PDR

OYSTER CREEK NUCLEAR GENERATING STATION

JCP&L / GPU

Jersey Central Power & Light Company is a Member of the General Public Utilities System

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November 14, 1980

Mr. Boyce H. Grier, Director
Office of Inspection and Enforcement
Region I
U. S. Nuclear Regulatory Commission
631 Park Avenue
King of Prussia, Pennsylvania 19406

Dear Mr. Grier:

Subject: Oyster Creek Nuclear Generating Station
Docket No. 50-219
I. E. Bulletin 80-11

Our letters of July 7, 1980 and September 19, 1980 supplied the preliminary information as required by I. E. Bulletin 80-11. In accordance with the request for information described in item 2b of the bulletin, the attached represents the majority of our response. The final reevaluation of the design adequacy of all identified walls to perform their intended function under all postualted loads and load combinations is however, not yet complete.

The delay in meeting our original schedule, enclosed in our July 7, 1980 submittal, is attributed primarily to difficulties encountered in the development of the reevaluation criteria. The final reevaluation for all walls will be submitted on or before May 1, 1981. If additional information or further clarification is needed, please contact Mr. J. Knubel of my staff at 201-455-8753.

Very truly yours,

Ivan R. Finfrock

Ivan R. Finfrock, Jr.
Vice President

Sworn and subscribed to before me this 14th day of November, 1980.

David C Hodge
Notary Public

cc: NRC Office of Inspection and Enforcement
Division of Reactor Operations Inspection
Washington, D.C. 20555.

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Oyster Creek Nuclear Station

Reevaluation of Safety-Related Concrete Masonry Walls NRC IE Bulletin 80-11

GPU Nuclear Corporation
100 Interpace Parkway
Parsippany, N.J. 07054

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ENCLOSURE 1

SUMMARY

1. Reanalysis Status

In the sixty day report we had identified forty-seven safety related walls that were required to be reanalyzed. Thereafter, for convenience, wall no. 36 had been incorporated in wall no. 42. Consequently, there are 46 safety-related walls that require reanalysis. In the course of the plant survey, it was determined that minor preemptive modifications to 19 selected walls would remove the potential missile hazard to the vital systems. The removal of effected sections of these walls will preclude further reanalysis. However, the reporting requirements for the Bulletin's 180 day response will still apply. The block walls included in this scope are wall no.'s 1, 3, 4, 9, 10, 11, 12, 13, 14, 16, 34, 35, 37, 38, 39, 40, 41, 46 and 47. The preliminary reanalysis of the remaining 27 walls have been performed. Only wall No. 18 has been completely reanalyzed. The results of the reanalysis will be discussed below.

2. Preliminary Analysis

The results of the preliminary evaluation for twenty seven walls can be classified in three categories.

a. Walls which appear to be difficult to qualify without some modification. Block walls included in this category are wall no.'s 8, 29 and 30. These walls have been reassigned to be high priority for reanalysis.

b. Walls which are marginal. These are walls for which calculated stresses exceed the allowable limit when modeled conservatively. These walls are wall no.'s 15, 17, 18, 19, 20, 21, 22, 23, 24, 25, 26, 27, 28, 42 and 44. With accurate modeling and detailed reanalysis some of the walls are expected to qualify. These walls have also been reassigned to be high priority for reanalysis.

c. Walls which should qualify. These are walls for which calculated stresses fall near or within the allowable limits even when modeled conservatively. The block walls in this category are wall no.'s 2, 5, 6, 7, 31, 32, 33, 43, and 45.

In the final reevaluation of the walls, consideration will be given to the secondary effects such as penetrations and doorways, attached equipment loads, and the effects of higher wall response modes beyond the fundamental.

3. Final Reevaluation

The results of the final analysis for wall no. 18 show that the calculated stresses are below the allowable (see enclosure 8). The mathematical model in this analysis had assumed pinned at all four sides of the wall. The north vertical boundary of the wall is interwoven with block wall no. 17. To be consistent with this design assumption, boundary supports will be provided at the top and south boundary of the wall.

4. Reevaluation Criteria

A reevaluation criteria for concrete masonry walls is provided in enclosure 2.

This criteria document has been used for evaluation of the block walls. The main topics in this criteria are: Governing codes, load and load combinations, Materials, Design allowables, Analytical techniques and alternative acceptance criteria.

5. Justification for the Criteria

The jusitification for the criteria for reevaluating the concrete masonry walls is in enclosure 3.

6. General Arrangement of Block Walls

The location of the walls including wall identification number, floor elevation and the plant structures are shown in enclosure 4.

7. Configuration of Block Walls

For configuration of the walls, specific location and the type of material see enclosure 5.

8. Function of the Walls

The function of the block walls including the reevaluation status is shown in enclosure 6.

9. Floor Response Spectra

The design floor response spectra for block wall reanalysis are shown in enclosure 7.

10. Construction Practices

Enclosure 9 is the original plant design masonry specification that outlines the requirements for the material, codes and standards, testing and installation for the concrete masonry walls.

11. Schedule

Enclosure 10 shows the schedule for reevaluation of the remaining walls. This schedule is based on experience to date performing the reanalysis of the concrete Block walls.

ENCLOSURE 2

CRITERIA FOR THE RE-EVALUATION

OF CONCRETE MASONRY WALLS

OYSTER CREEK NUCLEAR POWER PLANT

Prepared for

GPU SERVICE CORPORATION
Parsippany, NJ

by

COMPUTECH ENGINEERING SERVICES, INC.
2150 Shattuck Ave.
Berkeley, CA

October 1980

computech

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CRITERIA FOR THE RE-EVALUATION

OF CONCRETE MASONRY WALLS

FOR THE

OYSTER CREEK NUCLEAR POWER PLANT

1.0 GENERAL

1.1 Purpose

This specification is provided to establish design requirements and criteria for use in re-evaluating the structural adequacy of concrete block walls as required by NRC IE Bulletin 80-11, Masonry Wall Design, dated May 8, 1980.

1.2 Scope

The re-evaluation shall determine whether the concrete masonry walls will perform their intended function under loads and load combinations specified herein. Concrete masonry walls not supporting safety systems but whose collapse could result in the loss of required function of safety related equipment or systems shall be evaluated to demonstrate that an SSE, accident or tornado load will not cause failure to the extent that functions of safety related items is impaired. Verification of wall adequacy shall take into account support condition, global response of wall, and local transfer of load. Evaluation of anchor bolts and embedments are not considered to be within the scope of IE Bulletin 80-11.

2.0 GOVERNING CODES

For the purposes of re-evaluation, the American Concrete Institute "Building Code Requirements for Concrete Masonry Structures" (ACI 531-79) will be used except as noted herein.

3.0 LOADS AND LOAD COMBINATIONS

The walls shall be evaluated for the following loads.

3.1 Service Load Conditions

D + R + T + E

3.2 Factored Load Conditions

D + R + T + E'

3.3 Definition of Terms

D - Dead loads or their related internal moments and forces including any permanent equipment loads.

R - Pipe reactions during normal operating or shutdown conditions, based on the most critical transient or steady-state conditions.

T - Thermal effects and loads during normal operating or shutdown conditions, based on the most critical transient or steady-state conditions.

E - Loads generated by the operating basis earthquake.

E'- Loads generated by the safe shutdown earthquake.

4.0 MATERIALS

The project specifications indicate that materials used for the performance of the work were originally specified to meet the following requirements.

4.1 Concrete Masonry Units

Hollow concrete blocks:

Non load bearing C-129

Load bearing C-90

4.2 Mortar

ASTM C-270 Type M

4.3 Grout

None specified.

4.4 Horizontal Joint Reinforcing

Extra heavy "Dur-o-wal" - where shown.

4.5 Bar Reinforcement

ASTM Designation A15, intermediate grade, deformed bars per ASTM 305. (Grade 40)

5.0 DESIGN ALLOWABLES

5.1 Stresses

Allowable stresses for the loads and load combinations given in Section 3.1 will be as given in this section based on the following compressive strengths:

Hollow Concrete Units	f'_m = 1200 psi	(load bearing)
Hollow Concrete Units	f'_m = 400 psi	(non-load bearing)
Mortar	M_o = 2500 psi	

Stresses in the reinforcement and masonry shall be computed using working stress procedures.

The allowable stresses for service loads given in Section 3.1 shall be the S values given in Tables 1 and 2 for reinforced and unreinforced masonry respectively. For walls subjected to thermal effects the allowable stress shall be 1.3 times the S values given in Tables 1 and 2. The allowable stresses for the factored loads given in Section 3.2 shall be the U values given in Tables 1 and 2 for reinforced and unreinforced masonry respectively.

5.2 Damping

The damping values to be used shall be as follows:

Unreinforced Walls

2% - OBE
4% - SSE

Reinforced Walls

4% - OBE
7% - SSE

6.0 ANALYSIS AND DESIGN

6.1 Structural Response of Unreinforced Masonry Walls

6.1.1 Out of Plane Effects

The following sequence of analysis methods will be applied.

1. Walls without significant openings shall be assumed to be a simply supported beam spanning vertically and/or horizontally and the natural frequency shall be determined. A fully grouted wall may be evaluated either as an uncracked wall or if it is grouted it may be assumed that the mortar joint on the tension side is cracked and the moment of inertia calculated by neglecting the mortar and block on the tension side. If the latter is used the grout core tensile stress is evaluated.

2. The maximum moment and stress shall be determined by applying a uniform load to the beam. The maximum value of the uniform load shall be mass times acceleration taken from the response spectrum curve at the appropriate frequency for the fundamental mode. If only one mode of vibration is calculated the moments and stresses shall be multiplied by 1.05 to account for higher mode effects.

3. If the calculated stresses exceed the allowables or the wall has a significant opening(s) the wall shall be modeled as a plate with appropriate boundary conditions. For a multimode analysis the modal responses shall be combined using the square root of the sum of the squares.

4. If the calculated stresses exceed the allowables and the wall is multiwythe steps 1, 2 and 3 shall be repeated using composite action if the wall contains a verifiable collar joint.

5. If the calculated stresses exceed the allowables in step 3 for a single wythe wall and step 4 for a multiwythe wall the wall will be evaluated for operability.

6.1.2 Frequency Variations in Out of Plane

Uncertainties in structural frequencies of the masonry wall resulting from variations in mass, modulus of elasticity, material and section properties shall be taken into account by varying the modulus of elasticity as follows:

Ungrouted Walls - $1000f'_m$ to $600f'_m$

Grouted or
Solid Walls - $1200f'_m$ to $800f'_m$

If the wall frequency using the lower value of E is on the higher frequency side of the peak of the response spectrum it is considered conservative to use the lower value of E. If the wall frequency is on the lower frequency side of the peak of the response spectrum the peak acceleration shall be used. If the frequency of the wall using the higher value of E is also on the lower frequency side of the peak the higher value of E may be used with its appropriate spectral value provided due consideration is given to frequency variations resulting from all possible boundary conditions.

6.1.3 In Plane and Out of Plane Effects

Provided both the allowable stress criteria for out of plane effects and the in plane stress or strain criteria are satisfied the walls shall be considered to satisfy the re-evaluation criteria. If either criterion is exceeded walls will be evaluated for operability.

6.1.4 Stress Calculations

All stress calculations shall be performed by conventional methods prescribed by the Working Stress Design method. The collar joint shear stress shall be determined by the relationship VQ/Ib.

6.2 Structural Response of Reinforced Masonry Walls

6.2.1 Out of Plane Effects

The following sequence of analysis methods will be applied.

1. Walls without significant openings will initially be assumed to be uncracked and Steps 1 and 2 of Sec. 6.1.1 will be followed. Note that either or both the uncracked section or the section neglecting the block and mortar on the tension side may be used. If the latter is used the grout core tensile stress is evaluated. If the allowable stresses for an unreinforced wall given in Table 2 are

exceeded the wall will be assumed to crack and the equivalent moment of inertia for a cracked section given in Sec. 6.2.2 shall be used. If the calculated stresses exceed the allowables of Table 1, Step 2 shall be used.

2. For walls with openings or those exceeding the reinforced stress levels in Step 1 the wall shall be modeled as a plate with appropriate boundary conditions assuming the wall is uncracked. See Step 1 for section properties. If the allowable stresses for an unreinforced wall given in Table 2 are exceeded the plate will be assumed to crack and the equivalent moment of inertia given in Sec. 6.2.2 shall be used. For a multimode analysis the modal responses shall be combined using the square root of the sum of the squares. If the calculated stresses exceed the allowables of Table 2 a single wythe wall will be evaluated for operability; a multiwythe wall will be further evaluated using Step 3.

3. For multiwythe walls where a single wythe of the wall does not meet the stress criteria in Steps 1 and 2, Steps 1 and 2 shall be repeated using composite action provided the wall contains a verifiable collar joint.

6.2.2 Equivalent Moment of Inertia

6.2.2.1 Uncracked Condition

The equivalent moment of inertia of an uncracked wall (I_t) shall be obtained from a transformed section consisting of the block, mortar, cell grout or core concrete. (Note that a centrally reinforced wall has the same moment of inertia as an unreinforced section.) Alternatively if the mortar joint is assumed to crack or actually cracks the equivalent moment of inertia may be calculated by neglecting the mortar and block on the tension side.

