technique produces a curve with smooth, broad-band characteristics. As found in Section 3.0 of this report, the resulting design curve compares favorably with a broad-band response suggested by Regulatory Guide 1.122 to account fo uncertainties in modeling.

Additionally, response conservatism is included by the use of 1/2% damping for walls 1, 2 and 3. SGEB criteria (reference 4) allows 4% damping for OBE and 7% damping for SSE. The Control Complex Design Floor Response curve at elevation 145'-0" for 4% damping is included in Figure 13 for comparison.

2.10 ITEM 10:

Reference 3 indicated that several areas of the plant, such as the containment, t' waste gas storage tank room, the spent resin storage tank room, the deborating demineralized room, the cation demineralizer room, and the air shaft were inaccessible. Confirm whether a field survey has been conducted to verify that no safety-related equipment is jeopardized by masonry walls in these areas according to construction drawings. If any discrepancy exists, provide explanations and/or remedial actions and a schedule of completion.

Response:

All available documentation including construction drawings indicate that there are no masonry walls in the containment building, waste gas storge tank room, deborating demineralizer room, cation demineralizer room, and makeup and purification demineralizer room. Site verification to assure absence of masonry construction in these areas is not possible due to radiological conditions at these locations.

2.11 **ITEM 11:**

Since there are no QA/QC records available, provide any test data to justify the allowables used in the analysis. Also, identify the year (date of publication) of ACI 531 used in the analysis. Indicate and justify any higher stress allowables when compared to ACI 531-79. If using any tests that are different from on-site tests, the licensee should justify the applicability of those tests to the Crystal River Unit 3 masonry structures.

Response:

No test data is available for the masonry wall material. The allowable stresses were obtained from both ACI 531-79 and the Southern Standard Building Code (SSBC), 1969 Edition, (reference 3).

The allowable axial compression stress, as specified in Table 4 of the SSBC, is 70 psi on the gross area. This allowable was used in lieu of the ACI limit of $f_a \leq .225\ f'm\ (1-(h/40t)^3)$, which was derived for load bearing walls and is unreasonable to apply to partition walls. Section 3.0 discusses alternate criteria for allowable compressive stress.

Allowable flexural compression on the net section was based on ACI 531-79 requirements. However, the allowable stress of $.33f'm = 396$ psi was based on an unjustifiably high value of f'm. ACI 531-79 Table 4.3 specifies $f'm = 700$ psi for hollow masonry units comparable to ASTM C 90-66T, Grade G-II with Type N mortar. The allowable flexural stress should therefore be 231 psi. This downgrade has no impact on the evaluation since mortar tension controls wall adequacy.

The allowable tension stresses were based on ACI 531-79 requirements using an appropriate value of $m_o = 750$ psi for Type N mortar. The resulting allowables are $.5\ \sqrt{m_o} = 13.7$ psi for tension perpendicular to bed joint, and 1.0 $\sqrt{m_o}$ = 27.4 psi for tension parallel to bed joint.

Shear stress allowables were not specified since shear was judged not critical. Section 3.0 of this report addresses shear adequacy.

2.12 ITEM 12:

Explain how earthquake motions in three directions are treated in the analysis. Indicate whether walls are subject to in-plane loading.

Response:

The Crystal River Unit #3 FSAR (reference 6), Section 5.2.1.2.9 specifies that seismic design considers a horizontal ground acceleration and a vertical ground acceleration simultaneously. The maximum horizontal ground accelerations acting with arbitrary direction are 0.05g for OBE and 0.10g for SSE. The maximum vertical accelerations are 2/3 of the corresponding horizontal accelerations. The total seismic response is obtained by adding the absolute values of the horizontal and vertical responses. Three directions of motion need not be considered simultaneously.

For masonry walls, the critical horizontal earthquake will cause effects either totally in-plane, or totally out-of-plane. Through-thickness bending controls wall adequacy, and the out-of-plane (or normal) horizontal earthquake was found to be critical in the analysis. Section 3.0 of this report includes results for both an in-plane and out-of-plane analysis.

2.13 ITEM 13:

Provide sample calculations to justify that stresses for in-plane loadings are less critical than for out-of-plane loadings.

Response:

Sample calculations comparing stresses from in-plane and out-of-plane loadings are included in Section 5.0 of this report.

3.0 RE-EVALUATION

3.1 CONTROL COMPLEX WALLS

3.1.1 Reanalysis

3.1.1.1 Models

To achieve conservatism of the Control Complex wall evaluations, modified boundary conditions as described in Section 2.4 were examined. As in the original analysis, the SAP IV computer program (Gilbert Associates program S087) was used to analyze the walls.

Revised models of walls 1 through 5 reflecting the alternate boundary conditions are shown in Figures 14 through 18. The wall elements used were Type 6 - Plate and Shell elements as defined in the SAP IV user's manual with a thickness equivalent to the wall thickness. Grid refinement was varied according to expected stress concentrations. Stiff spring elements were used to impose boundary restraint, allowing reactions to be obtained along clamped edges. A modulus of elasticity of 1000 f'm = 700,000 psi as defined by ACI 531-79 was assumed.

3.1.1.2 Response

The SGEB criteria (reference 4) specifies masonry wall damping values as those given for reinforced concrete in Regulatory Guide 1.61. These values are 4% of critical damping for OBE, and 7% for SSE.

The Control Complex Floor Response Spectra at elevation 145'-0" are shown in Figure 19. These curves are for horizontal OBE with 4% damping. The design curve, shown dashed, envelopes the "true" floor response curves to account for model uncertainty.

For comparison, the broad-band design curve suggested by Regulatory Guide 1.22, part c.2 is shown in Figure 20. Model uncertainty is considered by broadening the peaks of the "true" response curve by ± 15% of the peak frequency, and smoothing the remainder of the curve. The result is quite similar to the design envelope.

The broad-band design curve was used for the analysis of walls 1, 2, and 3. Horizontal OBE response was obtained from Figure 20. Appropriate factors were used to obtain vertical OBE, horizonal SSE, and vertical SSE responses.

Walls 4 and 5 are located at elevation 95'-0". The appropriate design responses are given by the Design Ground Response Spectra for horizontal OBE, shown in Figure 21. Appropriate factors were used to obtain verticl OBE, horizontal SSE, and vertical SSE responses.

3.1.1.3 Load Combinations

The SGEB criteria (reference 4) defines load and load combinations appropriate for masonry wall design. In the presence of only dead and seismic loads, the following load combinations are critical.

Service Load Condition:

D + E

Extreme Environmental, Abnormal, Abnormal/Severe Environmental, Abnormal/Extreme Environmental Conditions:

D + E'

3.1.1.4 Allowable Stresses

The following working stress allowables are specified by ACI 531-79 for stress on net section:

Axial Compression	$0.225\ f'm\ (1-(h/40t)3)$
Tension perpendicular to bed joint	$0.5\ \sqrt{m_o} = 13.7$ psi
Tension parallel to bed joint	$1.0\ \sqrt{m_o} = 27.4$ psi
Flexural Compression	$.33f'm = 231$ psi
Shear in Flexural Members	$1.1\ \sqrt{f'm} = 29.1$ psi
Shear in Shearwalls	
$M/Vd_v \geq 1$	$0.9\ \sqrt{f'm} = 23.8$ psi
$M/Vd_v < 1$	$2.0\ \sqrt{f'm} = 52.9$ psi $\leq$ 40 psi (1.85-M/Vdv)

The ACI 531-79 allowable for axial compressive stress is based on load bearing masonry. This allowable is too restrictive for non-load bearing masonry walls.

A factor of safety of 9 applied to the theoretical buckling strength of the critical Control Complex wall (modelled as a vertical cantilever under uniformly distributed axial load) gives an allowable axial compressive stress of 90 psi at the wall base. An allowable of .22f'm = 154 psi is recommended by reference 5, Section 2 for walls without significant vertical load. A 70 psi compressive allowable on gross sections is recommended by the Southern Standard Building Code (reference 8), the code which has been used in past evaluations.

3.1.2 <u>Adequacy</u>

3.1.2.1 Wall 1

The ACI minimum thickness requirement is exceeded by forty percent. The ACI allowable compressive formula becomes negative for the h/t ratio determined. However, axial compression is insignificant compared to the high bending stresses developed. Flexure along the bottom west side (see Figure 4) exceeds allowables by at least eight times. Bending near the corner of the door where wall 2 intersects is more severe. Shear presents no problem in this wall.

3.1.2.2 Wall 2

Bending stresses on the north side (see Figure 5) near the vertical joint are fifty percent over the ACI limit. Bending stresses near the top south corner are fifteen percent over the allowable for an SSE condition but are adequate for the OBE case. Shear stresses are adequate at all boundaries.

3.1.2.3 Wall 3

The ACI minimum wall thickness criteria is exceeded by forty percent. The ACI allowable compressive formula becomes negative for the computed h/t ratio. Bending along the bottom east side exceeds ACI allowables by sixty percent for OBE and much more for SSE. Shear along the bottom is within limits. Both shear and bending are grossly exceeded in the upper west portion of the wall.

3.1.2.4 Wall 4

The ACI minimum thickness criteria is exceeded by thirty percent. The ACI allowable compressive formula provides negative results. However, by using a 70 psi allowable in compression (see 3.1.1.4), stresses along the base are within acceptable limits.

Relaxing side boundary conditions to partial fixity to account for moment redistribution through continuous adjacent wall panels will maintain stresses within allowables as mid-panel stresses are presently less than one-third the allowable. Stresses along the bottom are also about one-third the allowable if the 70 psi compressive limit is used. Shear along the edges is low.

3.1.2.5 Wall 5

The ACI minimum thickness requirement is exceeded by forty percent. The ACI allowable compressive formula is not applicable so the Southern Standard Building Code allowable of 70 psi is utilized (conservatively applied on the net section). With this criteria stresses along the base are within limits. Flexural stresses in the upper east region (see Figure 8) are about ten percent over for OBE and twenty percent over for SSE. However, complete fixity along the sides as modelled in the finite element program will not occur. Relaxing the boundary conditions will provide acceptable stresses in this region. Results at all other locations are within ACI limits. Shear is adequate at all edge locations.

3.1.2.6 In-Plane Seismic Considerations

Adequacy of the five walls discused in 3.1.2.1 through 3.1.2.5 is based on results of an out-of-plane horizontal seismic event occurring simultaneously with a vertical earthquake. To verify that this produces the most severe conditions, models of walls 2, 3, and 4 were checked with an in-plane horizontal seismic loading. Results indicate that in-plane effects are much less severe than those caused by out-of-plane conditions. In-plane loads are resisted by in-plane tension or compression and produce small stresses whereas out-of-plane loads produce high through-thickness bending moments and result in high flexural stresses.

3.1.3 Interstory Drift Effects

Reference 5 provides in-plane permissible strain criteria for confined and unconfined masonry walls. Section 2.5 of this report summarizes recommended strain limits. The acceptance criteria in reference 5 is based on an uncoupled system (separate behavior of in-plane and out-of-plane deflections). This agrees with the one-directional horizontal plus vertical seismic requirement of Crystal River Unit #3. Results of all five walls indicate maximum strains due to interstory drift are less than one-half the unconfined wall allowable provided in the reference.

Out-of-plane drift effects are calculated by applying the maximum relative story deflection to the top of a cantilever wall. The flexural stresses produced at the base for all five Control Complex walls are less than one-half the ACI allowable.

3.2 Turbine Building Walls

The Turbine Building Instrument Room was evaluated using a single model having three sides which were pinned along the base. Figures 22 and 23 show the finite element representation of this structure. The concrete roof slab was not modelled, but its mass was distributed along the top of the walls. Diaphragm action of the slab was approximated by pinning the top edges in the two horizontal directions. The side edges of the walls were also restrained against horizontal translation, except where they abutted steel. Free lateral movement was provided at the steel column.

Results of the analysis show maximum flexure occurs near mid-height in the element strip adjacent to the door. However, the maximum bending stress induced is only twenty-five percent of the compressive stress in this wall at this location. Based on ACI and NRC criteria, the combined stresses are also less than twenty-five percent of the allowable. Due to the inherent strength of the shearwall system, shear stresses are also small. The walls are, therefore, adequate for the imposed seismic loads.

The seismic analysis for the Instrument Room walls provided data using the first mode of vibration (36 cycles/sec.). Effects of higher modes were considered by increasing first mode results by five percent. Further seismic investigation on these walls is unnecessary because of the low stress condition that exists. (Dead load compression is considerably higher than seismic flexure.)

The Turbine Building air shaft walls are not evaluated in this report. Reference 7 states that their seismic adequacy is no longer a concern since the motor control center located near the air shaft has no safety related function.

4.0 CONCLUSIONS

Based on the analysis described in this report walls 4 and 5 in the Control Complex are adequate for the seismic conditions specified in reference 6 with the following modifications. The ACI minimum thickness requirement is not maintained, and the Southern Standard Building Code compression allowable is conservatively implemented. Control Complex walls 1, 2, and 3 do not meet the ACI and NRC criteria. Modifications implemented in wall 1 as a result of the initial IE 80-11 investigation eliminate potential safety concerns resulting from the postulated failure of the wall. The Turbine Building Instrument Control Room walls meet the updated criteria without any ACI limitations. The Turbine Building air shaft walls do not require evaluation based on subsequent considerations in system protection philosophy.

5.0 SAMPLE CALCULATIONS

5.1 MULTIPLE MODE FACTOR

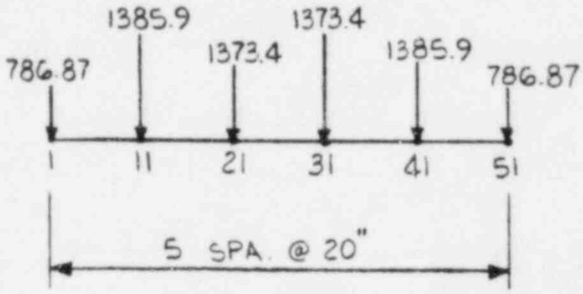
Comparison of moments and shears at governing wall regions for selected numbers of modes in the revised models are tabulated below. Loads are taken from the SAP IV computer output and are based on out-of-plane horizontal SSE conditions, except where noted.

WALL / NODE (ELEMENT)	LOAD	1 MODE	8 MODES	12 MODES	8/1 RATIO
1/9	M	25,180.	26,440	26,440	1.05+
1/9	V	348.6	406.3	406.8	1.17+
1/120	M	2951	35550	35550	12.05
1/120	V	5652	7281	7281	1.29
1/(128)	M/in	2900	3506	3506	1.21
2/29*	M	914.3	972.9	-	1.06
2/29*	V	32.29	36.08	-	1.12
2/54*	M	5451	5459	-	1.00 +
2/54*	V	330.2	332.8	-	1.01 +
2/(24)*	M/in	71.96	72.22	-	1.00
3/7	M	60250	60260	-	1.00 +
3/7	V	2535	2545	-	1.00 +
3/51	M	7375	7500	-	1.02
3/51	V	399.8	406.4	-	1.02
4/1	M	845.2	864.7	-	1.02 +
4/1	V	38.24	41.81	-	1.09 +
4/57	M	204.4	435	-	2.13
4/66	V	9.8	22	-	2.25
4/(50)	Mxx/in	21.57	23.98	-	1.11
4/(50)	Myy/in	23.31	23.61	-	1.01
5/57	M	2460	2520	-	1.02
5/57	V	55.01	59.08	-	1.07
5/108	M	3442	3458	-	1.01 +
5/108	V	209.5	211.6	-	1.01 +

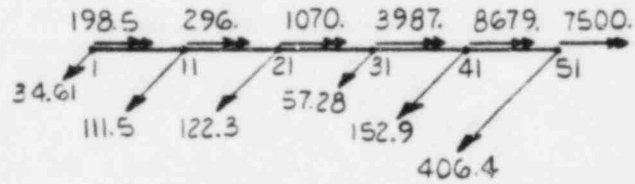
* results for out-of-plane horizontal OBE
\+ highest stressed region

5.2 WALL STRESS

This example provides calculations for the governing SSE conditions at the base of wall 3 using the out-of-plane horizontal SSE in combination with vertical. The dead load is based on a 50 lb. weight of 8 inch block per square foot of wall surface. Surface finish weight is negligible.