6.2.2.2 Cracked Condition

If the stresses due to all load combinations exceed the allowables the wall shall be considered to be cracked. In this event the equivalent moment of inertia (I_e) shall either be conservatively calculated from the fully cracked section properties of the wall or as follows:

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_t + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \qquad (1)$$

$$M_{cr} = f_r \left(\frac{I_t}{y}\right) \qquad (2)$$

where:

M_{cr} = Uncracked moment capacity

M_a = Applied maximum moment on the wall

I_t = Moment of inertia of transformed section

I_{cr} = Moment of inertia of the cracked section

f_r = S value of tensile stress defined in Table 2 multiplied by 2 for mortar and grout if the masonry joint is assumed to be cracked.

y = Distance of neutral plane from tension face

If I_e of Equation 1 is calculated this should be used over the full length of the wall. If I_{cr} is used this can be used in the cracked region only.

If the use of I_e results in an applied momemt M_a which is less than M_{cr}, then the wall shall be verified for M_{cr}.

6.2.3 Frequency Variations

Uncertainties in structural frequencies of the masonry wall resulting from variations in mass, modulus of elasticity, material and section properties shall be taken into account by varying the modulus of elasticity from $1200f'_m$ to $800f'_m$. It is considered conservative to use the lower value of E if the wall frequency is on the higher frequency side of the peak response spectrum. If the wall frequency using the lower values of E is on the lower frequency side of the peak of the response spectrum the peak acceleration shall be used. If the frequency of the wall using the higher value of E is also on the lower frequency side of the peak the higher value of E may be used with its appropriate spectral value provided due consideration is given to frequency variations resulting from all possible boundary conditions.

6.2.4 In Plane and Out of Plane Effects

Provided both the allowable stress criteria for out of plane effects and the in plane stress or strain criteria are satisfied the walls shall be considered to satisfy the re-evaluation criteria. If either criterion is exceeded the walls will be evaluated for operability.

6.2.5 Stress Calculations

All stress calculations shall be performed by conventional methods prescribed by the Working Stress Design method. The collar joint shear stress shall be determined by the relationship VQ/Ib for uncracked sections and in the compression zone of cracked sections. The relationship V/bjd shall be used for collar joints in cracked sections between the neutral axis and the tension steel.

6.3 Accelerations

For a wall spanning between two floors the envelope of the spectra for the floor above and below shall be used to determine the stresses in the walls.

6.4 Interstory Drift Effects

The magnitude of interstory drift effects shall be determined from the original dynamic analysis.

6.5 In Plane Effects

If a masonry wall is a load bearing structural element shear stresses shall be evaluated and compared with the allowable stresses of Tables 1 and 2.

If the wall is an infill panel or non-load bearing element, shear stresses resulting from interstory drift effects will not be calculated. In this case the imposed interstory deflections of Sec. 6.4 shall be compared to the displacements calculated from the following permissible strains for service loads. For factored loads the strains shall be multiplied by 1.67. The deflections shall be calculated by multiplying the permissible strain by the wall height.

Unconfined Walls (1)
$\gamma_u = 0.0001$

Confined Walls (2)
$\gamma_c = 0.0008$

Notes: (1) An unconfined wall is attached on one vertical boundary and its base.

(2) A confined wall is attached in one of the following ways:

(a) On all four sides.
(b) On the top and bottom of the wall.
(c) On the top, bottom and one vertical side of the wall.
(d) On the bottom and two vertical sides of the wall.

If an infill panel or non-load bearing element is subjected to both interstory drift effects and shear stresses due to inplane loads from equipment or piping the following criteria shall apply.

$$\frac{\text{actual inplane shear stress}}{\text{allowable inplane shear stress}} + \frac{\text{actual interstory deflection}}{\text{allowable interstory deflection}} \leq 1$$

A more refined analysis may be performed if necessary.

6.6 Equipment

If the total weight of attached equipment is less than 100 lbs. the effect of the equipment on the wall shall be neglected. If the total weight of the equipment is greater than 100 lbs. the mass of the equipment shall be added to that of the wall in calculating the frequency of the wall.

Stresses resulting from each piece of equipment weighing more than 100 lbs. shall be combined with the wall inertial loads using the absolute sum method. The SRSS method may be used provided its application is justified.

Stresses resulting from the equipment shall be calculated by applying a static load consisting of the weight of equipment multiplied by the peak acceleration of the response spectrum for the floor level above the wall. If the frequency of the equipment is known it may be used to determine the static load.

6.7 Distribution of Concentrated Out of Plane Loads

6.7.1 Beam or One Way Action

For beam action local moments and stresses under a concentrated load shall be determined using beam theory. An effective width of four times the wall thickness shall be used; however, such moments shall not be taken as less than that for two way plate action.

6.7.2 Plate or Two Way Action

For plate action local moments and stresses under a concentrated load shall be determined using appropriate analytical procedures for plates or determined numerically using a finite element analysis.

A conservative estimate of the localized moment per unit length for plates supported on all edges can be taken as:

$$M_L = 0.4P$$

where: M_L = Localized moment per unit length (in-lbs/in)

P = Concentrated load perpendicular to wall (lbs)

For loads close to an unsupported edge the upper limit moment per unit length can be taken as:

$$M_L = 1.2P$$

6.7.3 Localized Block Pullout

For a concentrated load block pullout shall be checked using the allowable values for unreinforced shear walls in Table 2. This allowable shall be used for both reinforced and unreinforced walls.

7.0 ALTERNATIVE ACCEPTANCE CRITERIA (OPERABILITY)

7.1 Reinforced Masonry

Where bending due to out-of-plane inertial loading causes flexural stresses in the wall to exceed the allowable stresses for reinforced walls, the wall can be evaluated by the "energy balance technique".

7.1.1 Effects on Equipment

If the deflection calculated by the energy balance technique exceeds three times the yield deflection, the resulting deflection shall be multiplied by a factor of 2 and a determination made as to whether such factored displacements would adversely impact the function of safety-related systems attached and/or adjacent to the wall.

7.1.2 Effects on Walls

The maximum deflection in the wall due to out-of-plane inertia loading shall be limited to 5 times the yield displacement. The yield displacement shall be calculated by reinforced concrete ultimate strength theory, and the masonry compression stresses of $0.85f'_m$ based on a rectangular stress distribution shall be used.

7.2 Unreinforced Masonry

When, due to out-of-plane loading, the allowable stresses for unreinforced masonry are exceeded, the arching theory for masonry walls may be used to measure the capacity of the walls. Due regard must be paid to the boundary conditions.

7.2.1 Limiting Deflection

The deflection of the three hinged arch could be determined by assuming that the arch members are analogous to regular compression members in a truss. The method of virtual work (unit load method) may be used to compute the deflection at the arch interior hinge. The calculated deflection should not be more than 0.3T where the "T" is the thickness of the wall. A determination should be made as to whether such calculated displacements would adversely impact the function of safety-related systems attached and/or adjacent to the wall.

7.2.2 Allowable Stresses

The total resistance of the wall (f) shall be calculated using the following stresses:

I. Tensile stress through the assumed tension crack shall be $6\sqrt{f'_c}$ for grouted walls or f_t for ungrouted walls.

II. The crushing stress of block material = $0.85f'_m$.

By applying a factor of safety of 1.5 to the total resistance (f) as calculated above, the allowable load on the wall is limited to f /1.5.

7.2.3 Boundary Supports

The boundary supports should be checked if they are capable of transmitting the reaction forces applied to them. The effect of support stiffness on the reaction forces should be considered.

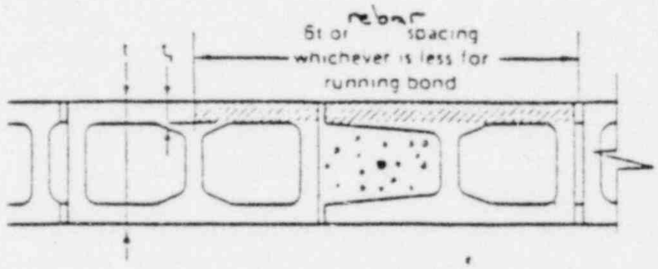
Table 1: Allowable Stresses in Reinforced Masonry

Description	S Allowable (psi)	S Maximum (psi)	U Allowable (psi)	U Maximum (psi)
Compressive				
Axial[(1)]	$0.22f'_m$	1000	$0.44f'_m$	2000
Flexural	$0.33f'_m$	1200	$0.85f'_m$	2400
Bearing				
On full area	$0.25f'_m$	900	$0.62f'_m$	1800
On one-third area or less	$0.375f'_m$	1200	$0.95f'_m$	2400
Shear				
Flexural members[(2)]	$1.1\sqrt{f'_m}$	50	$1.7\sqrt{f'_m}$	75
Shear Walls[(3,4)]				
Masonry Takes Shear				
$M/Vd \geqslant 1$	$0.9\sqrt{f'_m}$	34	$1.5\sqrt{f'_m}$	56
$M/Vd = 0$	$2.0\sqrt{f'_m}$	74	$3.4\sqrt{f'_m}$	123
Reinforcement Takes Shear				
$M/Vd \geqslant 1$	$1.5\sqrt{f'_m}$	75	$2.5\sqrt{f'_m}$	125
$M/Vd = 0$	$2.0\sqrt{f'_m}$	120	$3.4\sqrt{f'_m}$	180
Reinforcement				
Bond				
Plain Bars		60		80
Deformed Bars		140		186
Tension				
Grade 40		20,000		$0.9F_y$
Grade 60		24,000		$0.9F_y$
Joint Wire		$.5F_y$ or 30,000		$0.9F_y$
Compression		$0.4F_y$		$0.9F_y$

Notes to Table 1:

(1) These values should be multiplied by $(1 - (\frac{h}{40t})^3)$ if the wall has a significant vertical load.

(2) This stress should be evaluated using the effective area shown in Figure 1.



Area assumed effective in flexural compression, force normal to face

FIGURE 1

(3) Net bedded area shall be used with these stresses

(4) For M/Vd values between 0 and 1 interpolate between the values given for 0 and 1.

Table 2: Allowable Stresses in Unreinforced Masonry

Description	S		U	
	Allowable (psi)	Maximum (psi)	Allowable (psi)	Maximum (psi)
Compressive				
Axial(1)	$0.22f'_m$	1000	$0.44f'_m$	2000
Flexural	$0.33f'_m$	1200	$0.85f'_m$	3000
Bearing				
On full area	$0.25f'_m$	900	$0.62f'_m$	2250
On one-third area or less	$0.375f'_m$	1200	$0.95f'_m$	3000
Shear				
Flexural members(2, 3)	$1.1\sqrt{f'_m}$	50	$1.7\sqrt{f'_m}$	75
Shear walls(2)	$0.9\sqrt{f'_m}$	34	$1.35\sqrt{f'_m}$	51
Tension				
Normal to bed joints				
Hollow units	$0.5\sqrt{m_o}$	25	$0.83\sqrt{m_o}$	62
Solid or grouted	$1.0\sqrt{m_o}$	40	$1.67\sqrt{m_o}$	67
Parallel to bed joints(4)				
Hollow units	$.0\sqrt{m_o}$	50	$1.67\sqrt{m_o}$	84
Solid or grouted	$1.5\sqrt{m_o}$	80	$2.5\sqrt{m_o}$	134
Grout Core	$2.5\sqrt{f'_c}$		$4.2\sqrt{f'_c}$	
Collar joints				
Shear		8		12
Tension		8		12

Notes to Table 2:

(1) These values should be multiplied by $(1 - (\frac{h}{40t})^3)$ if the wall has a significant vertical load.

(2) Use net bedded area with these stresses.

(3) For stacked bond construction use two-thirds of the values specified.

(4) For stacked bond construction use two-thirds of the values specified for tension normal to the bed joints in the head joints of stacked bond construction.

ENCLOSURE 3

JUSTIFICATION FOR THE CRITERIA FOR THE RE-EVALUATION OF CONCRETE MASONRY WALLS

OYSTER CREEK NUCLEAR POWER PLANT

Prepared for

GPU SERVICE CORPORATION
Parsippany, NJ

by

COMPUTECH ENGINEERING SERVICES, INC.
2150 Shattuck Ave.
Berkeley, CA

October 1980

computech

CONTENTS

JUSTIFICATION FOR THE CRITERIA FOR THE RE-EVALUATION OF CONCRETE MASONRY WALLS

1.0 GENERAL

The specification is provided to establish design requirements and criteria for use in re-evaluating the structural adequacy of concrete block walls in nuclear power plants. Direct reference to building code criteria was not used for the following reasons:

1) The definition of the magnitude of seismic loads in building codes is different than that used in nuclear power plants. In building codes damping, ductility, site effects and framing systems are factored into the seismic design base shear force. In nuclear power plants these factors are considered explicitly in the design of components.

2) Building code allowable stresses do not consider two levels of earthquake ground motion and the magnitude of the ground motion included in the building code design spectrum is not explicit.

3) Factors such as damping, analysis procedures, effect of attached equipment, two levels of allowable stresses, operability and frequency variations are not considered in building codes.

Thus the specification was developed to address the problems unique to nuclear power plants.

2.0 GOVERNING CODES

As noted in Sec. 1 the specification covers most of the factors unique to nuclear power plants. Items not explicitly covered by the specification will be governed by the American Concrete Institute "Building Code Requirements for Concrete Masonry Structures". ACI-531(29). This code incorporates most of the recent research data available on concrete masonry.

3.0 LOADS AND LOAD COMBINATIONS

These are in conformance with the plant FSAR and are in accordance with the design of all structural elements.

4.0 MATERIALS

The project specifications indicate that materials used for the performance of the work were originally specified to meet the requirements given in this section.

5.0 DESIGN ALLOWABLES

The design allowable stresses given in Tables 1 and 2 are based on f'_m the prism compressive strength, m_o the mortar compressive strength or f_y the steel yield strength. The mortar compressive strength is based on the mimimun specified compressive strength of ASTM C-270. The concrete block unit compressive strength is based on the applicable ASTM Standard - ASTM C-90 for hollow units, ASTM C-145 for solid units and ASTM C-129 for hollow non-load bearing units. The steel yield strength is based on the specified grade of the steel.

The prism compressive strength f'_m is based on the specified values given in Table 4-3 of ACI 531-79. This Table provides a conservative estimate of f'_m based on the mortar and concrete block unit compressive strengths. The minimum ASTM specified values of these variables was used in determining the conservative estimate of f'_m.

5.1 ALLOWABLE STRESSES

The justification for the allowable stresses of Tables 1 and 2 follows.

5.1.1 AXIAL COMPRESSION (Reinforced and Unreinforced)

The following discussion of test results has been extracted from the commentary to the NCMA Specification for the Design and Construction of Load Bearing Concrete Masonry.

The objective was to develop reasonable and safe engineering design criteria for nonreinforced concrete masonry based on all existing data. A review in 1967 of the compilation of all available test data on compressive strength of concrete masonry walls did not, according to some, provide a suitable relationship between wall strength and slenderness ratio. From a more recent analysis, it was noted in many of the 418 individual pieces of data that either the masonry units or mortar, or in some cases, both units and mortar, did not comply with the minimum strength requirements established for the materials permitted for use in "Engineered Concrete Masonry" construction. Accordingly, it was decided to re-examine the data, discarding all tests which included materials that did not comply with the following minimum requirements:

Material	Compressive Strength
Solid units	1000 psi
Hollow units	600 psi (gross)
Mortar	700 psi

Also eliminated from the new correlation were walls with a slenderness ratio of less than 6; walls with h/t ratio less than 6 were considered to be in the category of "prisms." For evaluation of slenderness reduction criteria, only axially loaded walls were used. The data that was available consisted of tests on 159 axially loaded walls with h/t ratio ranging between 6 and 18. With this as a starting point, the data were analyzed assuming that the parabolic slenderness reductfon function, $(1 - (\frac{h}{40t})^3)$, is valid.

Basic equation used to evaluate the test data was:

$$\frac{f_{test}}{S.F.} = C_o\, f'_m \left(1 - \left(\frac{h}{40t}\right)^3\right) \quad (1)$$

$$\frac{f_{test}}{f'_m \left(1 - \left(\frac{h}{40t}\right)^3\right)} = C_o \times S.F. \quad (2)$$

$$C_o \times S.F. = K \quad (3)$$

where

f'_m = Assumed masonry strength, net area, based on strength of units

f_{test} = Net area compressive strength of panel

S.F. = Safety factor

C_o = Strength reduction coefficient

h = Height of specimen, inches

t = Thickness of specimen, inches

Net area used in the above formulae is net area of the masonry, and does not distinguish between type of mortar bedding. In the evaluation, mortar strength was assumed to be constant and was not considered as a significant influence on wall strength.

It was determined that the objective of reasonable and safe criteria would be met if 90% of the "K" values were greater than the K value selected and gave a minimum safety factor of 3. Accordingly, the K values were listed in ascending order and the válue satisfying the above conditions was K = .610 for the 159 tests as seen from Table 3. Therefore, from equation (3):

$$C_o \times S.F. = K$$

$$C_o \times 3 = 0.610$$

$$C_o = \frac{0.610}{3} = 0.205$$

This value, 0.205, agrees very closely with the coefficient 0.20 which had been used for a number of years with reinforced masonry design.

An analysis of the safety factors present with the formula:

$$f_m = 0.205 f'_m \left(1 - \left(\frac{h}{40t}\right)^3\right)$$

indicates the following:

Safety factor greater than 3 is available in 93% of the tests; greater than 4 in 51% of the tests; greater than 5 in 15% of the tests, and greater than 6 in 5% of the tests.

In ACI 531 the factor of 0.20 was increased to 0.225. The recommended value of 0.22 for unfactored loads has factors of safety comparable to those given above. Doubling this value for the factored loads was deemed reasonable and gives a factor of safety of 1.5 for 93% of all tests performed. Although the derivation given is for unreinforced walls the same values are recommended for reinforced walls.

Based on formula (2), "K" factors were calculated for all of the test specimens as listed in the following table:

TABLE 3

Ref.	Concrete Masonry Units: Percent Solid	Concrete Masonry Units: Strength, psi, net area	Concrete Masonry Units: f'_m, psi	Mortar: Str., psi	Mortar: Bedding	Walls: h/t	Walls: Strength, psi, net f_{test}	Walls: f'_m (C)	Walls: K	Walls: S.F.
1	63	1160	980	1180	Full	6.0	750	978	.798	3.83
	63	1160	980	1180	Full	6.0	685	978	.701	3.49
	63	1160	980	1160	FS	6.0	670	978	.686	3.42
	63	1160	980	900	FS	6.0	555	978	.568	2.83
	63	1200	1000	1230	Full	6.0	860	995	.863	4.30
	63	1200	1000	730	Full	6.0	625	995	.627	3.12
	63	1200	1000	960	FS	6.0	580	995	.582	2.89
	63	1200	1000	780	FS	6.0	650	995	.652	3.25
	63	1320	1060	880	Full	6.0	1110	1055	1.050	5.25
	63	1320	1060	810	Full	6.0	970	1055	.918	4.58
	63	1320	1060	810	FS	6.0	780	1055	.738	3.69
	63	1160	980	1080	Full	6.0	800	978	.818	4.08
	63	1160	980	1080	Full	6.0	670	978	.686	3.42
	63	1810	1275	1270	Full	6.0	940	1270	.739	3.67
	63	1810	1275	1270	Full	6.0	940	1270	.739	3.67
	63	1505	1150	1670	Full	6.0	825	1145	.719	3.60
	63	1505	1150	1670	Full	6.0	820	1145	.715	3.57
	63	1240	1020	980	Full	6.0	1010	1015	.993	4.95
	63	1240	1020	980	Full	6.0	870	1015	.856	4.26
	63	1720	1230	880	Full	6.0	1035	1225	.844	4.21
	63	1720	1230	880	Full	6.0	940	1225	.766	3.81
	63	1380	1090	1730	Full	6.0	1000	1085	.920	4.58
	63	1380	1090	1730	Full	6.0	1010	1085	.930	4.63
	63	1780	1262	1870	Full	6.0	1450	1257	1.152	5.75
	63	1780	1262	1870	Full	6.0	1570	1257	1.248	6.22
	43	3300	1790	1230	Full	6.0	1560	1782	.874	4.36
	43	3300	1790	1230	Full	6.0	1720	1782	.969	4.84
	70	1645	1208	1140	Full	6.0	1000	1200	.830	4.15
	70	1645	1208	1140	Full	6.0	1220	1200	1.013	5.06
8	63	509	458	3140	Full	6.0	303	455	.664	3.30
	63	509	458	1610	Full	6.0	295	455	.646	3.21
	63	509	458	1060	Full	6.0	295	455	.646	3.21
	63	840	756	3140	Full	6.0	532	753	.706	3.52
	63	840	756	1610	Full	6.0	540	753	.716	3.58
	63	840	756	1060	Full	6.0	505	753	.670	3.33
	63	875	788	3140	Full	6.0	438	785	.558	2.79

TABLE 3 (Continued)

Ref.	Concrete Masonry Units: Percent Solid	Strength, psi, net area	f'_m, psi	Mortar: Str., psi	Bedding	Walls: h/t	Strength, psi, net f_{test}	f'_m C	K	S.F.
8	63	875	788	1610	Full	6.0	430	785	.547	2.74
	63	875	788	1060	Full	6.0	500	785	.637	3.17
	63	1080	940	3140	Full	6.0	605	936	.646	3.22
	63	1080	940	1610	Full	6.0	715	936	.763	3.81
	63	1080	940	1060	Full	6.0	765	936	.817	4.07
	63	1230	1015	3140	Full	6.0	1160	1010	1.146	5.70
	63	1230	1015	1610	Full	6.0	1000	1010	.988	4.92
	63	1230	1015	1060	Full	6.0	1110	1010	1.097	5.46
	63	1410	1105	3140	Full	6.0	1140	1100	1.030	5.16
	63	1410	1105	1610	Full	6.0	985	1100	.893	4.45
	63	1410	1105	1060	Full	6.0	1030	1100	.935	4.66
	63	1520	1157	3140	Full	6.0	660	1152	.572	2.85
	63	1520	1157	1610	Full	6.0	740	1152	.642	3.20
	63	1520	1157	4780	Full	6.0	830	1152	.719	3.58
	63	1860	1295	3140	Full	6.0	1476	1290	1.143	5.70
	63	1860	1295	1610	Full	6.0	1539	1290	1.192	5.94
	63	1860	1295	1060	Full	6.0	1365	1290	1.058	5.27
	63	2510	1554	3140	Full	6.0	1698	1550	1.096	5.47
	63	2510	1554	1610	Full	6.0	1365	1550	.881	4.39
	63	2510	1554	1060	Full	6.0	1325	1550	.856	4.27
	63	3030	1710	3140	Full	6.0	2222	1705	1.304	6.50
	63	3030	1710	1610	Full	6.0	2222	1705	1.304	6.50
	63	3030	1710	1060	Full	6.0	1984	1705	1.164	5.80
	63	3740	1923	3140	Full	6.0	1857	1918	.969	4.82
	63	3740	1923	1610	Full	6.0	2523	1918	1.316	6.56
	63	3740	1923	4780	Full	6.0	2317	1918	1.209	6.03
	63	6640	2400	3140	Full	6.0	3587	2392	1.499	7.48
	63	6640	2400	1610	Full	6.0	3856	2392	1.612	8.04
	63	6640	2400	4780	Full	6.0	5031	2392	2.102	10.49
12**	100	1383	1257	2562	Full	7.0	1140	1254	.910	4.13
	100	1388	1640	3017	Full	7.0	1358	1635	.830	4.57
	100	1892	1853	2317	Full	7.0	1469	1846	.795	4.52
	100	1923	1630	2153	Full	7.0	1394	1625	.858	4.29
	100	2508	2390	2427	Full	7.0	1947	2380	.817	4.56
	100	2529	2630	2347	Full	7.0	2151	2620	.820	4.68
	100	2545	2130	2143	Full	7.0	1930	2120	.909	4.17
	100	2610	2220	3195	Full	7.0	2078	2210	.939	4.71
	100	2678	2030	2322	Full	7.0	1832	2020	.905	3.99
	100	4474	2210	2792	Full	7.0	1810	2200	.821	4.10
	100	4474	2540	2154	Full	7.0	2157	2530	.937	4.09

** f'_m values from this reference were determined from prism tests instead of assumed values. Test results multiplied by factor of 1.2.