Dead Load



SSE
(Vertical reactions negligible)

(units in pounds or inch-pounds)

Computer runs for vertical seismic loads produce negligible reactions since no vertical mode predominates in the eight modes obtained. Therefore, 2/3 of the horizontal ZPA will be used for the vertical acceleration.

Reactions at node 51 reflect peak conditions at the corner. (Node 51 represents total load over a 10 inch strip whereas node 41 provides reactions over a 20 inch section.) Therefore, an average load acting over the area represented by nodes 41 and 51 is used.

Block properties: $S = 80.0\ in^3/ft \times 30/12 = 200.0\ in^3$
$A_{net} = 45.0\ in^2/ft \times 30/12 = 112.5\ in^2$

Total axial load $= [1385.9 + 786.87]\,[1 \pm 2/3(0.14)]$
$= 2375.6$ lb or 1970.0 lb (over 30" length)

Total moment $= 8679 + 7500 = 16179$ in-lb

Total shear $= 152.9 + 406.4 = 559.3$ lb

$f_a = 2375.6 / 112.5 = 21.1$ psi or $f_a = 1970.0 / 112.5 = 17.5$ psi

$f_m = 16179 / 200.0 = 80.9$ psi

From ACI 531-79, 11.3.1,

$t_{min.} = 1/36$ (distance between lateral supports)
$= 1/36\,(2 \times 204) = 11.3\ in > 8\ in.$ N.G.

(2 represents the effective height of a cantiever as required by ACI 9.4.8.2)

$F_a = 0.225(700)\,[1 - ((2 \times 204)/(40 \times 8))^3] < 0$ N.G.   (ACI,10.1.3)

Use Fa = 70.0 psi

F_m = 231 psi for compressive flexure

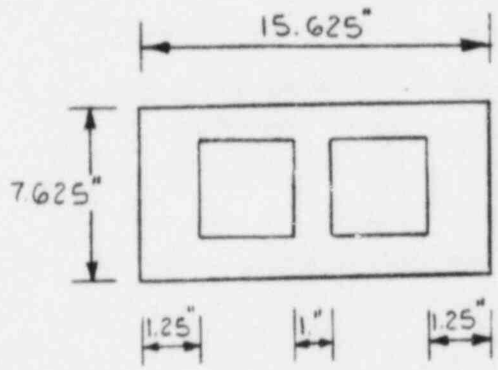
F_m = 13.7 psi for tension normal to bed joints

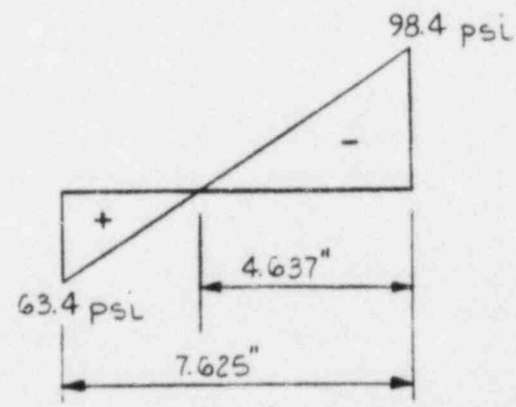
From reference 4, SSE factors are 2.5 for axial or flexural compression and 1.3 for tension normal to the bed joint.

Using ACI 11.1.1, $[21.1/(2.5 \times 70.0)] + [80.9/(2.5 \times 231)] = 0.26 < 1.0$ O.K.

$[80.9 - 17.5] / [1.3(13.7)] = 3.56 > 1.0$ N.G.

Shear check:





Effective shear area = $(1.25 + 1.0 + 1.25)(4.637) = 16.23$ in^2

$v = [559.3(16/30)] / [1.3(16.23)] = 14.1$ psi

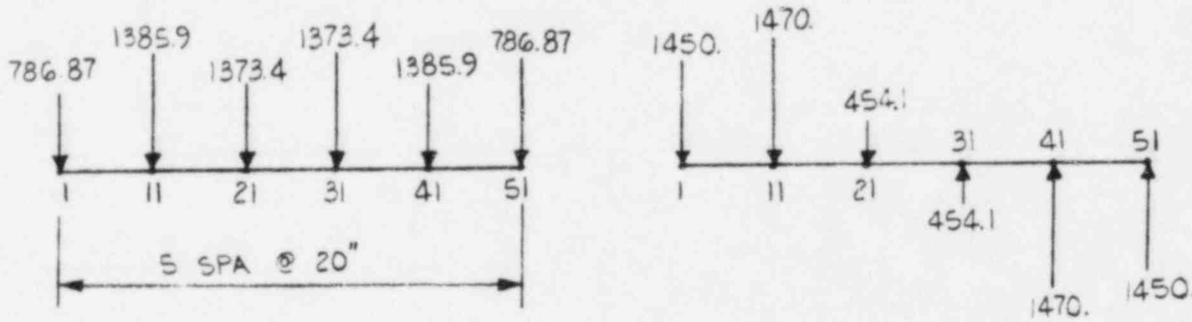
where 1.3 is the shear factor from reference 4

$v_m = 1.1\sqrt{f_m} = 29.1$ psi   (ACI, Table 10.0 - wall is behaving as a flexural member for this situation)

$14.1 < 29.1$ $\therefore$ O.K. in shear

5.3 IN-PLANE STRESS VS. OUT-OF-PLANE STRESS

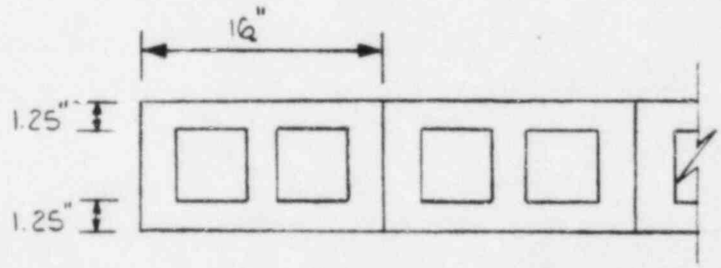
In-plane horizontal SSE conditions were run using the SAP IV computer program for Control Complex walls 2,3, and 4. Only wall 3 provided a situation where seismic uplift exceeded wall dead load. Stress calculations for wall 3 in-plane stresses using reactions at bottom nodes follow.



Dead Load SSE

(Units in Pounds)

$\Sigma M_{\mathcal{L}} = 2[1450(50) + 1470(30) + 454.1(10)] = 242{,}282$ in-lb.



$A_{net} = 45.0\ in^2/block$

$I \approx 1/12[(2.5)(100)^3] + [45.0(100/16) - 2.5(100)](25)^2 = 227{,}864\ in^4$

$f_m = 242{,}282(50)/227{,}864 = 53.2$ psi

$f_{a_{D.L.}} = 2[786.87 + 1385.9 + 1373.4] / [100/16(45.0)] = 25.2$ psi

For vertical SSE acceleration use 2/3 of the horizontal ZPA.

$f_{a_{D.L.-SSE}} = 25.2[1 - 2/3(0.14)] = 22.8$ psi

$f_a/F_a + f_m/F_m \leq 1.0$ (ACI,11.1.1) $f_m > f_a$ $\therefore$ Use F_m allowable

F_m = 13.7 psi For masonry tension normal to bed joint, use SRP factor of 1.3.

[53.2 - 22.8] / [1.3(13.7)] = 1.71 > 1.0 N.G.

For this wall subjected to an out-of-plane SSE, the above factor along the base is 3.56 (see Section 5.2), proving the out-of-plane condition is more severe. Of the three walls checked, only this wall failed to meet the in-plane criteria.

6.0 REFERENCES

1. Report on Masonry Wall Re-Evaluation Program, Crystal River Unit #3, Gilbert Associates, Inc., Nov. 6, 1980.

2. J.F. Stolz, Chief, Operating Reactors Branch #4, Division of Licensing (NRC)
Letter to W.S. Wilgus, Vice President, Nuclear Operations (FPC)
Subject: Request for Additional Information - Crystal River Unit #3, Masonry Wall Design, IE Bulletin 80-11
Nov. 29, 1983 Docket No. 50-302

3. Building Code Requirements for Concrete Masonry Structures (ACI 531-79), American Concrete Institute Committee 531, 1978.

4. Interim Criteria for Safety-Related Masonry Wall Evaluation, Appendix A to SRP Section 3.8.4, USNRC, Rev. 0, July 1981.

5. Recommended Guidelines for the Reassessment of Safety-Related Concrete Masonry Walls, prepared by Owners and Engineering Firms Informal Group on Concrete Masonry Wals, October 6, 1980.

6. Crystal River Unit #3 FSAR, Docket No. 50-302, Operating License No. DFR-72.

7. David Mardis, Acting Manager, Nuclear Licensing (FPC)
Letter to J.P. O'Reilly, Regional Administrator, Office of Inspection & Enforcement (NRC)
Subject: Crystal River Unit #3, Masonry Wall Design, IE Bulletin 80-11
March 23, 1982 #3F-0382-14