TABLE 3 (Continued)

Ref.	Concrete Masonry Units			Mortar		Walls				
	Percent Solid	Strength, psi, net area	f'_m, psi	Str., psi	Bedding	h/t	Strength, psi, net f_{test}	f'_m C	K	S.F.
5	62	2547	1556	1400	FS	9.0	1241	1540	.807	4.05
	62	1886	1305	1400	FS	9.0	1153	1290	.894	4.50
	62	1999	1350	1400	FS	9.0	967	1335	.724	3.63
	62	1499	1150	1400	FS	9.0	685	1135	.603	3.02
	62	1934	1325	1400	Full	9.0	1354	1310	1.033	5.19
	62	2305	1473	1400	FS	9.0	1096	1455	.752	3.78
	62	2136	1405	1400	FS	9.0	1128	1390	.812	4.07
	62	1773	1260	1400	FS	9.0	1088	1245	.873	4.38
	62	1298	1049	1400	FS	9.0	854	1037	.823	4.14
	62	1241	1031	1400	FS	9.0	685	1010	.678	3.41
	62	1612	1196	1400	FS	9.0	991	1180	.838	4.20
	62	1805	1273	1400	FS	9.0	1088	1260	.864	4.33
	62	1491	1146	1400	FS	9.0	854	1133	.754	3.78
	62	1088	944	1400	FS	9.0	629	933	.673	3.38
	62	1918	1318	1400	FS	9.0	1072	1302	.822	4.12
	62	1169	985	1400	FS	9.0	605	975	.621	3.12
	45	2655	1598	1400	FS	9.0	989	1578	.626	3.15
	62	1088	944	1400	FS	9.0	564	933	.604	3.03
	62	1290	1045	1400	FS	9.0	701	1032	.678	3.41
	62	1999	1350	1400	FS	9.0	1104	1335	.826	4.16
	62	1862	1296	1400	Full	9.0	1378+	1280	1.075	5.44+
	62	- 967	870	1400	Full	9.0	758	860	.881	4.42
	62	1967	1338	1400	Full	9.0	1241	1320	.938	4.72
5	57	2280	1463	1400	FS	9.3	1228	1450	.849	4.27
	67	1917	1318	1400	FS	9.3	836	1302	.642	3.23
	67	1380	1090	1400	FS	9.3	724	1078	.672	3.37
	67	1902	1312	1400	FS	9.3	1223	1300	.943	4.74
	67	1246	1023	1400	FS	9.3	739	1010	.731	3.67
	57	2037	1386	1400	FS	9.3	1193	1370	.871	4.38
	57	2087	1386	830	FS	9.3	1298	1370	.948	4.76
	57	2385	1505	1400	FS	9.3	719	1485	.484	2.44
	57	2385	1505	1400	FS	9.3	789	1485	.530	2.67
	57	2385	1505	1400	FS	9.3	1105	1485	.743	3.74
	57	2385	1505	1400	FS	9.3	1140	1485	.766	3.85
1	39	1590	1187	1130	Full	9.5	885	1170	.756	3.79
	39	1590	1187	1010	Full	9.5	1000	1170	.853	4.28
	39	1718	1238	1070	Full	9.5	949	1220	.777	3.89
	39	1718	1238	840	Full	9.5	910	1220	.745	3.73

TABLE 3 (Continued)

Ref.	Concrete Masonry Units: Percent Solid	Concrete Masonry Units: Strength, psi, net area	Concrete Masonry Units: f'_m, psi	Mortar: Str., psi	Mortar: Bedding	Walls: h/t	Walls: Strength, psi, net f_{test}	Walls: f'_m C	Walls: K	Walls: S.F.
1	63	1159	985	1180	Full	14.3	683	940	.726	3.62
	63	1159	985	1440	Full	14.3	690	940	.734	3.66
	63	1159	985	1440	Full	14.3	738	940	.784	3.91
	63	1159	985	1060	FS	14.3	532	940	.565	2.82
	63	1159	985	900	FS	14.3	563	940	.599	2.98
	63	1159	985	1920	FS	14.3	563	940	.599	2.98
	63	1206	1020	1230	Full	14.3	738	974	.758	3.80
	63	1206	1020	730	Full	14.3	683	974	.702	3.51
	63	1206	1020	1130	Full	14.3	746	974	.765	3.83
	63	1206	1020	960	FS	14.3	571	974	.586	2.94
	63	1206	1020	780	FS	14.3	603	974	.619	3.10
	63	1206	1020	1250	FS	14.3	595	974	.610	3.05
	63	1317	1080	880	Full	14.3	905	1030	.877	4.38
	63	1317	1080	750	Full	14.3	1063	1030	1.030	5.14
	63	1317	1080	810	Full	14.3	929	1030	.901	4.49
	63	1317	1080	1020	FS	14.3	714	1030	.692	3.45
	63	1317	1080	1020	FS	14.3	667	1030	.647	3.23
	63	1159	985	1120	Full	14.3	579	940	.616	3.07
	63	1159	985	1150	Full	14.3	635	940	.675	3.37
	63	1159	985	1080	Full	14.3	635	940	.675	3.37
	63	1810	1274	1270	Full	14.3	873	1218	.717	3.54
	63	1810	1274	940	Full	14.3	881	1218	.725	3.58
	63	1810	1274	1120	Full	14.3	817	1218	.671	3.32
	63	1508	1153	1380	Full	14.3	706	1100	.641	3.17
	63	1508	1153	1380	Full	14.3	746	1100	.677	3.34
	63	1508	1153	1670	Full	14.3	643	1100	.584	2.88
	63	1238	1025	1920	Full	14.3	833	978	.851	4.24
	63	1238	1025	980	Full	14.3	802	978	.819	4.09
	63	1238	1025	1280	Full	14.3	817	978	.835	4.16
	63	1714	1230	800	Full	14.3	1111	1172	.946	4.73
	63	1714	1230	800	Full	14.3	1127	1172	.959	4.79
	63	1714	1230	750	Full	14.3	1079	1172	.918	4.59
	63	1381	1090	1730	Full	14.3	968	1040	.930	4.64
	63	1381	1090	2200	Full	14.3	960	1040	.923	4.61
	63	1774	1245	2100	Full	14.3	1240	1190	1.043	5.21
	63	2253	1450	1230	Full	14.3	936	1385	.675	3.42
	63	2253	1450	1270	Full	14.3	920	1385	.664	3.37
	70	1643	1206	1180	Full	14.3	807	1150	.701	3.55
	70	1643	1206	1300	Full	14.3	986	1150	.857	4.33
	55	1273	1040	1220	Full	14.3	727	993	.732	3.66
	55	1273	1040	1220	Full	14.3	764	993	.770	3.84
7	100	2900	1665	1475	Full	15.0	1250	1565	.801	3.93
5	65	1746	1250	1400	Full	18.0	1100	1135	.975	4.87
	65	1246	1015	1400	Full	18.0	785	925	.850	4.25
	65	1562	1175	1400	Full	18.0	1203	1065	1.131	5.66

5.1.2 FLEXURAL COMPRESSION (Reinforced and Unreinforced)

It is assumed that masonry can develop 85% of its specified compressive strength at any section. The recommended procedure for calculating the flexural strength of a section is the working stress procedure, which assumes a triangular distribution of strain.

For normal loads an allowable stress of 0.33 f'_m has a factor of safety of 2.6 for the peak stress, which only exists at the extreme fibre of the unit and has been used in practice for many years. The recommended value for factored loads also only exists at the extreme fibre and is the value recommended in the ATC-3-06 provisions.

5.1.3 BEARING (Reinforced and Unreinforced)

These values for normal loads are taken directly from the ACI code. The value recommended for factored loads is the value recommended in the ATC-3-06 provision.

5.1.4 SHEAR (Reinforced)

Two major test programs have evaluated the shear strength on concrete block masonry walls. The first was performed by Schneider and his test results were used as the basis for developing the UBC, NCMA and ACI code allowable stresses for reinforced masonry.

A more recent and extensive test program has been performed at the University of California, Berkeley and these results will be used as a comparison with the code allowables. The test results are shown in Figure 2 and lower bound values are indicated for reinforcement taking all the shear and masonry taking all the shear. These are compared to the allowables recommended for unfactored and factored loads in Table 4.

For the unfactored loads the factor of safety varies from 2.22 to 3.0. For the factored loads the factor of safety varies from 1.20 to 1.76. The ductility indicator associated with stress levels for the factored loads is of the order of 3 which provides an added factor of safety.

Table 4: Comparison of Test Restuls and Code Allowables

Description	S	U	Test Results	$\frac{\text{Tests}}{\text{S}}$	$\frac{\text{Tests}}{\text{U}}$
Masonry Takes Shear					
M/Vd = 1	$0.9\sqrt{f'_m}$	$1.5\sqrt{f'_m}$	$2\sqrt{f'_m}$	2.22	1.33
M/Vd = 0	$2.0\sqrt{f'_m}$	$3.4\sqrt{f'_m}$	$5\sqrt{f'_m}$	2.50	1.47
Reinforcement Takes Shear					
M/Vd = 1	$1.5\sqrt{f'_m}$	$2.5\sqrt{f'_m}$	$3\sqrt{f'_m}$	2.0	1.20
M/Vd = 0	$2.0\sqrt{f'_m}$	$3.4\sqrt{f'_m}$	$6\sqrt{f'_m}$	3.0	1.76

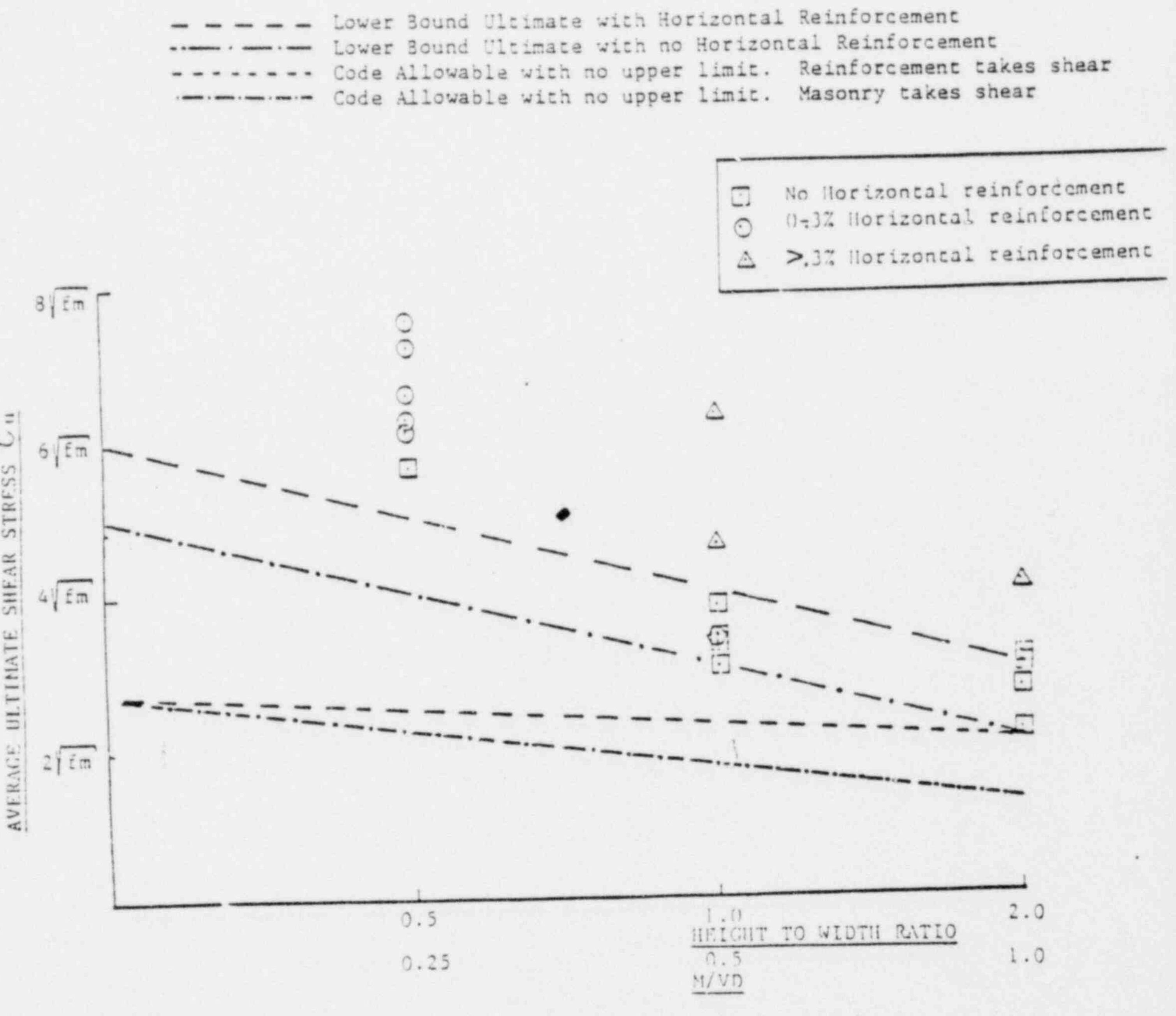

Lower Bound Ultimate with Horizontal Reinforcement
Lower Bound Ultimate with no Horizontal Reinforcement
Code Allowable with no upper limit. Reinforcement takes shear
Code Allowable with no upper limit. Masonry takes shear
No Horizontal reinforcement
0-.3% Horizontal reinforcement
>.3% Horizontal reinforcement
AVERAGE ULTIMATE SHEAR STRESS
8√fm
6√fm
4√fm
2√fm
0.5
1.0
2.0
HEIGHT TO WIDTH RATIO
0.25
0.5
1.0
M/VD

Figure 2

5.1.5 SHEAR (Unreinforced)

INTRODUCTION

The present literature on shear strength capability varies greatly on the approach used to determine acceptable values and to some extent, the controversey over these approaches and interpretation of the results. Debate, on the applicability of model or full size tests and the effects of monotonic versus cyclic loading further seems to complicate this resolution.