8. Southern Standard Building Code, by the Southern Building Code Congress, 1969 Edition.

7.0 FIGURES

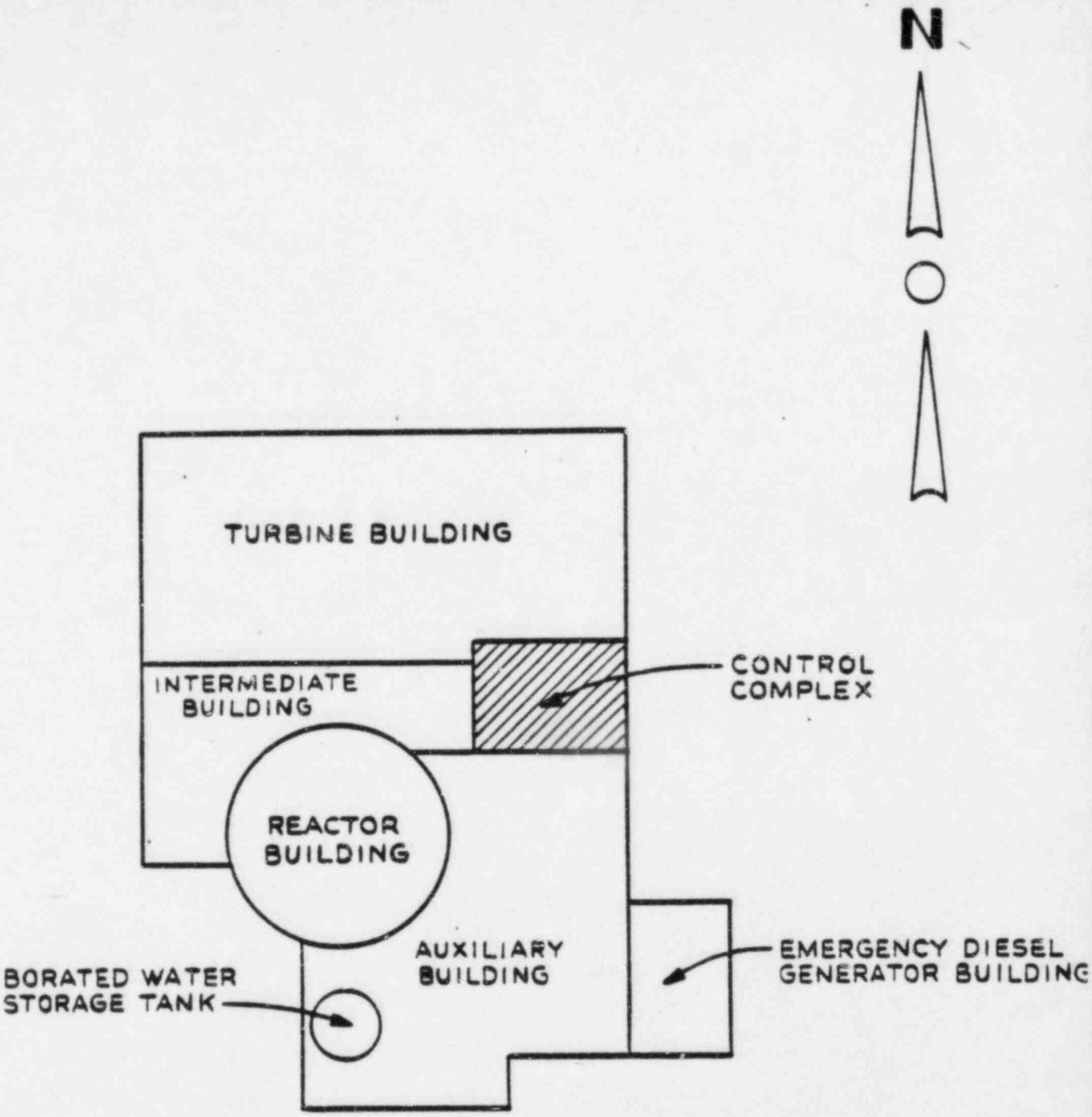
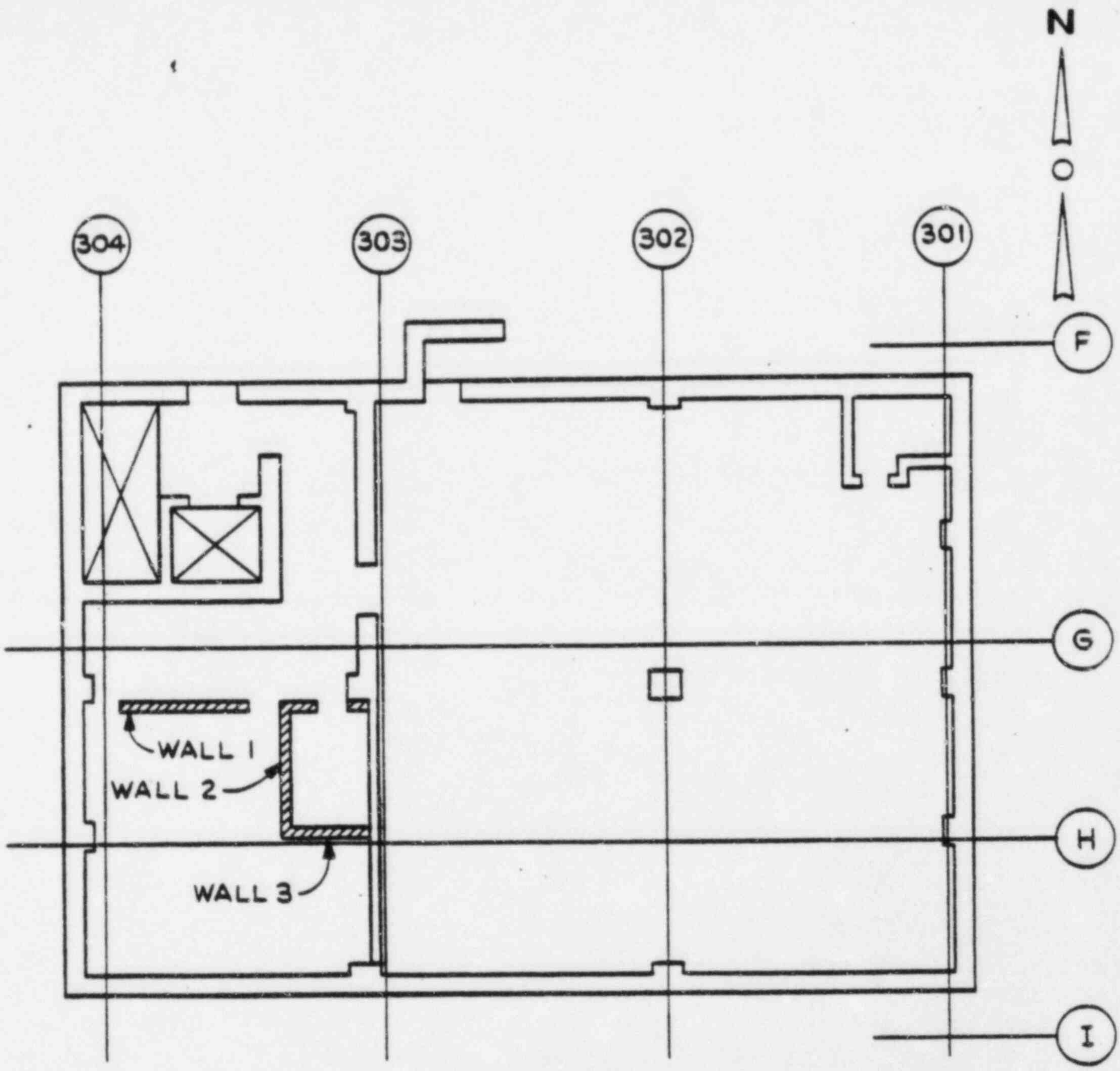
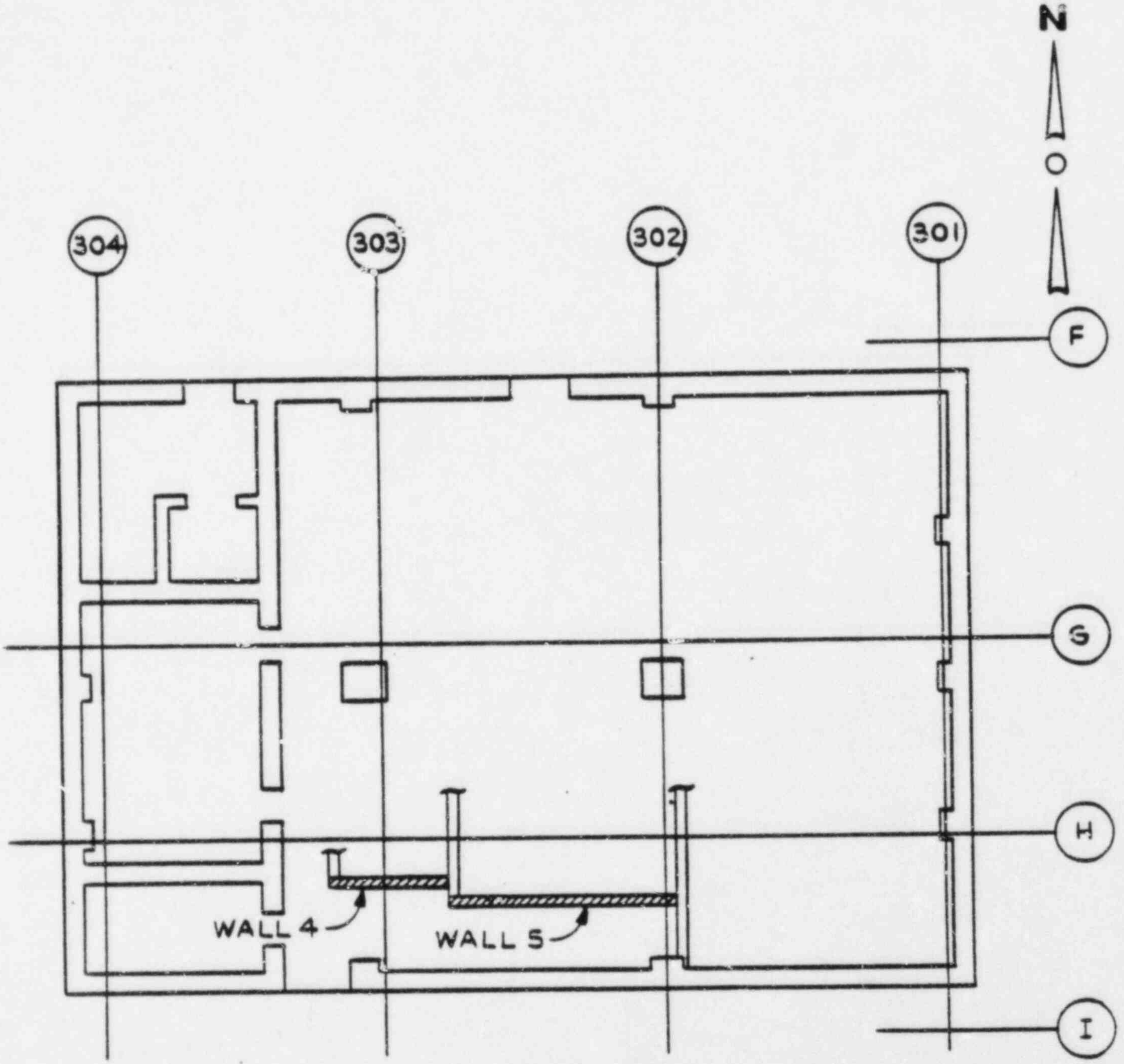

N
TURBINE BUILDING
INTERMEDIATE BUILDING
CONTROL COMPLEX
REACTOR BUILDING
AUXILIARY BUILDING
BORATED WATER STORAGE TANK
EMERGENCY DIESEL GENERATOR BUILDING

FIGURE 1



CONTROL COMPLEX
PLAN - EL 145'-0"

FIGURE 2



CONTROL COMPLEX
PLAN-EL 95'-0"

FIGURE 3

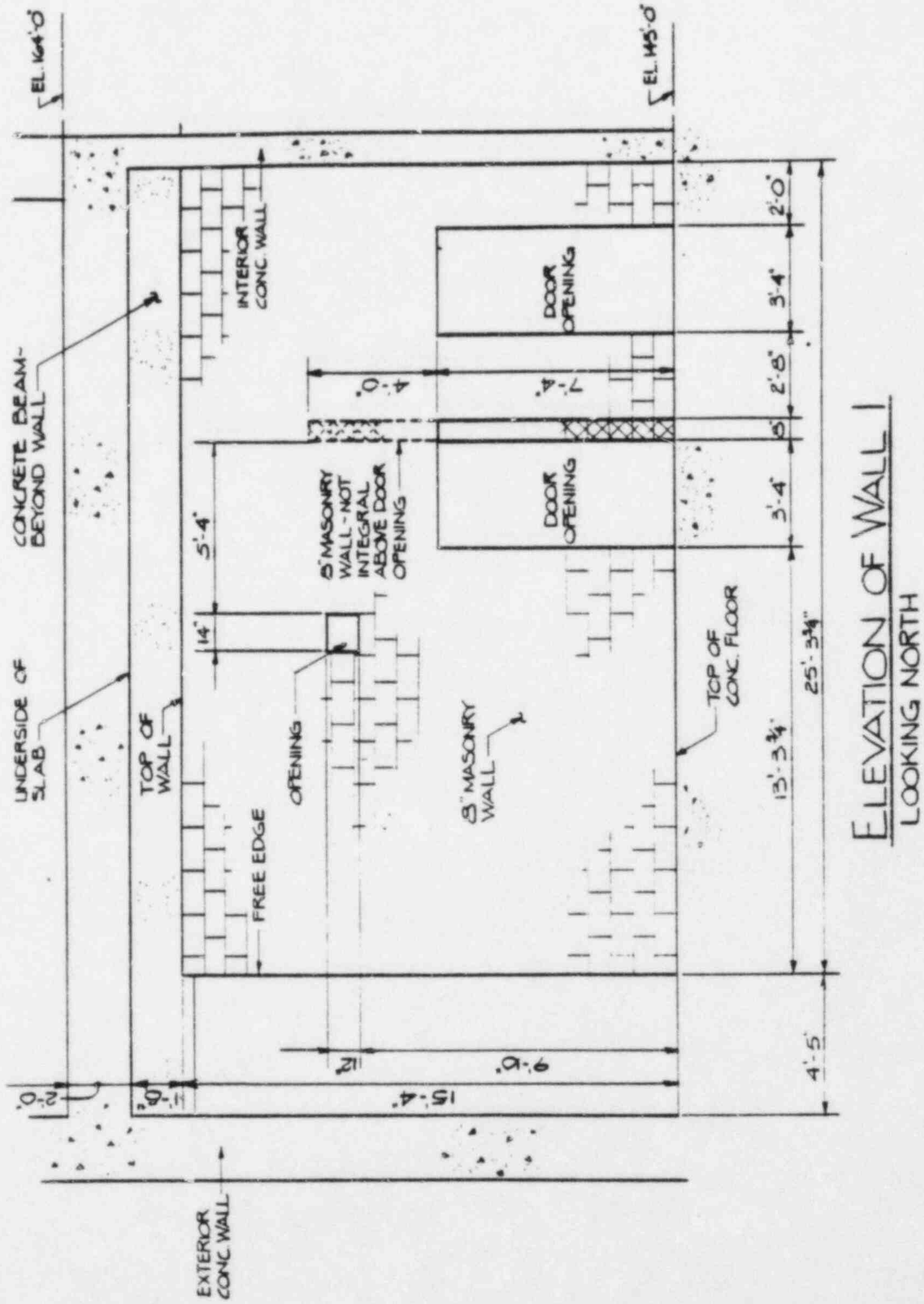
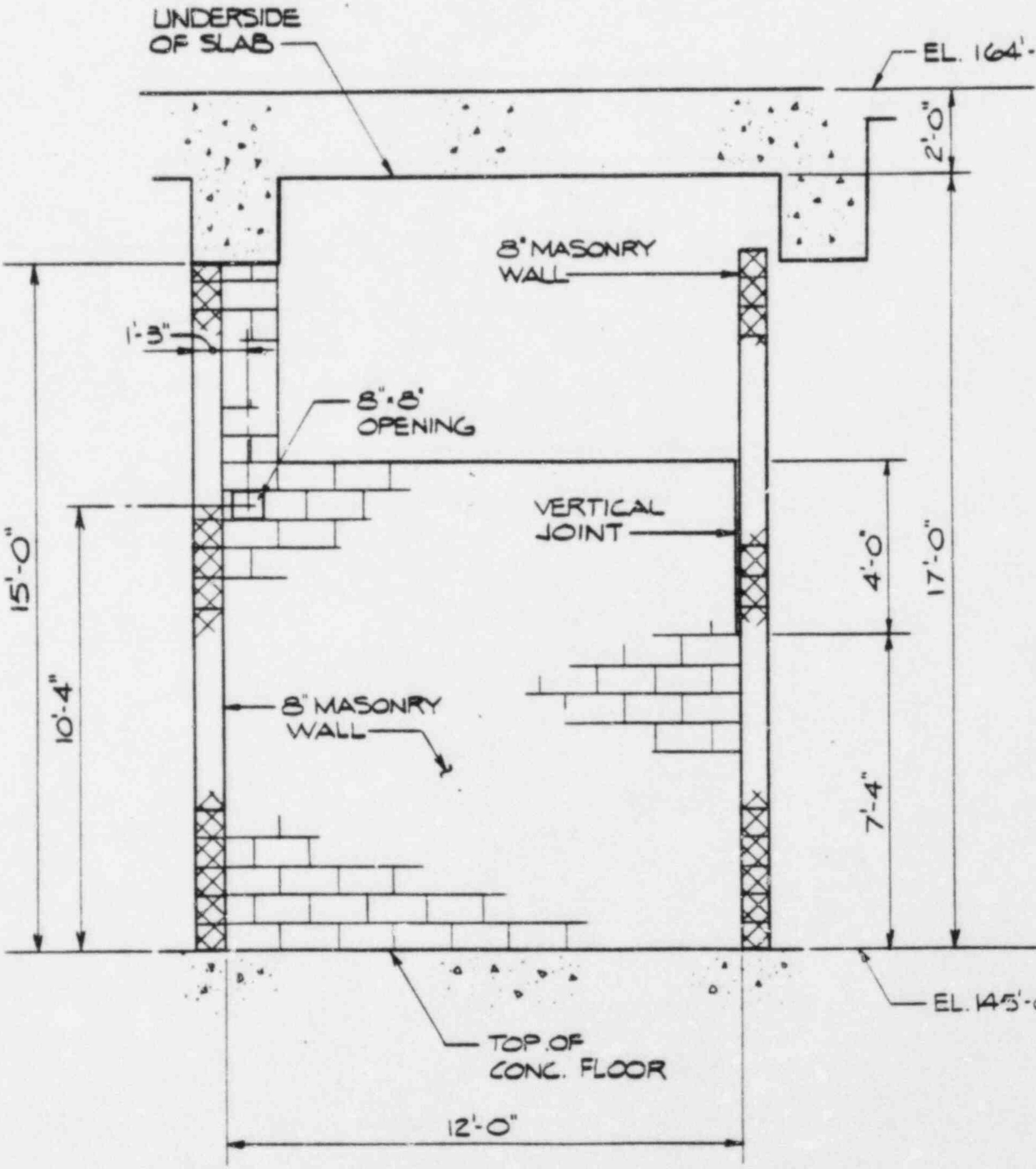
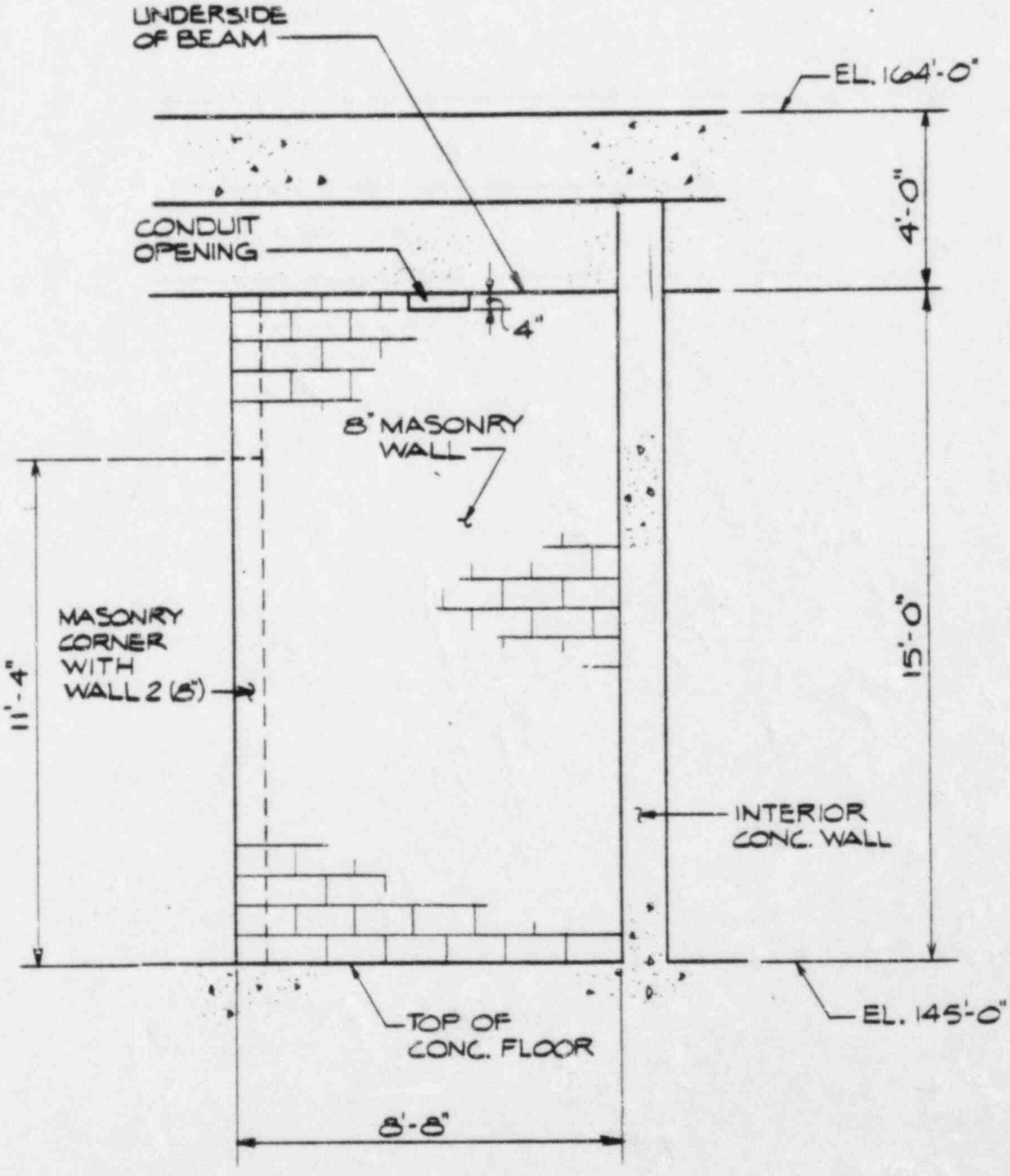

ELEVATION OF WALL 1
LOOKING NORTH
EL. 164'-0"
EL. 145'-0"
CONCRETE BEAM-BEYOND WALL
UNDERSIDE OF SLAB
TOP OF WALL
INTERIOR CONC. WALL
DOOR OPENING
DOOR OPENING
8" MASONRY WALL-NOT INTEGRAL ABOVE DOOR OPENING
OPENING
8" MASONRY WALL
TOP OF CONC. FLOOR
FREE EDGE
EXTERIOR CONC. WALL

FIGURE 4



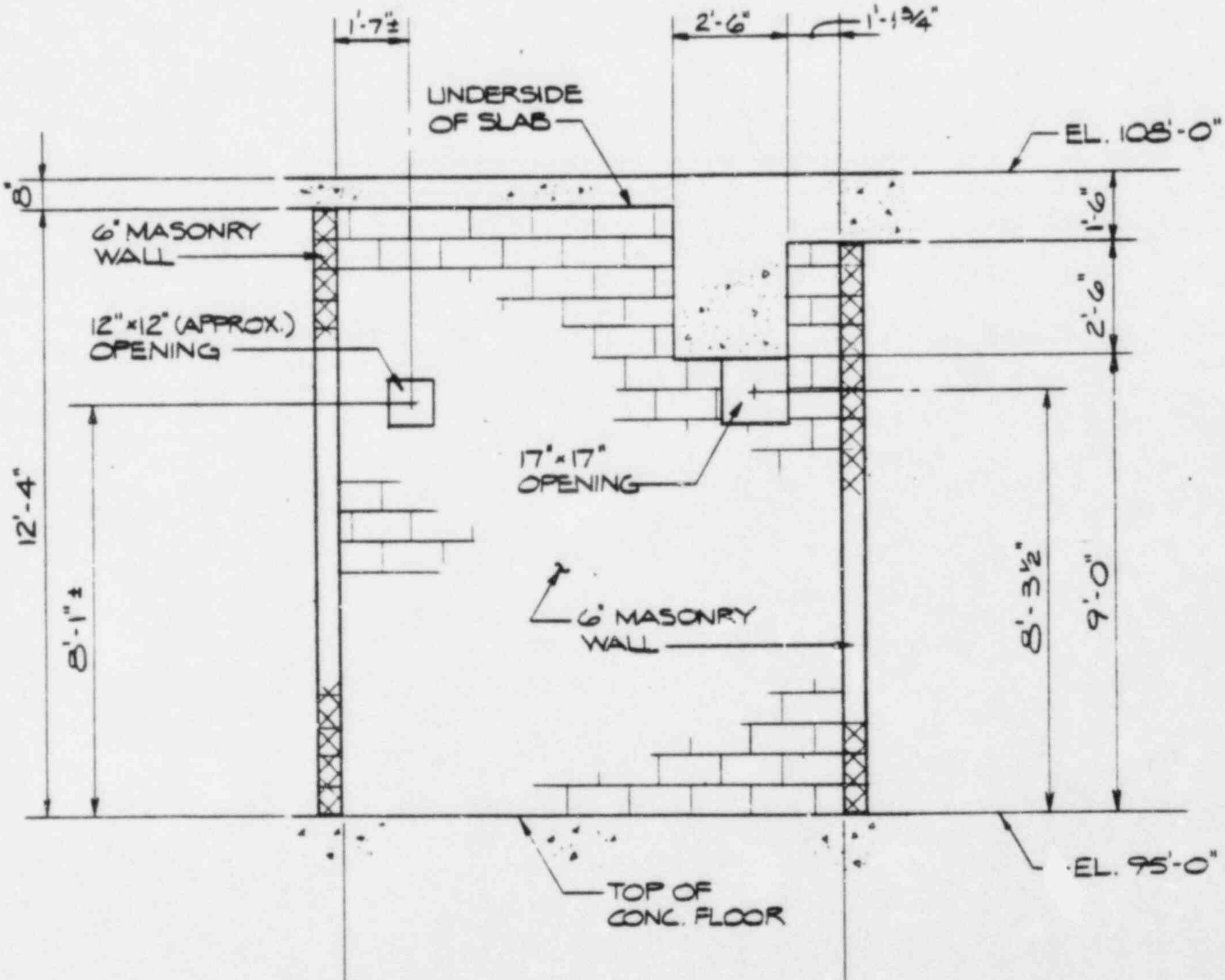
ELEVATION OF WALL 2
LOOKING WEST

FIGURE 5



ELEVATION OF WALL 3
LOOKING NORTH

FIGURE 6



ELEVATION OF WALL 4
LOOKING SOUTH

FIGURE 7

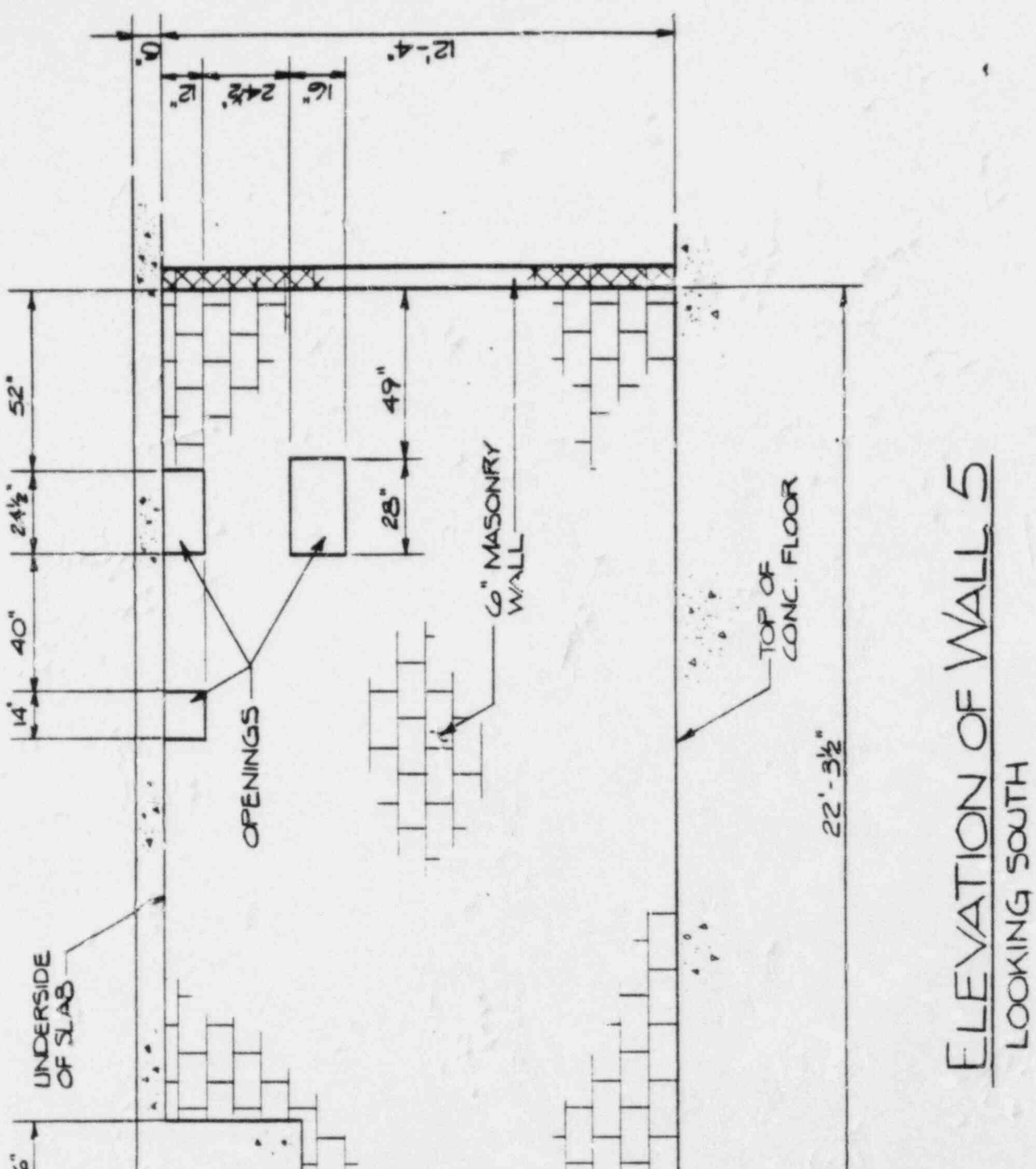
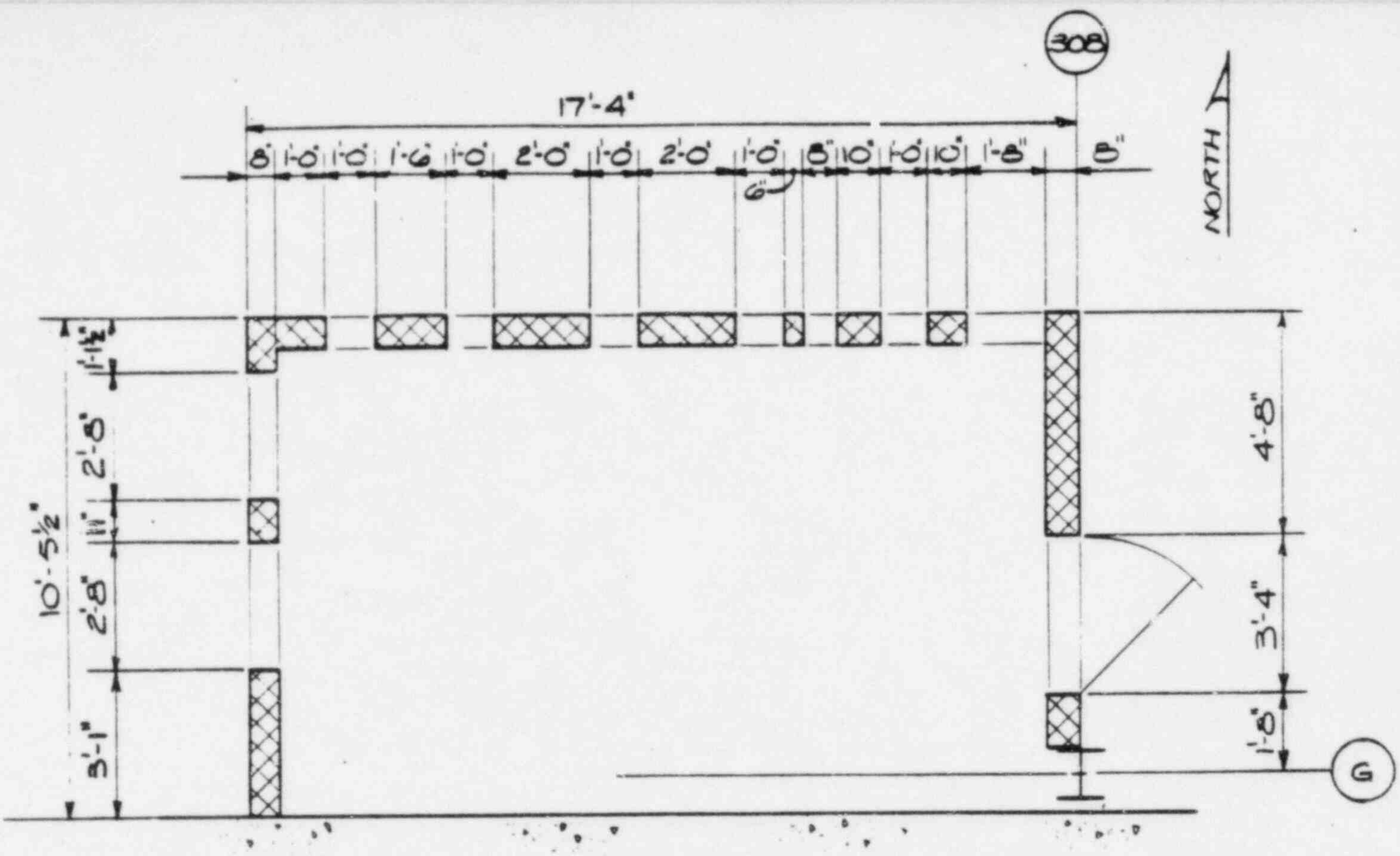
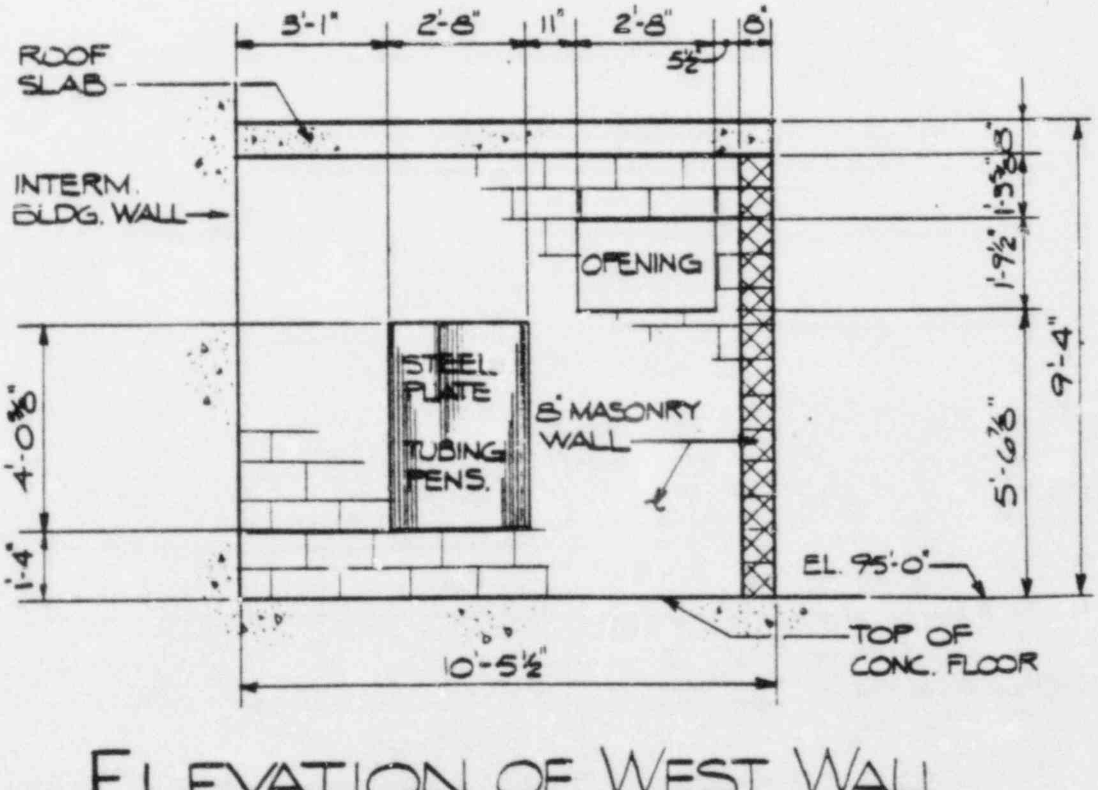