Much of the effort to define a permissible in-plane shear stress may be somewhat academic, in that the normal case for unreinforced walls being used in nuclear plant structures, the nature of the shear is one of being forced on the structural panel as a result of being confined by the building frame and not one of depending on the panel to transmit building shear forces. This forced drift or displacement results in shear stresses and strains, but because of the complex interaction between the panel and the confining structural elements strain or displacement is a more meaningful index for qualifying the in-plane performance of the panel. The area of in-plane strains is being addressed in another committee report.

The most extensive review on shear strength literature appears to have been done by Mayes, et al,[1] and published in Earthquake Engineering Research Center Report EERC No. 75-15 which was done for both brick and masonry block.

This report attempts to summarize some of the findings that appear to be pertinent towards defining permissible shear stress values that can be used for reevaluation of the non reinforced concrete masonry.

SUMMARY

The shear value of 0.9 $\sqrt{f'_m}$ provided by the ACI-531-79[2,3] code for reinforced masonry appear to be reasonable basis on which to proceed with the reevaluation program. This value appears to conservatively bound the actual expected shear strength of concrete block masonry. A summary of several different sources for shear stress-design values is shown by Table 5. An increase in these allowable values for the re-evaluation program of $1.35\sqrt{f'_m}$ for severe loading conditions appears warranted. Any further increase at this time without further substantiation and review is not seen as advisable.

DISCUSSION

A number of tests have been identified as being the primary basis for permissible shear stress values in both National Concrete Masonry Association (NCMA) "Specification for the Design and Construction of Load-Bearing Concrete Masonry" [4,5] and the American Concrete Institute Standard "Building Code Requirements for Concrete Masonry Structures" (ACI-531-79). [2,3] No apparent tests are traceable to the origin of the Uniform Building Code (UBC) chapter 24 on "Masonry."[6]

Those tests performed to substantiate the NCMA values are primarily performed by the National Bureau of Standards (NBS) on full size (4 ft by 8 ft, and 8 ft by 8 ft) test panels. These tests were performed by Whittemore, et al and Fishburn[10] within the period 1939 to 1961. The Whittemore tests were done, as usual in that period, utilizing a hold down detail and thereby introducing a clamping or compressive stress within the assemblage. A number of studies have shown that compressive stresses affect the shear strength significantly. The Fishburn tests, utilize a racking configuration with the testing being performed on the panel in its original laid up position. A load setting up principal tension stress causing failure is an accepted measure of shear stress determination by the American Society of Testing Material for brickwork.[11] The test results from the above references used by NCMA are shown on Table 6.

The principal tests that seem to formúlate the ACI 531 basis are the tests performed on concrete masonry piers for Masonry Research of Los Angeles, by Schneider.[12] These tests had a system for removing the compressive load on the specimen being loaded by shear and were set up to vary the a/d (M/Vd) ratio and measure this effect on a parametric basis.

The two predominant failure modes of a masonry panel under shear are diagonal tension (causing a "splitting" failure) and shear bond (causing a "joint separation" failure) or some combination of these two effects. The theory behind these were elaborated on by Yokel et al.[13] The parameter of normal stress and its effects on a shear strength, which was also reviewed by Yokel[13] and Mayes[1,14], has been demonstrated to be consequential on the determination of actual shear stress capability. This parameter is not identified, today, by any of the codes[2,4,6,15,16] shown in Table 5.

It is expected that under zero or small compressive loads the predominate shear failure will be by the shear bond mode of failure. Tests which have been done with regard to the determination of joint separation were performed by Copeland and Saxer,[17] as well as Hamid, et al.[19] These tests are, by their nature, extremely sensitive to normal stress and consequently do relate the effects of normal stress on permissable shear values. This relationship is shown on Table 5. It is of interest that there appears to be good correlation between these tests on the shear strength with zero normal stress.

The Applied Technology Council (ATC) is presently reviewing a formulation for increasing the shear stress as a function of normal stress. This formulation is developed to coincide with their present permissible shear stress of 12 psi and is consistent with the UBC's fundamental direction as a <u>design</u> code, forcing reinforcing for seismicly designed masonry structures.

As a practical matter, walls subject to the conditions of confinement will experience large compressive loads - although these are difficult to determine. Compressive loads for the most part, imparted by boundary conditions and behavior of the building frame are ignored in the evaluation of the masonry panel. If these normal stresses are added the shear resistance would be increased. This implies a conservatism on the allowable shear value when one assumes this value as chosen on the basis of zero normal stress. On this basis, and the tests results discussed, the shear value of $0.9\sqrt{f'_m}$ chosen by the ACI code appears to be justified and should be established as a reasonable basis by which to proceed with the re-evaluation.

Out of plane, or so called flexural shear is defined by the code as equalling $1.1\sqrt{f'_m}$. The derivation of this value is analogous to be permissible shear value of concrete, disregarding any reinforcement, of $1.1\sqrt{f'_c}$. Although this is somewhate different (there is no tension steel by which to determine the appropriate j distance), the actual value is a mute point since tension will be the critical value for determining out-of-plane acceptability of a flexural member.

Because of the nature of the stresses, however, and the various concerns with regard to the correctness of interpretation of the effects on boundary conditions as well as such conditions as: actual mortar properties; absorbtivity of the mortar; confinement or lack of it on the test specimen during test; arrangement and effect of actual load, it does not seem warranted to increase these stresses beyond a value of [illegible] $\sqrt{f'_m}$ (1.5 x 0.9 f'_m). This value is consistent with an adequate margin of safety for both the full panel wall test specimens referenced and the shear bond values observed by test. Any additional increase in the shear stress values for nonreinforced masonry under extreme environmental loads is not recommended at this time.

TABLE 5

SUMMARY - UNGROUTED MASONRY

	Source	Date	Shear Stress	Remarks
(1)	ACI-531 [2]	79	$0.9\sqrt{f'm} \leq 34$	M/VD $\geq$ 1
(1)	NMCA [4]	79	34 23	Type M or S Motor Type N Mortar Based on NBS tests (circa 1959-1961)
(1)	UBC [6]	79	12/10*	Type M or S/N Mortar *12 psi for solid units
(1)	ATC 3-06 [15]	78	12 $*12 + 0.20\sigma_c \leq 30$	Lightweight units limited to 85 percent shear value *being proposed for compressive stresses between 0 and 120 psi
(1)	Masonry Society [16]	Proposed	$1.0\sqrt{f'm} \leq 35$	a/l $\leq$ 1 May be increased by 0.20 c (due to dead load)
	Hamid, et al [18]	79	$76 + 1.07\sigma_c$	Ultimate value based on type S mortar
	Copeland/Saxen [17]	64	$70 + 9\sqrt{\sigma_c}$ (fitted)	Ultimate value based on 2630 compressive mortar strength

(1) Values based on inspected workmanship
σ_c = compressive stress.

TABLE 6 RACKING TEST DATA--NONREINFORCED CONCRETE MASONRY WALLS ①

Construction	Mortar Type	Ultimate Racking Load, psi, Net Mortar Bedded Area	S.F. Act./Allow	Ref.
8" Hollow Units	N	66	2.87	7
	N	58	2.52	7
	N	57	2.48	7
6" 3-Core Hollow	N	69	3.00	8
	N	62	2.70	8
	N	78	3.39	8
8" Hollow Units	N	79	3.43	10
	N	79	3.43	10
	N	73	3.17	10
	N	119	5.17	10
	N	129	5.61	10
	N	109	4.74	10
	S	132	3.88	10
	S	139	4.09	10
	S	129	3.79	10
	S	159	4.68	10
	S	132	3.88	10
	S	159	4.68	10
4-2-4 Cavity Wall of Hollow Units	M	103	3.03	9
	M	108	3.18	9
	M	102	3.00	9

Avg = 3.65

Range = 2.48 - 5.61

① From Reference 5

LIST OF REFERENCES
FOR SHEAR (Unreinforced)

1 Mayes and Clough, "Literature Survey - Compressive, Tensile, Bond and Shear Strength of Masonry," Earthquake Engineering Research Center, University of California, 1975.

2 ACI Standard, "Building Code Requirements for Concrete Masonry Structures," (ACI 531-79).

3 Commentary on "Building Code Requirements for Concrete Masonry Structures," (ACI 531-79).

4 "Specification for the Design and Construction of Load-Bearing Concrete Masonry" - NCMA - 1979.

5 Research Data and Discussion Relating to "Specification for the Design and Construction of Load Bearing Concrete Masonry" - NCMA - 1970.

6 Uniform Building Code, Chapter 24 "Masonry" - 1979.

7 Whittemore, Stang, and Parsons "Structural Properties of Six Masonry Wall Constructions," Building Materials and Structures Report No. 5., NBS - 1938.

8 Whittemore, Stang, and Parsons "Structural Properties of Two Buch-Concrete Block Constructions and a Concrete Block Wall Construction Sponsored by the National Concrete Masonry Association," Building Materials and Structures Report.

9 Whittemore, Stang, and Parsons, "Structural Properties of Concrete Block Cavity Wall Construction" Building Materials and Structures Report 21, NBS 1939.

10 Fishburn, "Effect of Motar Strength and Strength of Unit on the Strength of Concrete Masonry Walls," Monograph 36, NBS, 1961.

11 ASTM Standard Specification for Brick and Applicable Standard Testing Methods for Units and Masonry Assemblages - May 1975.

12 Schneider, "Shear in Concrete Masonry Piers," California State Polytechnic College, Pomona, California.

13 Yokel and Fattal "Failure Hypothesis for Masonry Shear Walls" - Journal of the Structural Division, March 1976.

14 "A State of the Art Review - Masonry Design Criteria" - Computech - 1980.

15 "Tentative Provisions for the Development of Seismic Regulations for Buildings" - Applied Technology Council Chapter 12 A - ATC 3-06-1978.

16 The Masonry Society Standard Building Code Requirements for Masonry Construction, First Draft.

17 Copeland and Saxer, "Tests of Structural Bond of Masonry Mortars to Concrete Block" - Journal of the Structural Division - November 1964.

18 Hamid, Drysdale, and Heidebrecht, "Shear Strength of Concrete Masonry Joints," Journal of the Structural Division - July 1979.

5.1.6 TENSION (Unreinforced)

A. Normal to the Bed Joint

~~A summar~~y of the static monotonic tests performed to determine code allowable stress for tension normal to the bed joint was given in the NCMA Specifications.

Stresses for tension in flexure are related to the type of mortar and the type of unit (hollow or solid). Research used to arrive at allowable stresses for tension in flexure in the veritcal span (i.e. tension perpendicular to the bed joints) consisted of 27 flexural tests of uniformly-loaded single-wythe walls of hollow units. These monotonic tests were made in accordance with ASTM E 72. Table 7 summarizes the test results.

From Table 7 the average modulus of rupture for walls built with Types M and S mortar is 93 psi on net area. For Type N mortar, the value is 64 psi. Applying a safety factor of four (4) to these values results in allowable stresses for hollow units as follows:

Mortar Type	Allowable Tension in Flexure
M&S	23 psi
N	16 psi

These values are consistent with those published in the 1970 ACI Committee 531 Report and which have been only slightly altered in ACI 531-79 Code.

Based upon these tests the minimum factors of safey for each mortar type are:

Mortar Type	Factor of Safety
M	3.87
S	2.60
N	2.81

To establish allowable tensile stresses for walls of solid units, the 8-inch composite walls in Table 8 were used. These walls, composed of 4-inch concrete brick and 4-inch hollow block, were greater than 75% solid, and thus were evaluated as solid masonry construction. Modulus of rupture (gross area) for these walls averaged 157 psi, giving an allowable stress of 39 psi when a safety factor of 4 is applied. The composite wall tests in Table 8 used Type S mortar. To establish allowable stresses for solid units with Type N mortar, the mortar influence established previously for hollow units was used:

$$\frac{23}{16} : \frac{39}{f} ; f = 27 \text{ psi}$$

The minimum factor of safety for these tests for Type S mortar was 2.33.

Recent dynamic tests have been performed at Berkeley and the values of tension obtained at cracking at the mid-height of the walls are as follows: 13 psi; 20 psi; 23 psi; 27 psi.

The recommended values have a factor of safety of 2.8 with respect to the lower bound of the static tests for the unfactored loads and are towards the lower limit of the initiation of cracking for the dynamic tests. An increase of 1.67 appeared reasonable for factored loads based on the static tests.

TABLE 7
FLEXURAL STRENGTH—SINGLE WYTHE WALLS OF HOLLOW UNITS—
UNIFORM LOAD—VERTICAL SPAN

Mortar Type Proportion ASTM C 270	Modulus of Rupture psi, Net Area	Reference
M	110	10
M	108	NCMA
M	102	10
M	97	10
M	95	NCMA
S	94	NCMA
M	91	NCMA
M	89	NCMA
N	88	4
S	84	10
S	83	NCMA
S	81	10
S	75	NCMA
S	69	NCMA
N	67	4
N	62	4
S	60	10
N	58	4
N	45	4
O	60	10
O	41	4
O	36	4
O	36	4
O	33	4
O	32	4
O	30	10
O	27	4

TABLE 8 FLEXURAL STRENGTH, VERTICAL SPAN CONCRETE MASONRY WALLS FROM TESTS AT NCMA LABORATORY

ASTM Mortar Type*	Wall: Nominal Thickness in.	Max. Uniform Load psf.	Net Section Modulus in 3/ft	Modulus of Rupture: Gross Area, psi	Modulus of Rupture: Net Mortar Bedded Area, psi
Monowythe Walls of Hollow Units					
M	8	85.15	80.97	61.74	88.73
M	8	87.10	80.97	63.15	90.76
M	8	91.00	80.97	65.97	94.82
M	8	103.35	80.97	74.93	107.69
S	8	62.40	80.97	45.24	69.47
S	8	72.15	80.97	52.31	75.18
S	12	183.3	164.64	57.11	93.94
S	12	161.2	164.64	50.22	82.62
Composite Walls of Concrete Brick & Hollow CMU					
S	8	222.3	103.82	161.16	180.67
S	8	219.7	103.82	159.29	178.55
S	8	187.2	78.16	135.72	202.09
S	8	228.8	103.82	165.88	185.95
S	8	218.4	78.16	158.34	235.77
S	8	223.6	78.16	162.11	241.38
S	12	171.6	139.83	53.46	103.55
S	12	150.8	139.83	46.98	91.00
S	12	156.0	139.83	48.60	94.14
S	12	213.2	139.83	66.42	128.66
Cavity Walls					
S	10	98.8	50.36	158.62	165.55
S	10	156.0	50.36	250.44	261.38
S	10	88.4	48.16	141.91	154.88
S	10	119.6	50.36	192.01	200.40
S	10	114.4	50.36	183.66	191.68
S	10	109.2	48.16	175.30	191.32
S	12(4-4-4)	145.6	50.36	233.73	243.94
S	12(4-4-4)	145.6	50.36	233.73	243.94
S	12(6-2-4)	125.2	77.80	127.38	146.63
S	12(6-2-4)	119.6	77.80	112.68	129.70

* [illegible]

B. Tension Parallel to Bed Joints

Values for allowable tension in flexure for walls supported in the horizontal span are established by doubling the allowables in the vertical span. While it is recognized that flexural tensile strength of walls spanning horizontally is more a function of unit strength than mortar, it is conservative to use double the vertical span values. Table 9 lists a summary of all published tests and indicates an average safety factor of 5.3 for the 43 walls containing no joint reinforcement and 5.6 for the 15 walls containing joint reinforcement.

It is important to note that the factor of safety for those walls loaded at the quarter points, Reference (6), have an average factor of safety of 2.02 with a minimum value of 1.22, while those loaded at the center had an average factor of safety of 6.08 with a minimum value of 3.59. However, it should be noted that the values tested at the ¼ points were also tested at 15 days.

The results associated with the early date of testing and the use of quarter point loading are difficult to explain other than to state they are at variance with all other test results.

An increase in the allowable by a factor of 1.67 is recommended for factored loads. The committee believes that the recommended values could be increased because of the larger factors of safety in the test results; however the value of 1.67 was chosen to be compatible with the increase in other stresses for unreinforced masonry.

The values recommended for stack bonded construction although at variance with current building codes (which allow zero) are thought to be reasonable values for a reevaluation program. In a test program performed by PCA[1] a horizontally spanning stack bonded wall had $^1/_3$ the capacity of an equivalent wall laid in running bond. The recommended values are in accordance with this test data. i.e. two-thirds of the value normal to the bed joint is equivalent to $^1/_3$ the values recommended for parallel to the bed joint.

Reference:

1) Portland Cement Association, "Load Tests of Patterened Concrete Masonry Walls, " Trowel Talk an aid to Masonry Industry, 1963.

TABLE 9 FLEXURAL STRENGTH, HORIZONTAL SPAN, NONREINFORCED CONCRETE MASONRY WALLS

Construction	Mortar Type	Loading Type	Loading psf	Modulus of Rupture Net Area, psi	S.F. Act./Allow	Ref.
Monowythe 8" Hollow, 3-Core	N	Uniform	127	1[illegible]2	4.13	4
	N	"	136	141	4.41	4
	N	"	127	132	4.13	4
	N	"	169	176	5.50	4
	N	"	173	180	5.63	4
	O	"	123	128	4.00	4
	O	"	158	164	5.13	4
Monowythe 8" Hollow, Joint Reinf. @ 16 in.cc	N	"	149	155	4.84	4
	N	"	160	166	5.19	4
	N	"	193	201	6.28	4
	O	"	150	156	4.88	4
	O	"	186	193	6.03	4
Monowythe 8" Hollow Joint Reinf. @ 8 in.cc	N	"	203	211	6.59	4
	N	"	196	204	6.38	4
	O	"	202	210	6.56	4
	O	"	195	203	6.34	4
Monowythe 8" Hollow	N	1/4 pt	56	58	1.81	6
	N	"	38	39	1.22	6
	N	"	61	63	1.97	6
	N	"	60	62	1.94	6
	N	"	69	71	2.22	6
	N	"	93	96	3.00	6
8" Monowythe Hollow, 2-Core	M	Center	199	217	4.72	26
	M	"	176	192	4.17	26
	M	"	151	165	3.59	26
4-2-4 Cavity Wall, Hollow Units	M	"	111	210	4.57	26
	M	"	135	255	5.54	26
	M	"	95	180	3.91	26
8" Monowythe Hollow 2-Core Joint Re. @ 8"oc	M	"	159	173	3.76	26
	M	"	159	173	3.76	26
	M	"	191	208	4.52	26
4-2-4 Cavity of Hollow Units Tied w/Joint Re. @ 8"oc	M	"	159	300	6.52	26
	M	"	159	300	6.52	26
	M	"	159	300	6.52	26

TABLE 9 (Continued)

Construction	Mortar Type	Loading Type	Loading psf	Modulus of Rupture Net Area, psi	S.F. Act./Allow	Ref.
4" Hollow Monowythe	N	Center	138	365	11.41	25
	N	"	157	415	12.97	25
	N	"	101	268	8.38	25
8" Hollow Monowythe	M	"	268	202	4.39	25
	M	"	314	237	5.15	25
	M	"	314	237	5.15	25
8" Hollow Monowythe	N	"	277	210	6.56	25
	N	"	314	237	7.41	25
	N	"	314	237	7.41	25
8" Hollow Monowythe	O	"	259	195	6.09	25
	O	"	277	210	6.56	25
	O	"	277	210	6.56	25
8" Hollow Monowythe	M	"	268	202	4.39	25
	M	"	297	224	4.37	25
	M	"	277	210	4.56	25
8" Hollow Monowythe	N	"	277	210	6.56	25
	N	"	259	195	6.09	25
	N	"	297	224	7.00	25
8" Hollow Monowythe	O	"	360	271	8.45	25
	O	"	297	224	7.00	25
	O	"	268	202	6.31	25
12" Hollow Monowythe	N	"	352	142	4.44	25
	N	"	314	127	3.97	25
	N	"	333	134	4.19	25

5.1.7 SHEAR AND TENSILE BOND STRENGTH OF MASONRY COLLAR JOINT

The collar joint shear and tensile bond strength is a major factor in the behavior of multi-wythe masonry construction, particularly with respect to weak axis bending. A widely stated position is that for composite construction the collar joint must be completely filled with mortar. However, even if this joint is filled, there must be a transfer of shearing stress across this joint without significant slip in order for full composite interaction of the multiple wythes to be realized. Since the cracking strength, moment of inertia, and ultimate flexural strength, of the wall cross section are significantly influenced by the interaction of multiple wythes, it is crucial to establish the collar joint shear bond strength.

The only applicable published data on the shear bond strength of collar joints is that determined by Bechtel on the Trojan Nuclear Power Plant. A number of 3/[illegible] collar joints were tested and the accepted NRC allowable for the shear bond strength was 12 psi. Based on this information 12 psi is the recommended value for factored loads.

There is conflicting data available on the relationship between the shear and tensile bond strengths. In most tests performed on mortar bed joints (couplet tests) the shear bond strength was approximately twice the tensile bond strength. In a more recent method of evaluation by means of centrifugal force the shear bond strength was found to be 60% of the tensile bond strength. The authors of the report consider the test procedure to be an improvement over present methods since joint precompression is essentially eliminated as a result of the testing procedure.

Because of the conflict in the test data the committee recommended that the values for tensile bond strength be the same as for shear bond.

Unless metal ties are used at closely spaced intervals (less than 16 inches on center) it is recommended that their contribution to shear and tensile bond strength be neglected.

Reference:

(1) Hatzinkolas, M., Longworth, J., and Wararuk, J., "Evaluation of Tensile Bond and Shear Bond of Masonry by Means of Centrifugal Force," Alberta Masonry Institute, Edmonton, Alberta.

5.1.8 BOND (reinforced)

Values for bond stress are taken directly from the ACI Code. Due to the sensitivity of workmanship, degradation under cyclic load and the implications of a bond mode of failure it is recommended that these values be increased by 33 1/3% for factored loads.

5.1.9 GROUT CORE TENSILE STRESS

The tensile value recommended for the grout core tensile stress is taken from ACI 318 for concrete with a factor of safety of three. An increase of 1.67 was deemed reasonable for the factored loads.

5.2 DAMPING

The damping values for unreinforced walls are based on judgment and include a differentiation for the OBE and SSE force levels. This is based on the premise that damping increases as the stress level increases.

The damping values for reinforced walls are based on the accepted values for reinforced concrete.

There is no test data available in the literature to validate or refute these damping values.

6.0 ANALYSIS AND DESIGN

6.1 STRUCTURAL RESPONSE OF UNREINFORCED WALLS.

6.1.1 OUT OF PLANE EFFECTS

The steps given in this section provide a logical conservative evaluation methodology to determine the stress levels in a masonry wall subjected to out of plane forces. The first two steps provide a lower bound estimate on the frequency of the wall since it assumes the wall spans in only one direction. For a wall with two or more sides capable of acting as boundaries the stresses resulting from one way or beam action will be conservative compared to those obtained from a more rigorous plate analysis.

If the stresses resulting from the analysis exceed the allowable stresses or the wall contains significant openings the beam analysis is not appropriate and the full effect of the actual boundary conditions must be accounted for in a plate analysis. For walls with openings it is recommended that a finite element plate analysis be performed to correctly model the effect of the opening. For walls without openings either a finite element analysis can be performed or standard test book formulae for plates may be used. If a multimode analysis is not performed it is recommended that the moments and stresses be increased by 1.05 to account for higher mode effects. Many parameter studies have been performed that indicate that in most cases the first mode of vibration contributes 98% or more to the total response of the wall. Thus the 1.05 factor is considered adequate.

6.1.2 FREQUENCY VARIATIONS OUT OF PLANE

This section acknowledges the fact that there will be variations in the frequency of the wall as a result of uncertainties in the mass of the wall and attached equipment, material and section properties and the modulus of elasticity of the masonry. The method selected to account for these uncertainties was a variation in the modulus of elasticity. The range of ± 25% for ungrouted walls and ± 20% for grouted walls is conservative when coupled with the use of a smoothed spectrum. If the frequency of a wall falls on the low frequency side of the amplified region of the response spectrum adequate provisions are included to ensure that the determination of the stress in the wall is conservative.

6.1.3 IN PLANE AND OUT OF PLANE EFFECTS

The plant FSAR provides for the design of a two-direction (one horizontal and one vertical) earthquake. The provisions of this section are consistent with the FSAR. The vertical component of motion is not included in the analysis procedure because the positive effect of the dead load on bed joint stresses is not included in the evaluation criteria. It should be noted however that the effect of vertical acceleration is included in determining the pipe and equipment loads on the wall.

6.2 STRUCTURAL RESPONSE OF REINFORCED MASONRY WALLS

6.2.1 OUT OF PLANE EFFECTS

The comments in Sec. 6.1.1 are applicable to the uncracked condition of a reinforced wall. If the wall cracks in either the vertical or horizontal direction cracked section properties of the wall are used to determine the frequency of either the beam or the plate. If a plate analysis is performed an orthotropic analysis must be performed in which different section properties in the horizontal and vertical directions are used.

6.2.2 EQUIVALENT MOMENT OF INERTIA

6.2.2.1 CRACKED CONDITION

The recommended value of I_e is taken from ACI 318. The formula was developed for slender columns and was considered to be appropriate for the out of plane analysis of masonry walls. The formula was checked against the test results of Dickey and Mackintosh[1] and reasonable agreement was obtained. It should be noted that if this formula is used it should be used over the total length of the wall and not over the cracked section.

The fully cracked section moment of inertia provides a lower limit and can be used over the cracked section of the wall. It is very conservative to use it over the full length of the wall.

Reference:

(1) Dickey, W. L., and Mackintosh, A., "Results of Variation of "b" or Effective Width in Flexure in Concrete Block Panels," Masonry Institute of America, 1971.

6.2.3 FREQUENCY VARIATIONS

See Sec. 6.1.2 for comments.

6.2.4 IN PLANE AND OUT OF PLANE EFFECTS

See Sec. 6.1.3 for comments.