ELEVATION OF WALL 5
LOOKING SOUTH
UNDERSIDE OF SLAB
OPENINGS
6" MASONRY WALL
6" MASONRY WALL
TOP OF CONC. FLOOR
12'-4"
10"
12"
24½"
16"
52"
24½"
40"
14"
2'-6"
5"
49"
28"
22'-3½"
4'-0"

FIGURE 8



INSTRUMENT ROOM PLAN

TURBINE BLDG. - ELEVATION 95'-0"

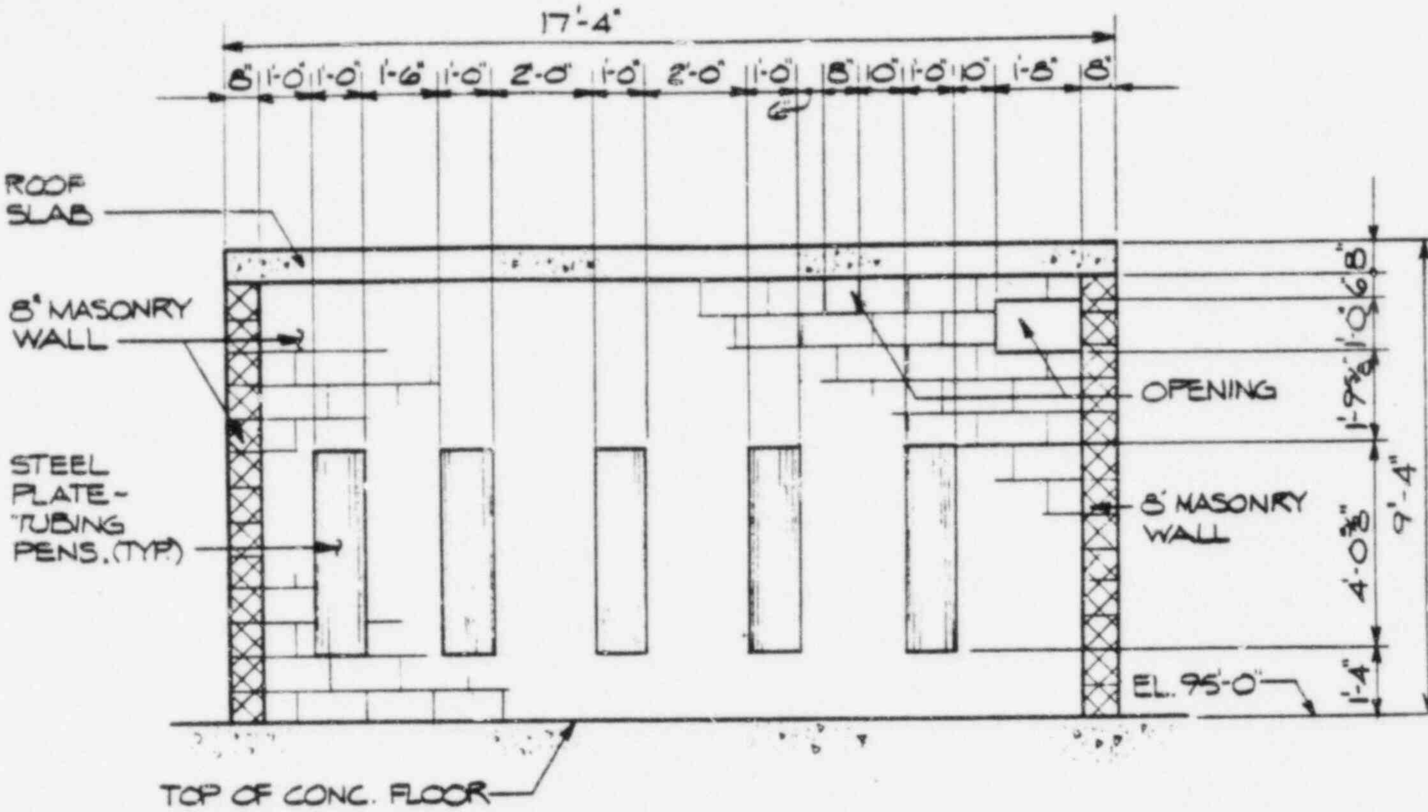


ELEVATION OF WEST WALL

LOOKING WEST

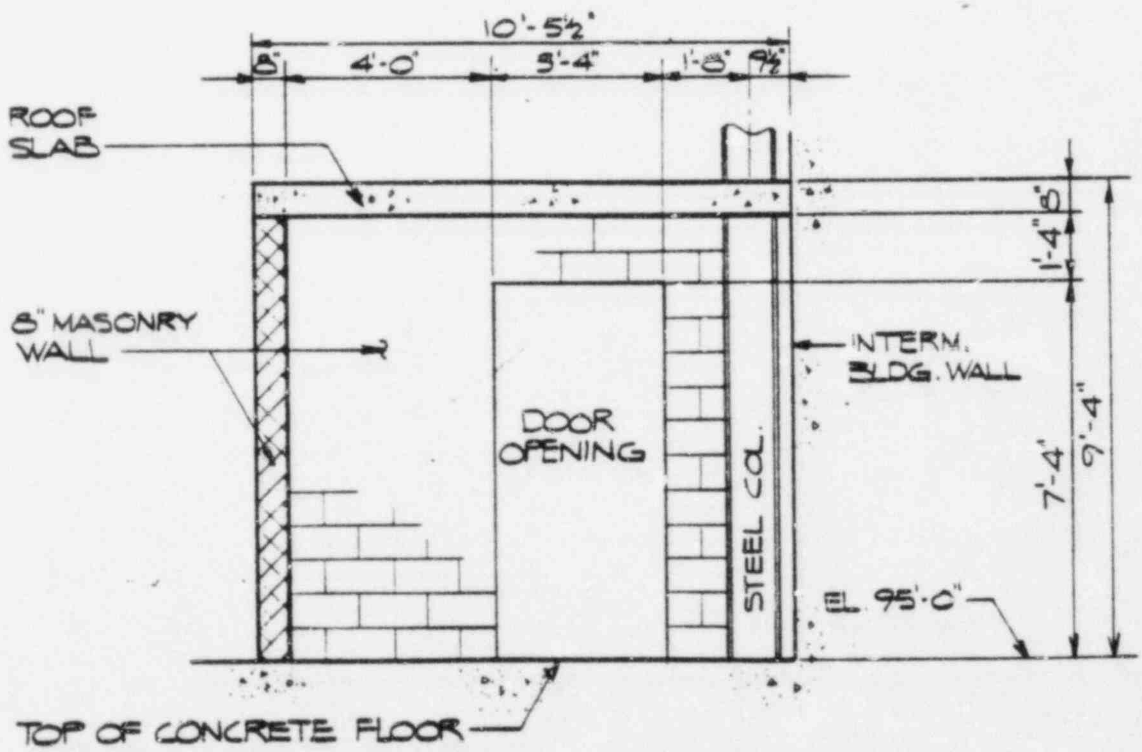
FIGURE 9

Gilbert / Commonwealth



ELEVATION OF NORTH WALL

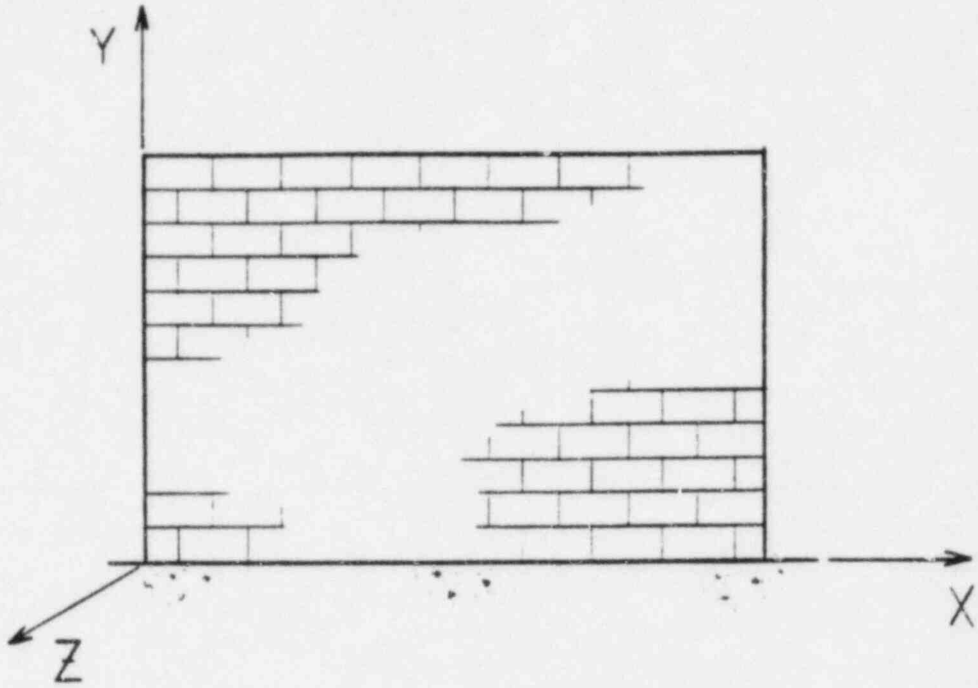
LOOKING NORTH



ELEVATION OF EAST WALL

LOOKING EAST

FIGURE 10



TYPICAL WALL ELEVATION

Notation:

Δx = Translation in x direction

Δy = Translation in y direction

Δz = Translation in z direction - Out-of-Plane

Θx = Rotation about x axis

Θy = Rotation about y axis

Θz = Rotation about z axis

FIGURE 11

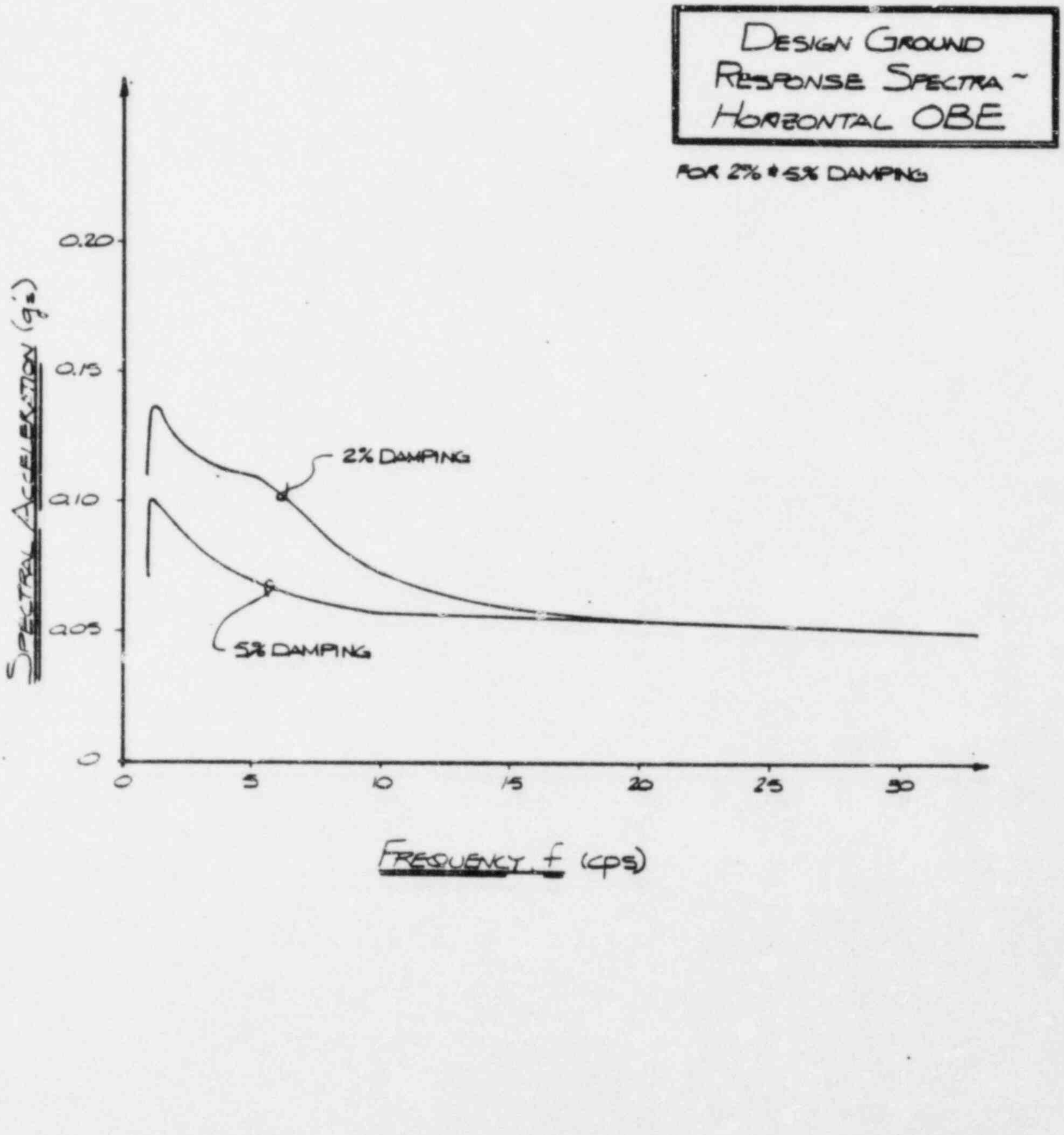

DESIGN GROUND RESPONSE SPECTRA ~ HORIZONTAL OBE
FOR 2% & 5% DAMPING
SPECTRAL ACCELERATION (g's)
0.20
0.15
0.10
0.05
0
2% DAMPING
5% DAMPING
0
5
10
15
20
25
30
FREQUENCY, f (cps)