6.3 ACCELERATIONS

The masonry walls are analyzed in a manner similar to that of equipment and piping systems. It is therefore conservative to use the envelop of the floor level spectra to which the wall is attached. If the wall is not attached at its top, forces will be induced from the floor level of the base of the wall and this should be used in the analysis.

6.5 IN PLANE EFFECTS

Load bearing structural masonry walls shall be evaluated on an allowable stress basis. The shear stress on the wall is determined from seismic analysis of the building and evaluated as in conventional design.

The majority of the masonry walls are not intended to be primary structural elements and for the purposes of this specification a non-load bearing or non structural wall is defined as follows.

1. It does not carry a significant part of the building's story shear or moment.

2. It does not significantly modify the behavior of adjacent structural elements.

In other words, the expected behavior of the building must be substantially the same whether such walls are present or not.

In-plane effects may be imposed on these masonry walls by the relative displacement between floors during seismic events. However, the walls do not carry a significant part of the associated story shear, and their stiffness is extremely difficult to define. In addition, since the experimental evidence to date demonstrates that the apparent in-plane strength of masonry walls depends heavily upon the in-plane stress boundary conditions, load or stress on the walls is not a reasonable basis for an evaluation criteria.

However, examination of the test data provided by the list of references for this section indicates that the gross shear strain of walls is a reliable indicator for predicting the onset of significant cracking. A significant crack is considered to be a crack in the central portion of the wall extending at least 10% of a wall's width or height. Cracking along the interface between a block wall and steel or concrete members does not limit the integrity of the wall, and is not addressed here. The gross shear strain is defined to be:

$$\gamma = \frac{\Delta}{H}$$

where: γ = strain
Δ = relative displacement between top and bottom of wall
H = height of wall

Test results indicate that to predict the initiation of significant cracking, masonry walls must be divided into two categories:

1. Unconfined Walls - not bounded by adjacent steel or concrete primary structure. Significant "confining" stresses cannot be expected.

2. Confined Walls - at a minimum, bounded top and bottom or bounded on three sides.

For unconfined concrete block masonry walls the works of Fishburn (2) and Becica (1) yield an allowable shear strain as defined above of 0.0001. It should be noted that Fishburn's test specimens were 15 days old, on average.

For confined walls, the most reliable data appears to be that of Mayes et al (4). In static and dynamic tests of masonry piers (confined top and bottom) varying block properties, mortar properties, reinforcement, vertical load and grout conditions, significant cracking was initiated at strains exceeding about $\gamma = 0.001$. It should be noted here that reinforcement can have no significant effect on the behavior prior to cracking. Similarly, the presence of cell grout should have no effect on stress or cracking in the mortar joints at a given strain. Both predictions are confirmed by the data in reference (4). In addition, the data shows that the onset of cracking is not sensitive to the magnitude of initial applied vertical load.

Klingner and Bertero (3) performed a series of cyclic tests to failure and found excellent correspondence with a non-linear analysis in which the behavior of an infilled frame prior to cracking is determined by an equivalent diagonal strut. While the equivalent strut technique has been used by many investigators to study the stiffness and load-carrying mechanisms of infilled frames, Klingner and Bertero found that the quasi-compressive failure of the strut could be used to predict the onset of significant cracking.

After some simplification of the relations in reference (3), the strength of the strut corresponds to a strain at cracking

$$= \frac{1 + \left(\frac{B}{H}\right)^2}{1000B/H} \qquad (1)$$

in which

B = wall width

H = wall height

assuming $E = 1000f'_m$

In summary, the recommended value for permissible in plane strain for service loads in unconfined walls is:

$\gamma_u = 0.0001$

and in confined walls

$\gamma_c = 0.001$

For factored loads these strains may be increased by 1.67.

For non-load bearing walls that are subjected to both in plane shear stresses and interstory drift effects the combination equation specified limits the combined effect such that the sum of the proportion of stress induced by each is less than 1. The complexity of this type of loading has not been validated by tests and the procedure recommended is deemed reasonable.

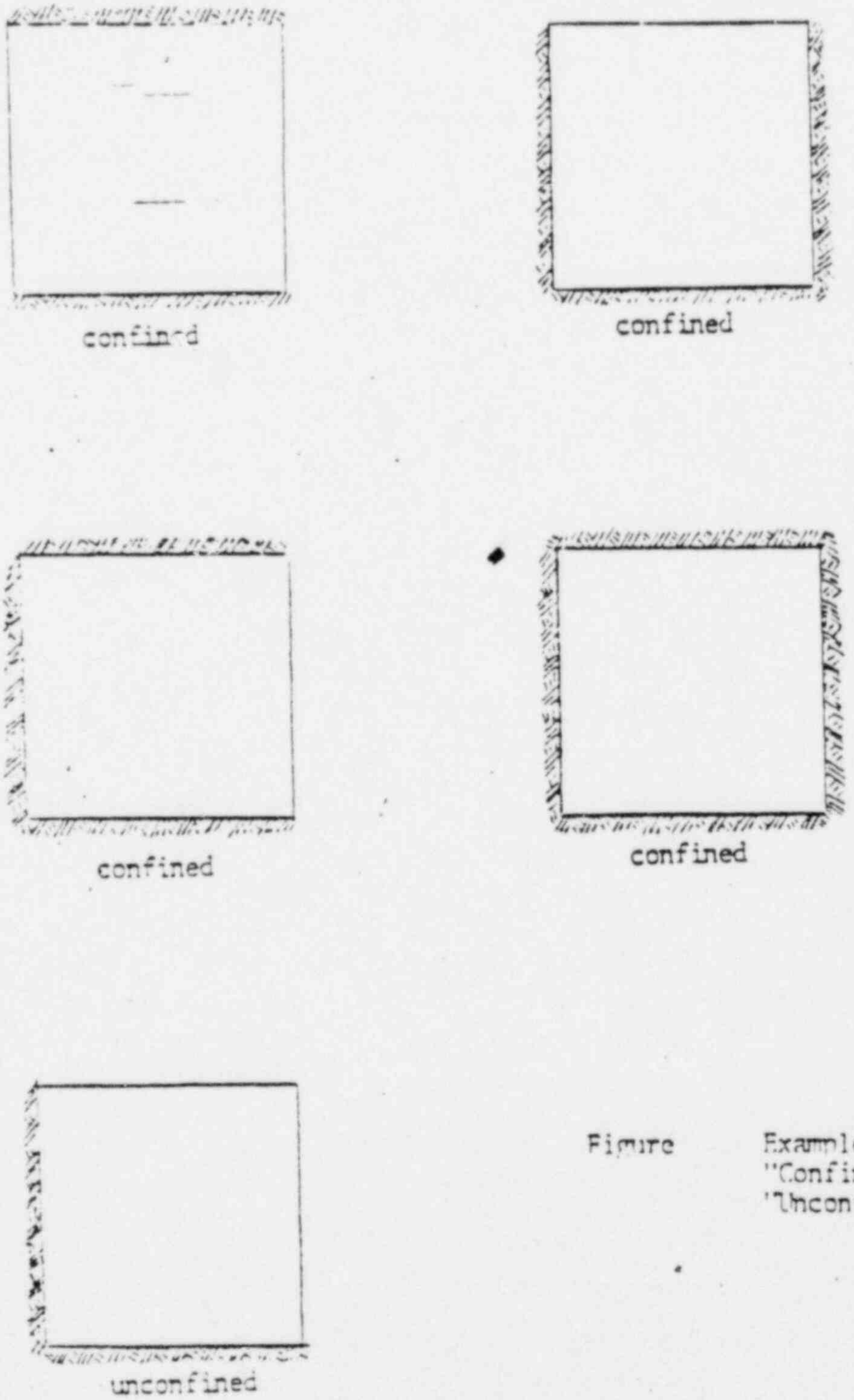


Figure Examples Defining "Confined" and, "Unconfined" Walls

REFERENCES

1. Becica, I.J. and H.G. Harris, "Evaluation of Techniques in the Direct Modeling of Concrete Masonry Structures, " Drexel University Structural Models Laboratory Report No. M77-1, June 1977.

2. Fishburn, C.C. "Effect of Mortar Properties on Strength of Masonry," National Bureau of Standards Monograph 36 U.S. Government Printing Office, Nov. 1961.

3. Klingner, R.E. and V.V. Bertero, "Earthquake Resistance of Infilled Frames," Journal of the Structural Division, ASCE, June 1978.

4. Mayes, R.L., Clough, R.W., et al, "Cyclic Loading Tests of Masonry Piers," 3 volumes; EERC 76/8, 78/28, 79/12 Earthquake Engineering Research Center, College of Engineering University of California, Berkeley, California.

5. Benjamin, J.R. and H.A. Williams, "The Behavior of One-Story Reinforced Concrete Shear Walls," Journal of the Structural Division, ASCE, Proceedings, Paper 1254, Vol. 83, No. ST3, May 1957, pp. 1254.1-1254.39.

6. Benjamin, J.R. and H.A. Williams, "The Behavior of One-Story Brick Shear Walls," Journal of the Structural Division, ASCE, Proceedings, Paper 1723, Vol. 84, ST4, July, 1958, pp. 1723.1-1723.30.

7. Benjamin, J.R. and H.A. Williams, "Behavior of One-Story Reinforced Concrete Shear Walls Containing Openings," Journal of the American Concrete Institute, Proceedings, Vol. 30, No. 5, November, 1958, pp. 605-618.

8. Holmes, M., "Steel Frames with Brickwork and Concrete Infilling," Proceedings of the Institution of Civil Engineers, Vol. 19, August, 1961, pp. 473-478.

9. Holmes, M., "Combined Loading on Infilled Frames," Proceedings of the Institution of Civil Engineers, Vol. 25, May, 1963, pp. 31-38.

10. Liauw, T.C., "Elastic Behavior of Infilled Frames," Proceedings of the Institution of Civil Engineers, Vol. 46, July, 1970, pp. 343-349.

11. Mallick, D.V. and R.T. Svern, "The Behavior of Infilled Frames Under Static Loading," Proceedings of the Institution of Civil Engineers, Vol. 39, February, 1968, pp. 261-287.

12. Smith, B.S., "Lateral Stiffness of Infilled Frames," Journal of the Structural Division, ASCE, Vol. 88, No. ST6, December, 1962, pp. 183-199.

13. Smith, B.S., "Behavior of Square Infilled Frames," Journal of the Structural Division, ASCE, Vol. 91, No. ST1, February, 1966, pp. 381-403.

14. Smith, B.S., "Model Test Results of Vertical and Horizontal Loading in Infilled Frames," Journal of the American Concrete Institute, Proceedings, Vol. 65, No. 8, August, 1968, pp. 618-623.

15. Smith, B.S. and C. Carter, "A Method of Analysis for Infilled Frames," Proceedings of the Institution of Civil Engineers, Vol. 44, September, 1969, pp. 31-48.

6.6 EQUIPMENT

The method specified to account for the effect of equipment is conservative. The effect of equipment mass is included in the frequency calculation of the wall and thus the inertia effect of the mass of the equipment is included in the determination of the stress in the wall. This procedure by itself may not be sufficient because it does not account for any amplification of the equipment. Thus it is recommended that the fully amplified effect of the equipment be included by applying a static load and combining the resulting stresses with the stresses from the inertia loads. The combination shall be performed by the absolute sum method.

Refinement to this procedure is permitted if the frequency of the equipment is known and the SRSS method of combining stresses can be justified.

6.7 DISTRIBUTION OF CONCENTRATED OUT OF PLANE LOADS

The criteria for distributing concentrated out of plane loads is taken from the Uniform Building Code and is applicable to both reinforced and unreinforced construction. The limitation on stresses for beam or one way action is specified to ensure that these are not lower than those obtained from plate or two way action.

The allowable stresses for block pullout are based on the shear bond strength of a block since this is the mode of failure for unconfined block pullout. The discussion given in Sec. 5.1.5 for the allowable values for unreinforced shear walls indicates that these values are in accordance with the available test data on the shear bond strength of concrete masonry.

7.0 ALTERNATIVE ACCEPTANCE CRITERIA

7.1 REINFORCED MASONRY

Reinforced masonry walls which are well anchored and supported can undergo large ductile inelastic and out of plane flexural deformations (1). An approximate analysis method of determining the out of plane inelastic seismic response is the "energy balance" technique. This analysis technique is, in essence, similar to Blume's (2) reserve energy technique and is analogous to Newmark's (3) inelastic seismic response spectrum technique.

References:

(1) Dickey, W.L. and Mackintosh, A., "Results of Variation in "b" the Effective Width in Flexural Concrete Block Panels," Masonry Institute of America, 1971.

(2) Blume, J.A., Newmark, N.M. and Corning, L.H., "Design of Multistory Reinforced Concrete Buildings for Earthquake Motions," Portland Cement Association, 1961.

(3) Newmark, N.M., "Current Trends in the Seismic Analysis and Design of High-Rise Structures," Chapter 16, Earthquake Engineering, Edited by R. L. Weigel, McGraw-Hill, 1970.