FIGURE 12

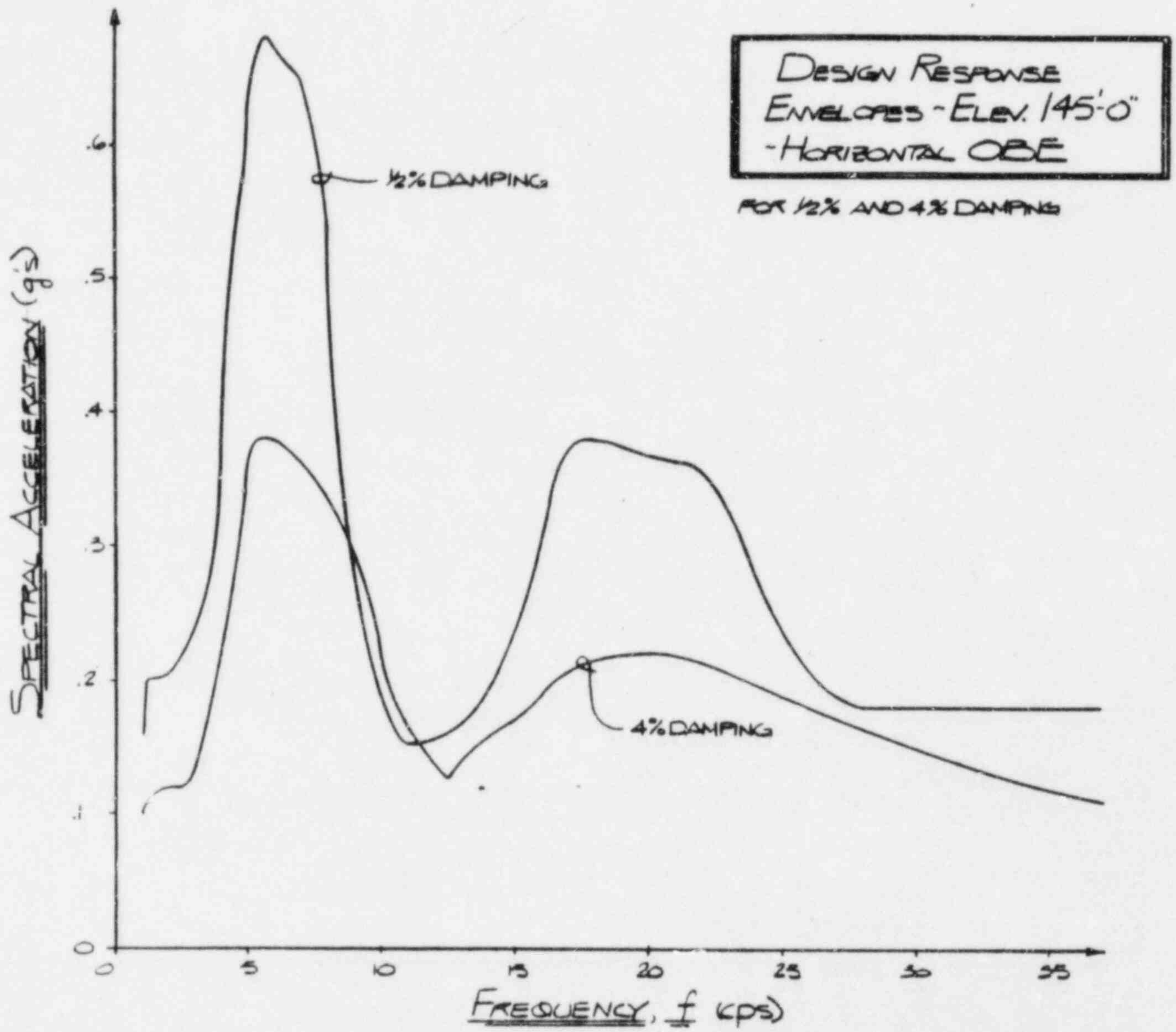
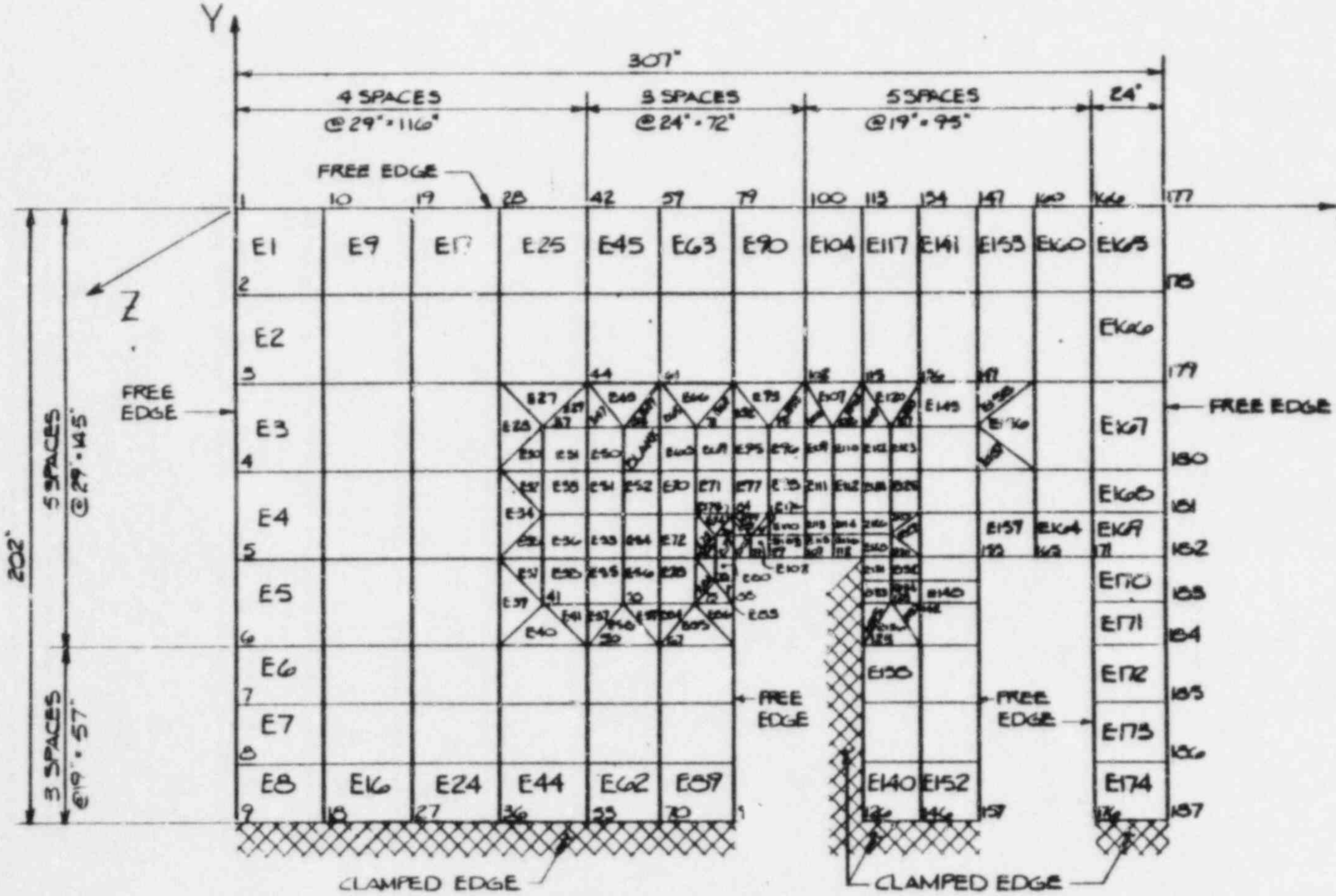

DESIGN RESPONSE
ENVELOPES - ELEV. 145'-0"
- HORIZONTAL OBE
FOR 1/2% AND 4% DAMPING
1/2% DAMPING
4% DAMPING
SPECTRAL ACCELERATION (g's)
FREQUENCY, f (cps)
0
.1
.2
.3
.4
.5
.6
0
5
10
15
20
25
30
35

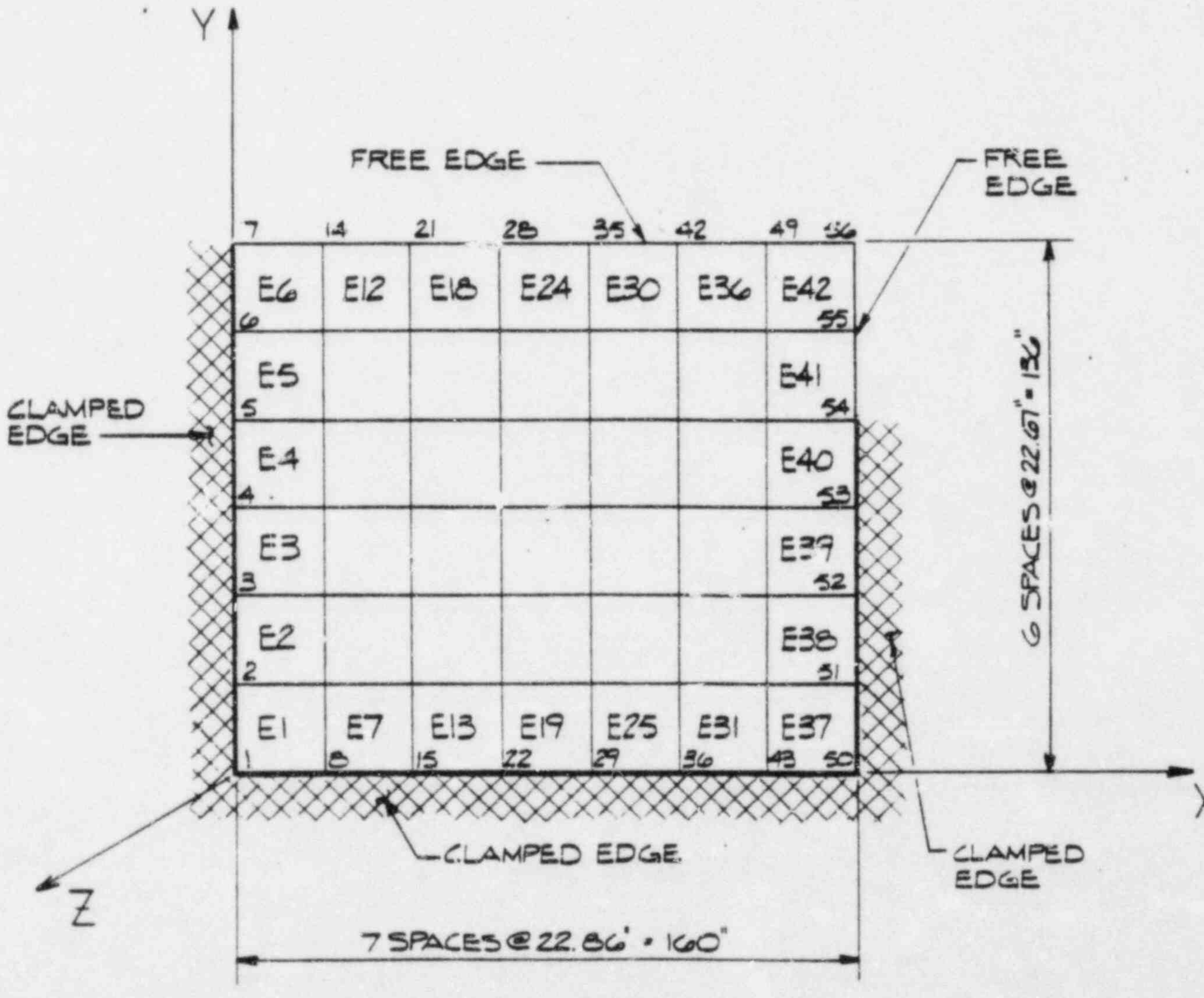
FIGURE 13



WALL I MODEL

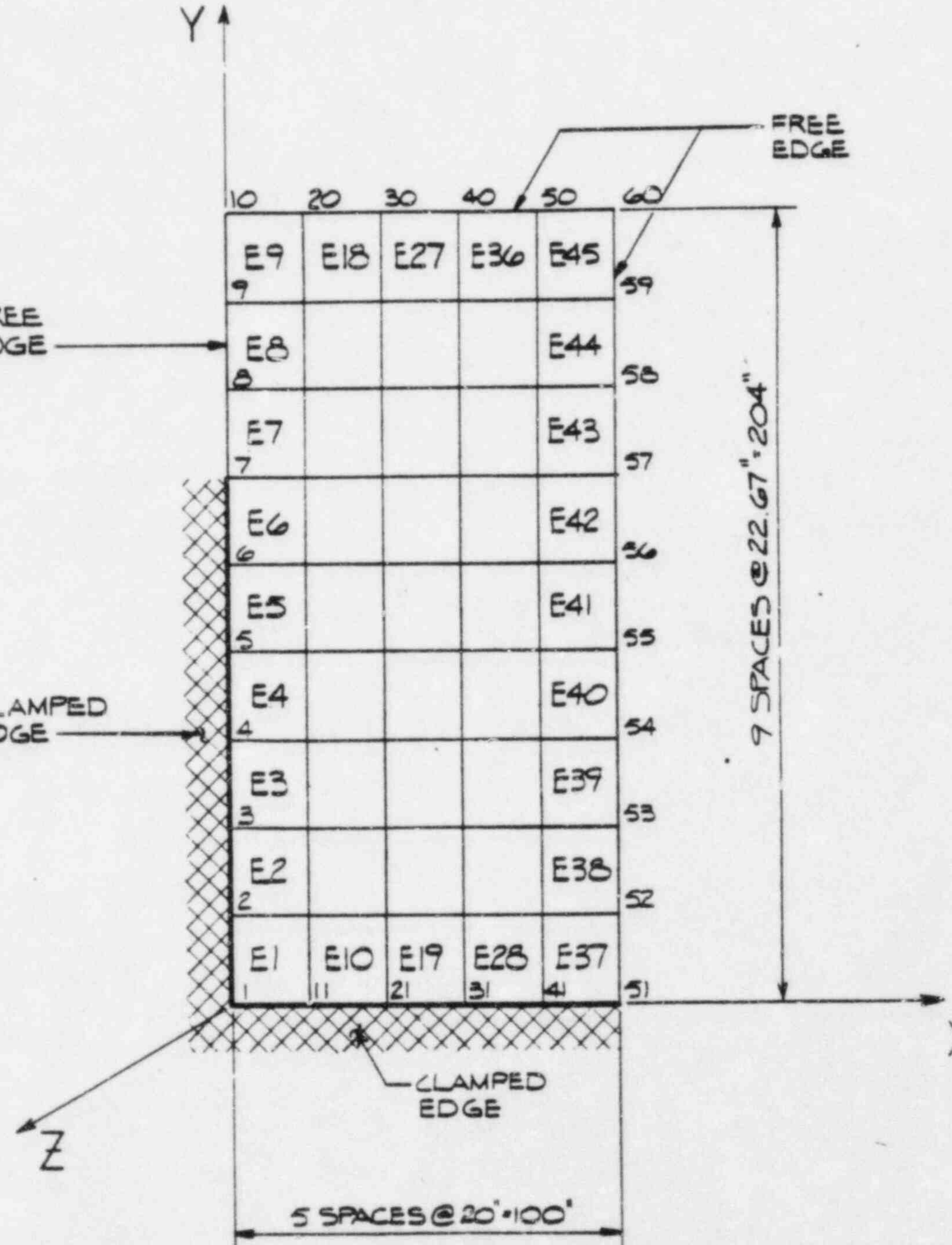
FIGURE 14

Gilbert/Commonwealth



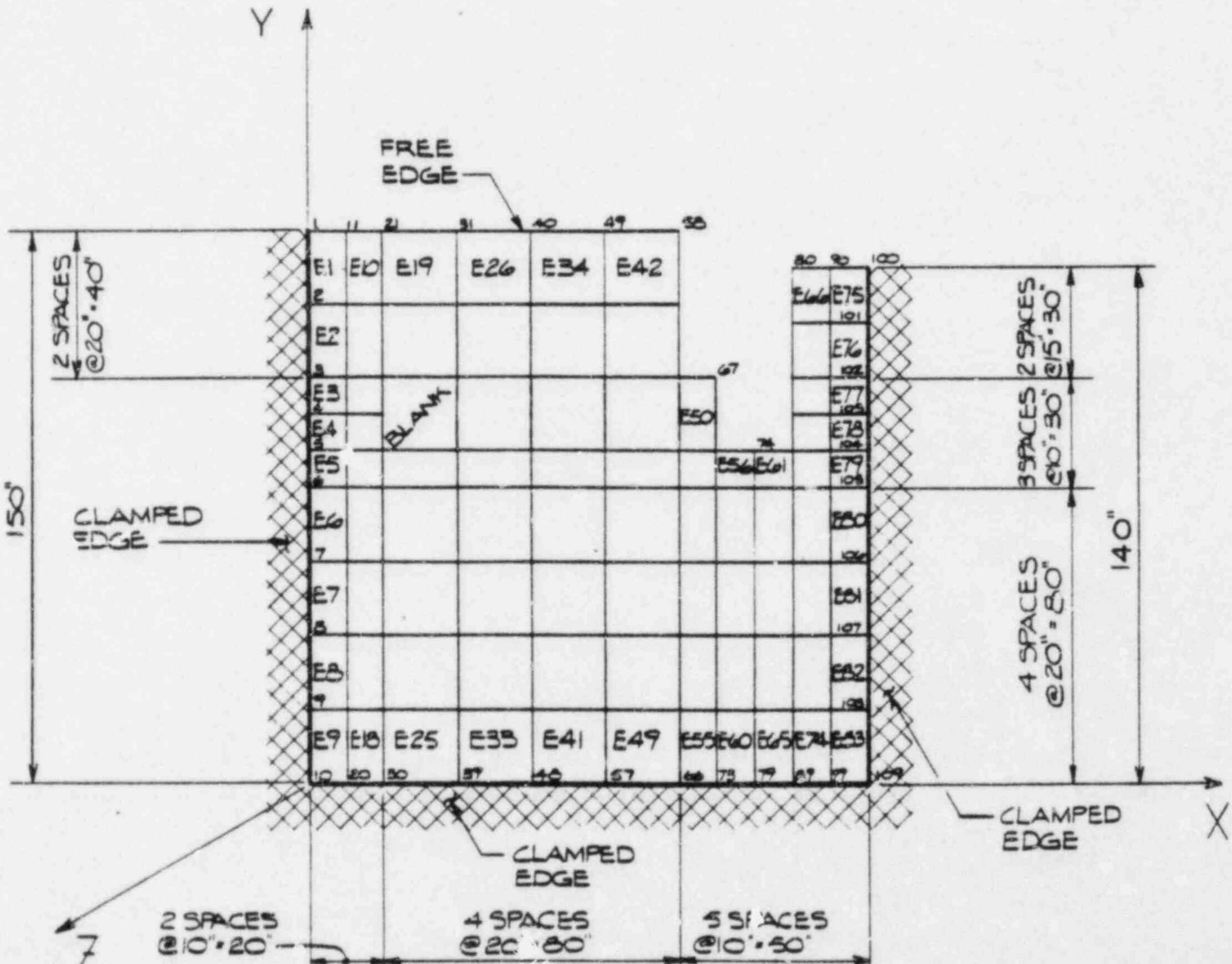
WALL 2 MODEL

FIGURE 15



WALL 3 MODEL

FIGURE 16

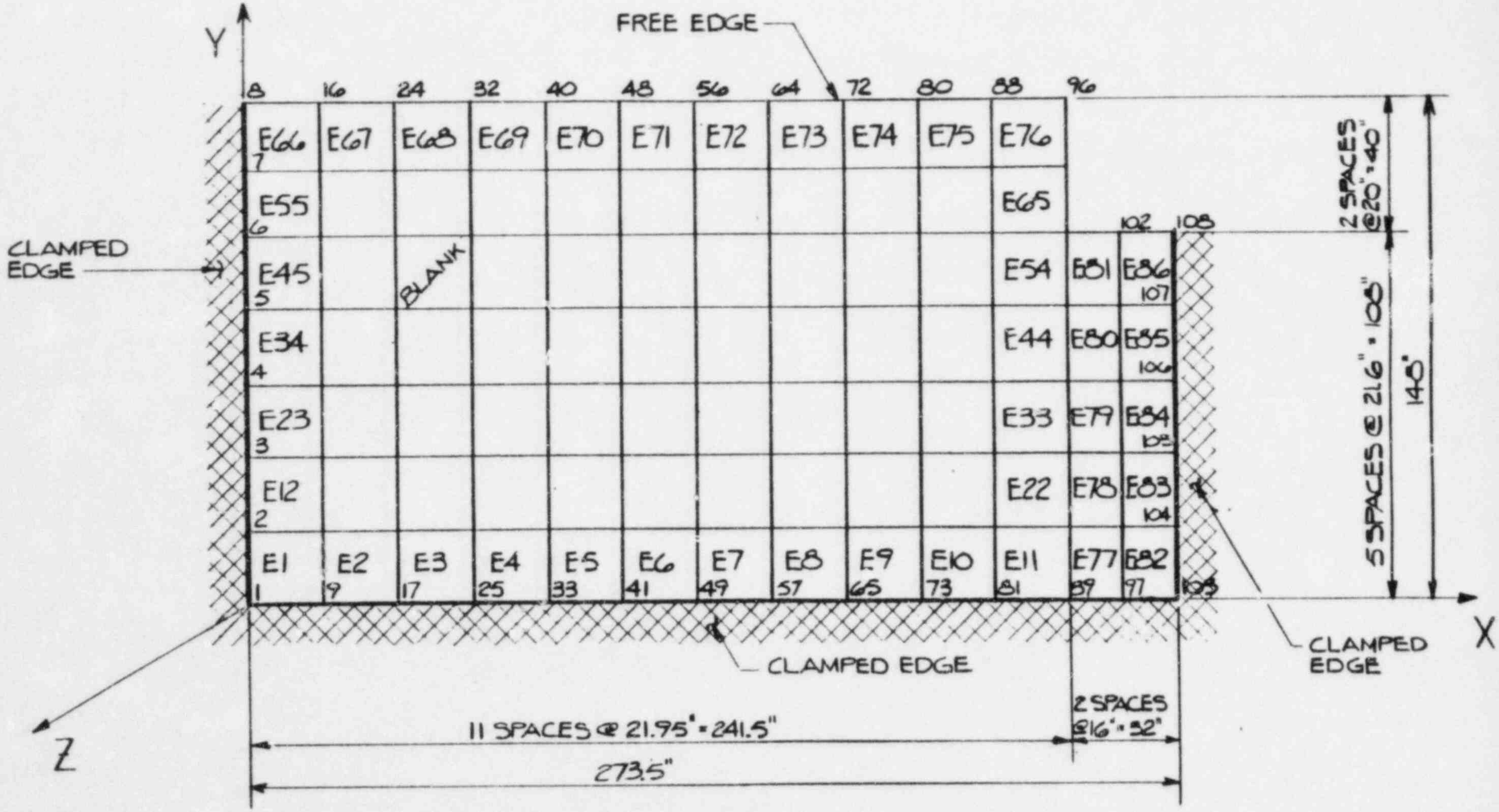


WALL 4 MODEL

FIGURE 17

FIGURE 18

Gilbert / Commonwealth



WALL 5 MODEL

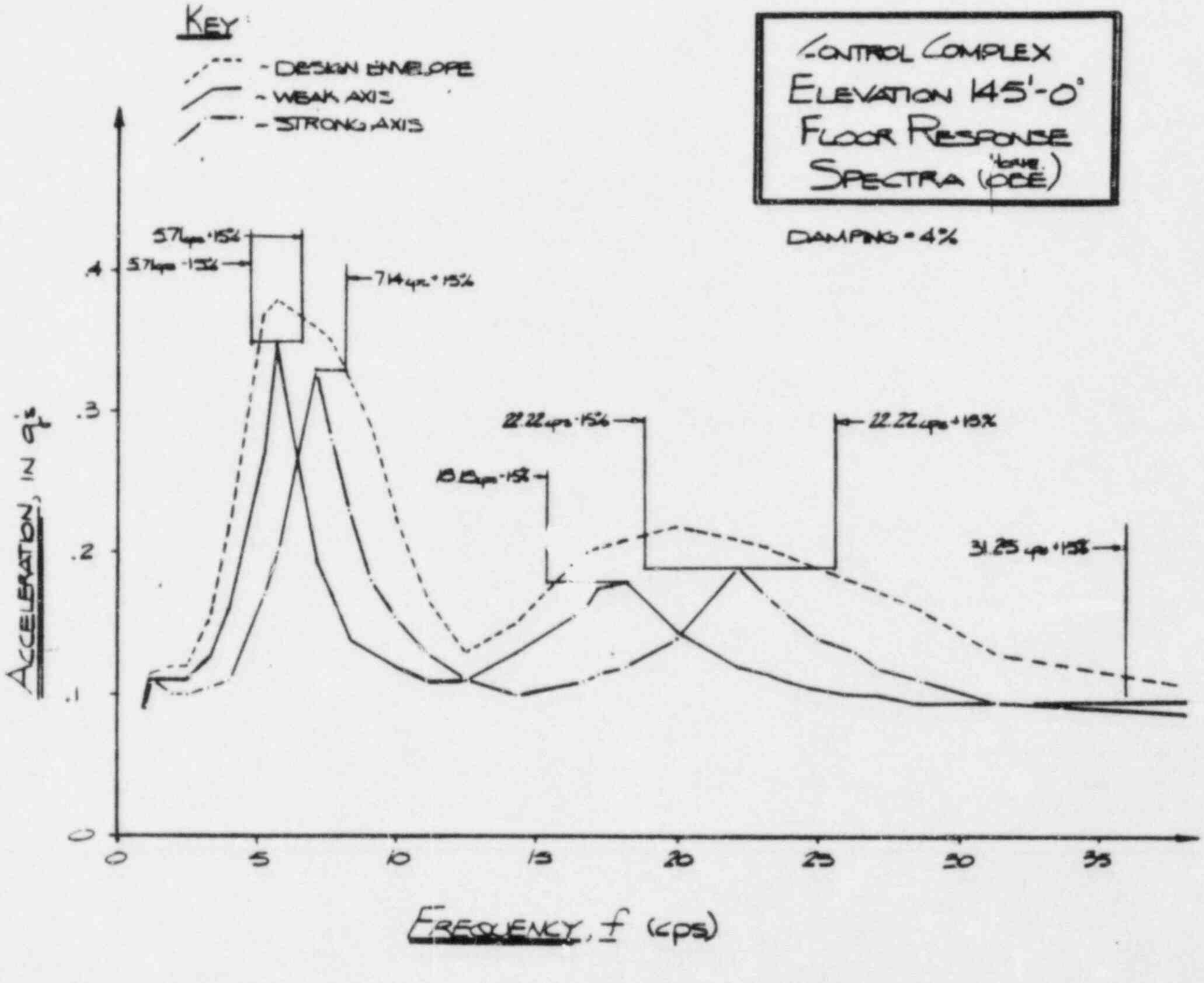

KEY
- DESIGN ENVELOPE
- WEAK AXIS
- STRONG AXIS
CONTROL COMPLEX
ELEVATION 145'-0"
FLOOR RESPONSE
SPECTRA (OBE)
DAMPING = 4%
5.71cps +15%
5.71cps -15%
7.14cps +15%
22.22cps -15%
22.22cps +15%
31.25cps +15%
ACCELERATION, IN g's
FREQUENCY, f (cps)
0
.1
.2
.3
.4
0
5
10
15
20
25
30
35

FIGURE 19

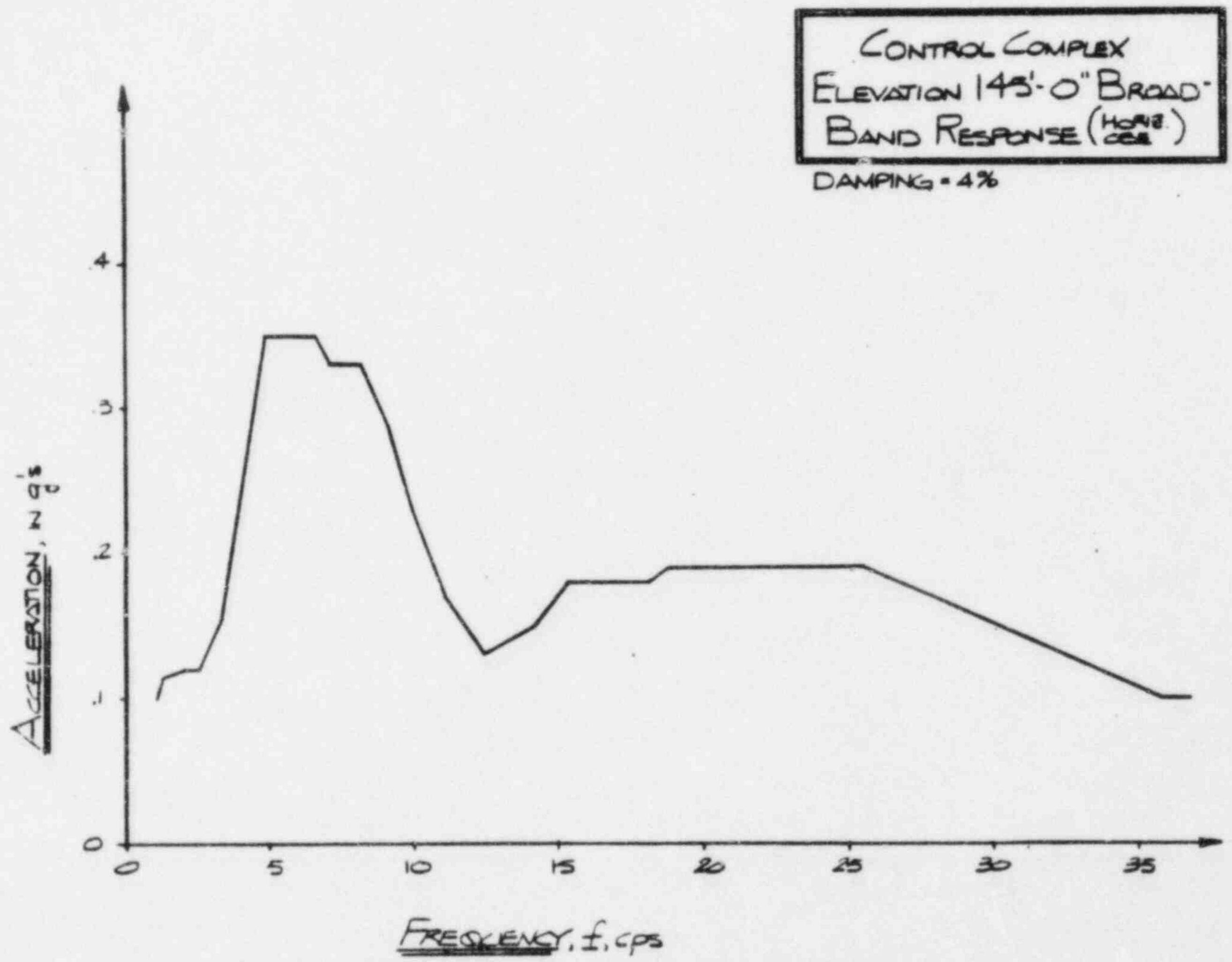
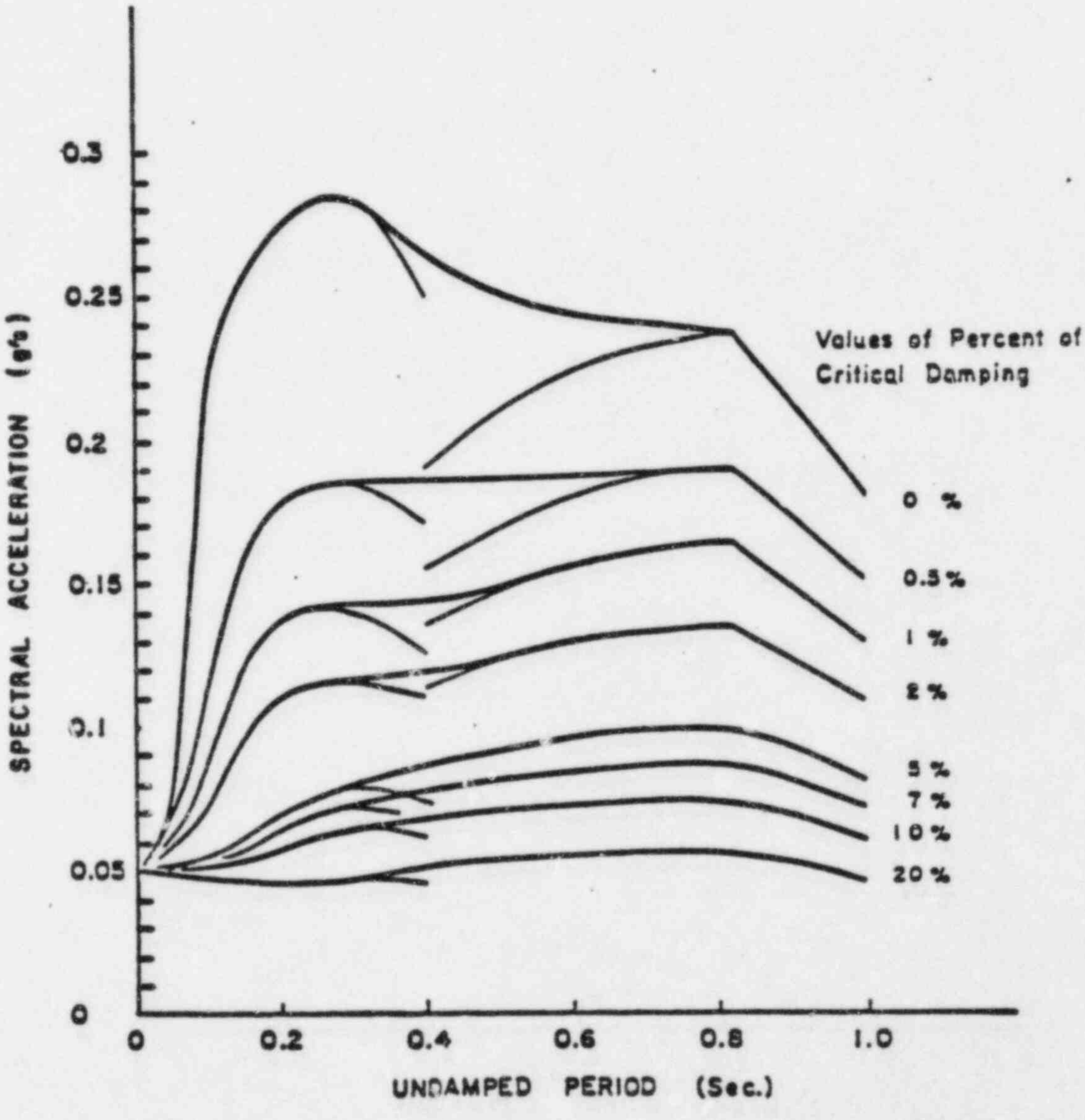
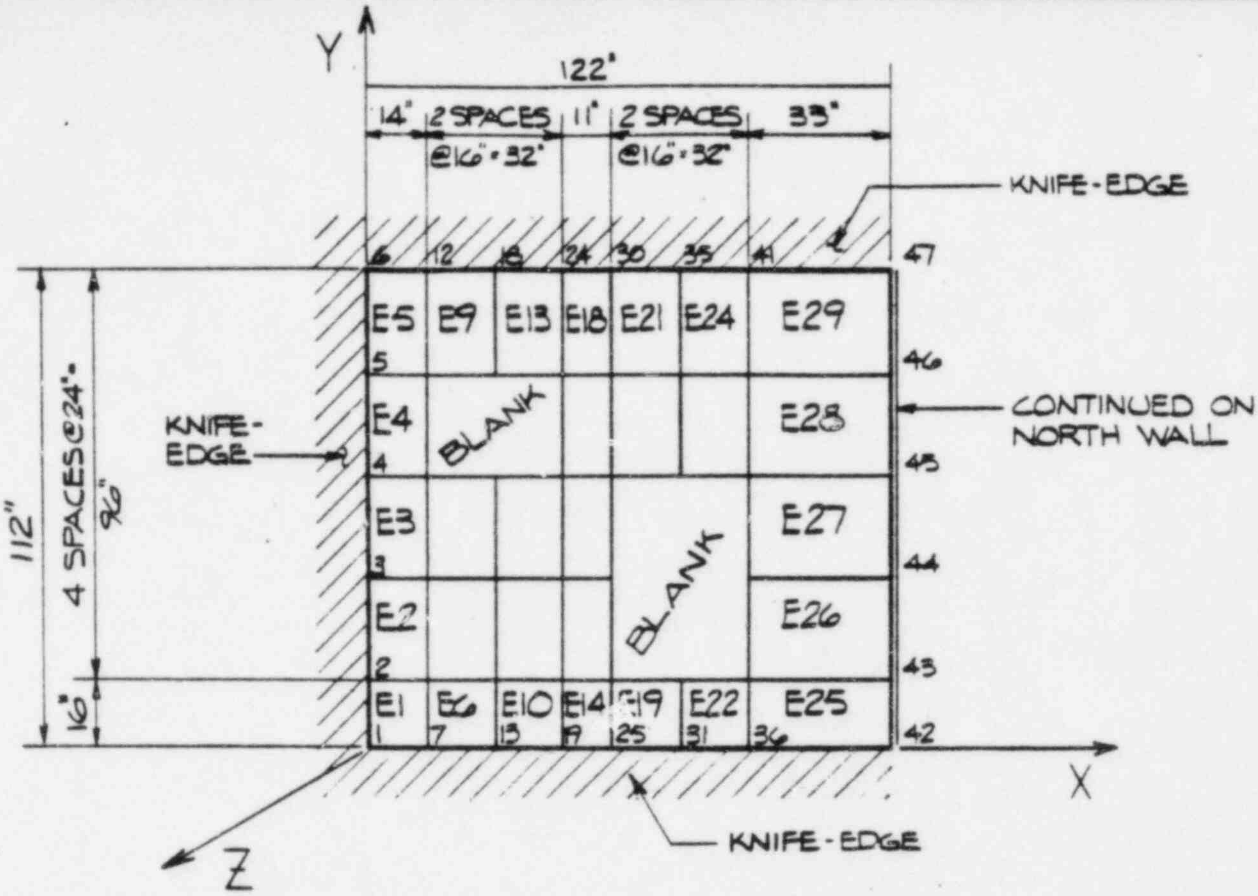

CONTROL COMPLEX
ELEVATION 143'-0" BROAD-
BAND RESPONSE (HORIZ. OBE)
DAMPING = 4%
ACCELERATION, IN g's
.4
.3
.2
.1
0
0
5
10
15
20
25
30
35
FREQUENCY, f, cps

FIGURE 20

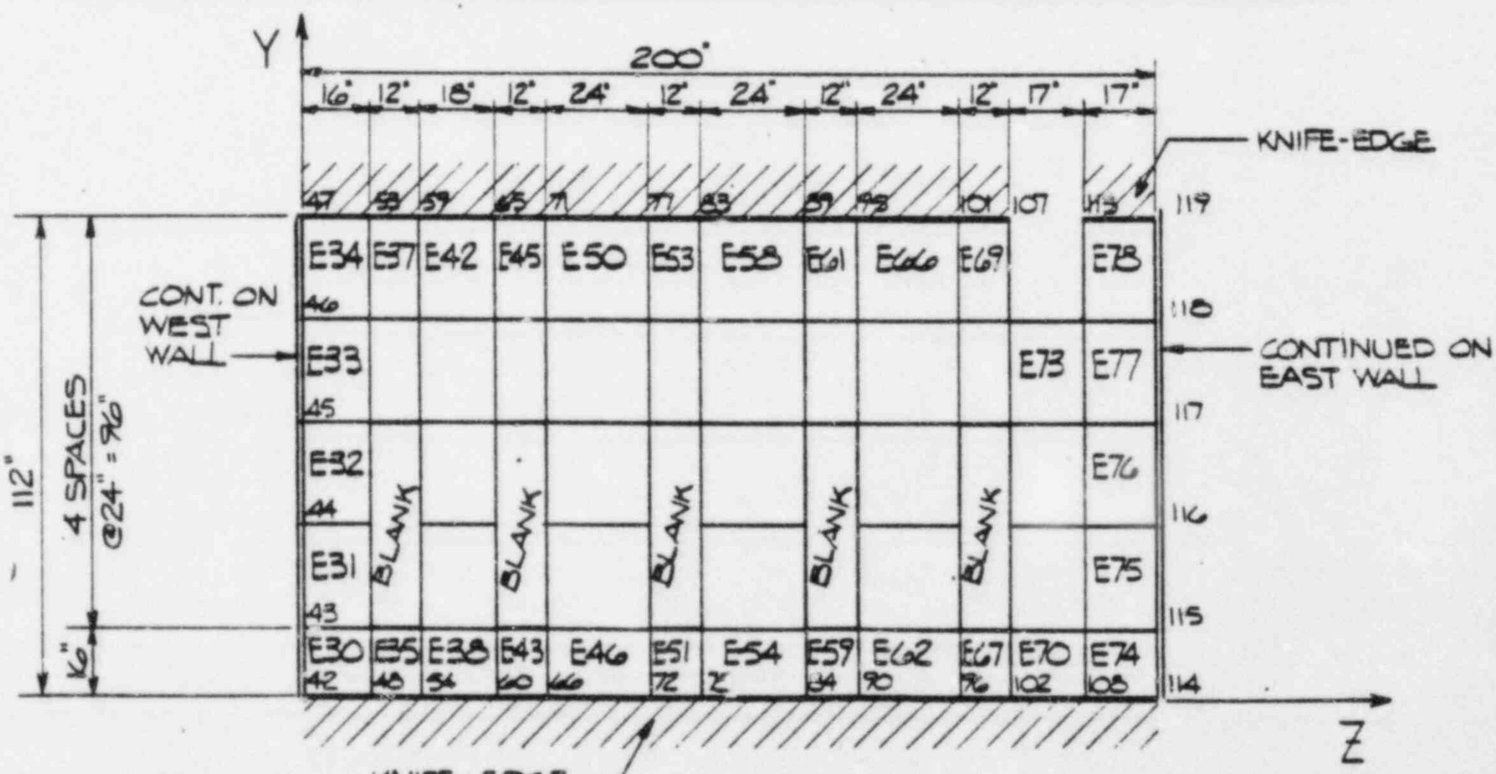


DESIGN GROUND RESPONSE SPECTRA

FIGURE 21



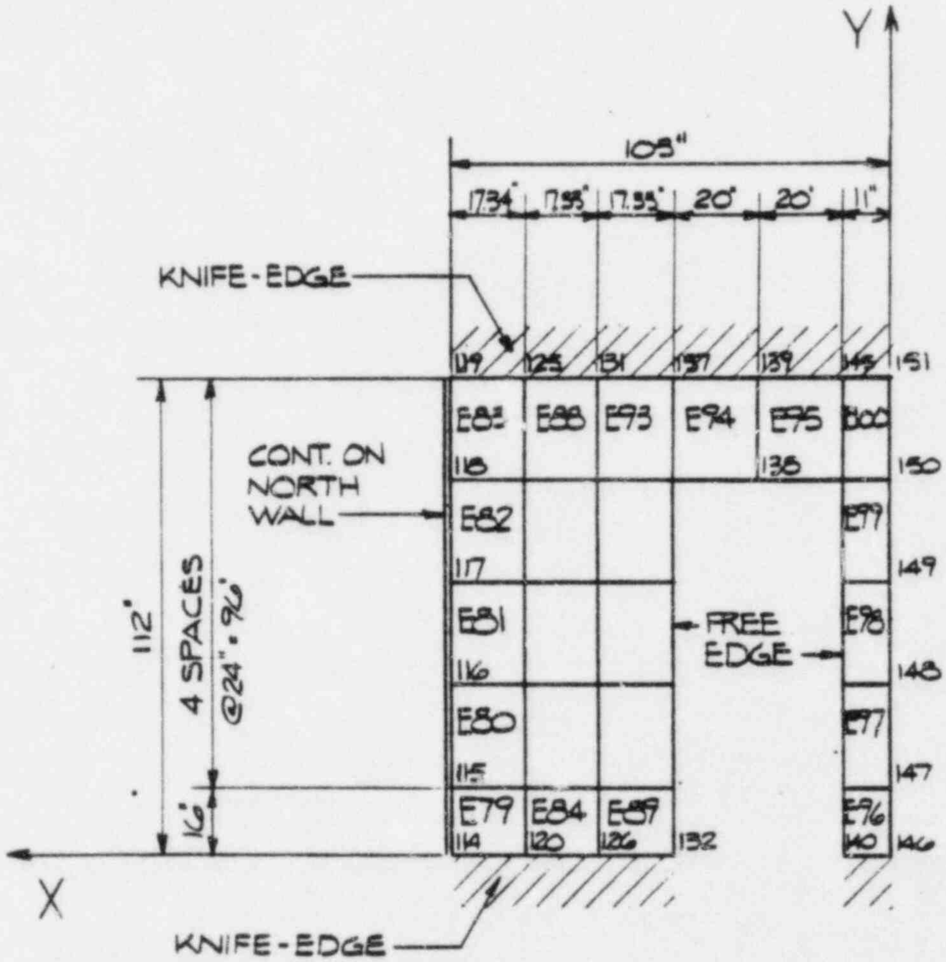
INSTRUMENT ROOM WEST WALL MODEL



INSTRUMENT ROOM NORTH WALL MODEL

FIGURE 22

Gilbert/Commonwealth



INSTRUMENT ROOM
EAST WALL MODEL

FIGURE 23