7.2 UNREINFORCED MASONRY

An extensive test program performed by Gabrielson (1) on blast loading of masonry walls provides validation of the concept of arch action of masonry walls subjected to loads that exceed those that cause flexural cracking of an unreinforced masonry wall. An analytical procedure was developed to predict with reasonable accuracy the ultimate capacity of the unreinforced walls tested. With a factor of safety of 1.5 the procedure is used to determine the ultimate or collapse capacity of masonry walls.

Reference:

(1) Gabrielson, G., Wilton, C. and Kaplan, K., "Response of Arching Walls and Debris from Interior Walls Caused by Blast Loading," URS Report 2030-23, URS Research Co., 1975.

ENCLOSURE 4

GPU Service

CALC. NO.

SHEET NO. 1 OF 5

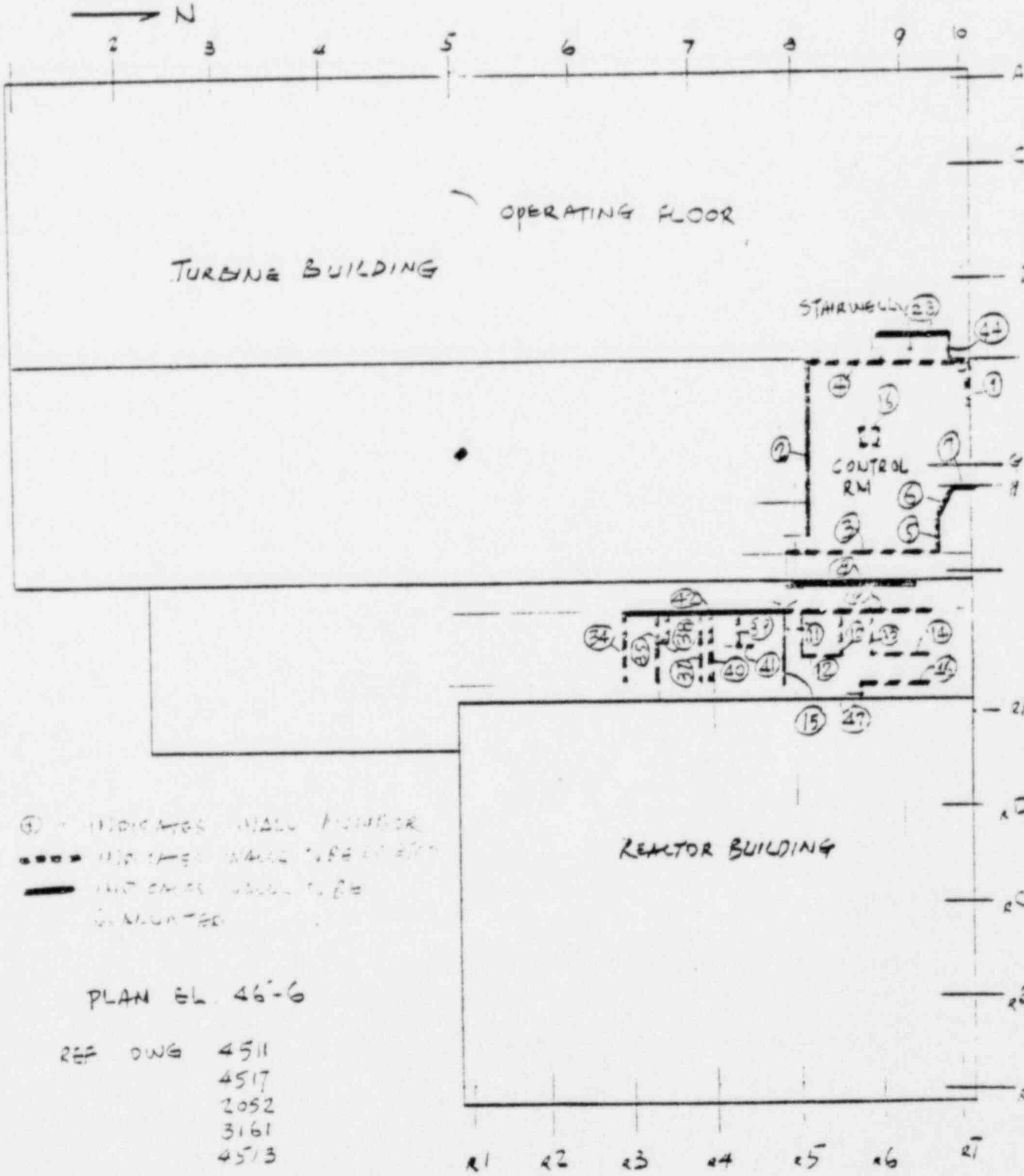
SUBJECT Oyster Creek Station - Unit #1

Plan - Masonry Wall Location

DATE

COMP. BY/DATE

CHK'D. BY/DATE A. T. / 10. 20. 80



POOR ORIGINAL

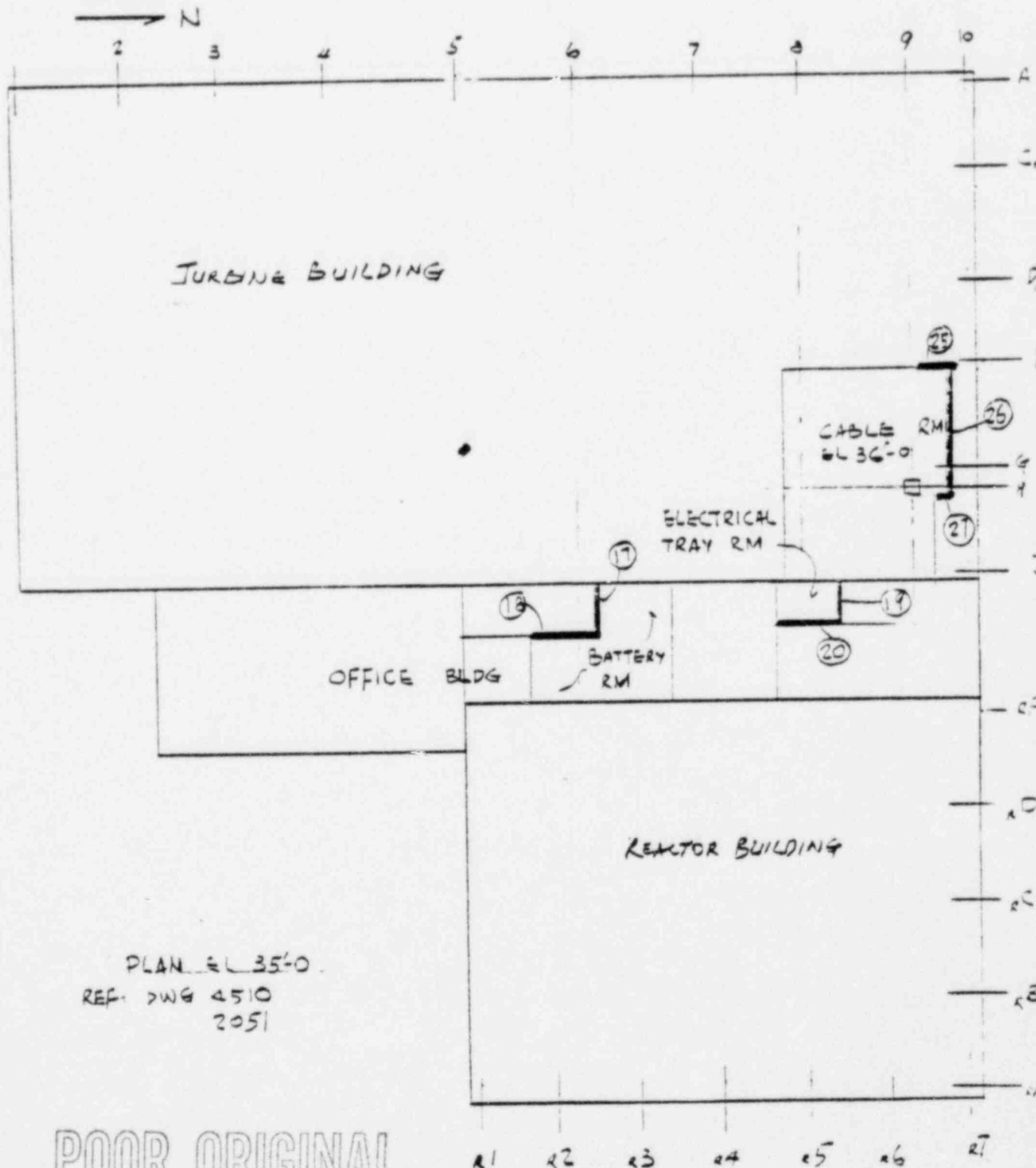
A000 0016

GPU Service

CALC. NO.
SHEET NO. 2 OF 5
DATE
COMP. BY/DATE G.W.
CHK'D. BY/DATE A.T./10.20.80

SUBJECT Oyster Creek Station - Unit #1

PLAN - MASONRY WALL LOCATION



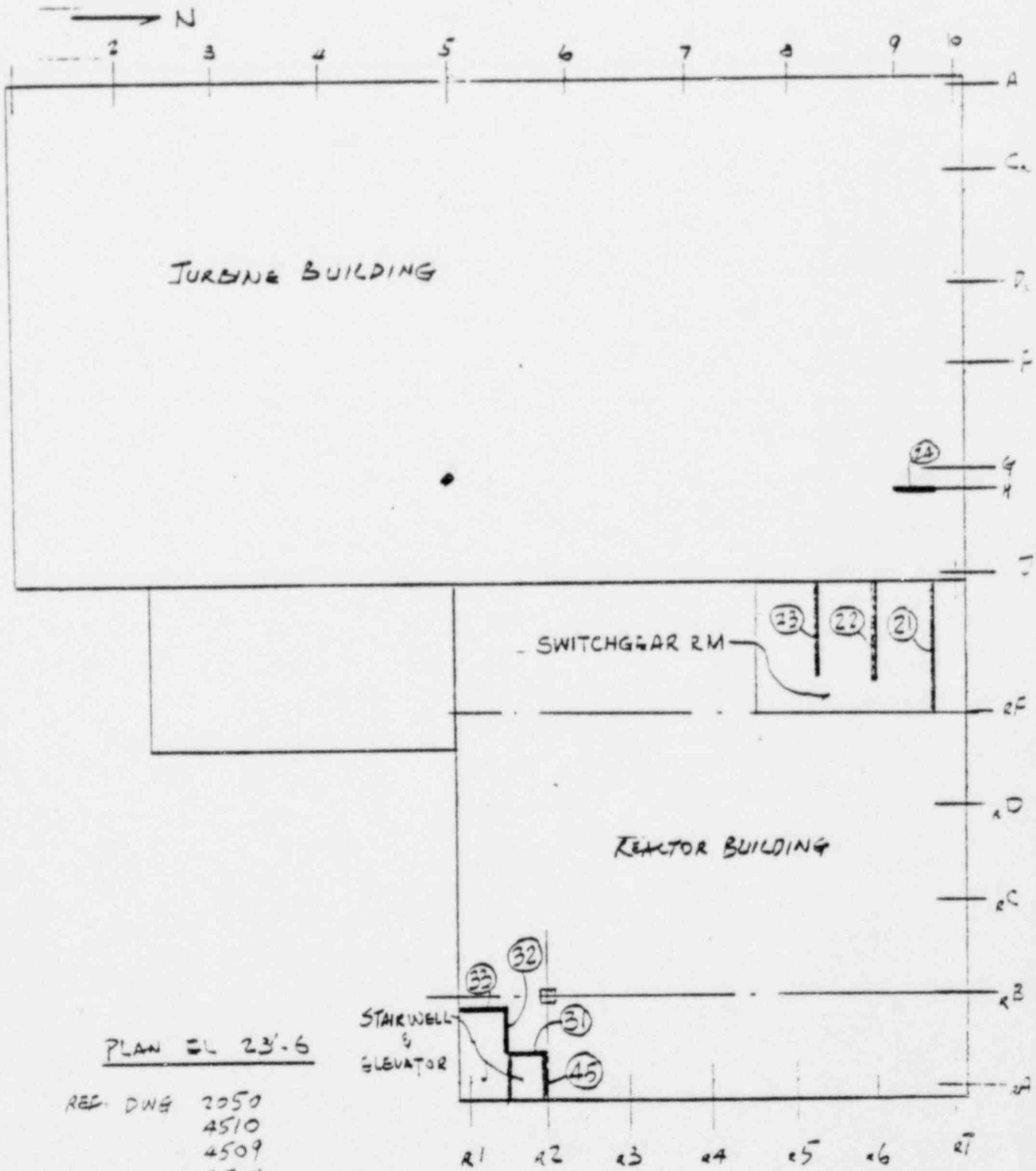
POOR ORIGINAL

A000 0016

GPU Service

CALC. NO.
SHEET NO. 3 OF 5
DATE
COMP. BY/DATE [illegible]
CHK'D. BY/DATE A.T. / 10-20-80

SUBJECT Oyster Creek Station - Unit #1
Plan - Masonry Wall Location



PLAN EL 23'-6

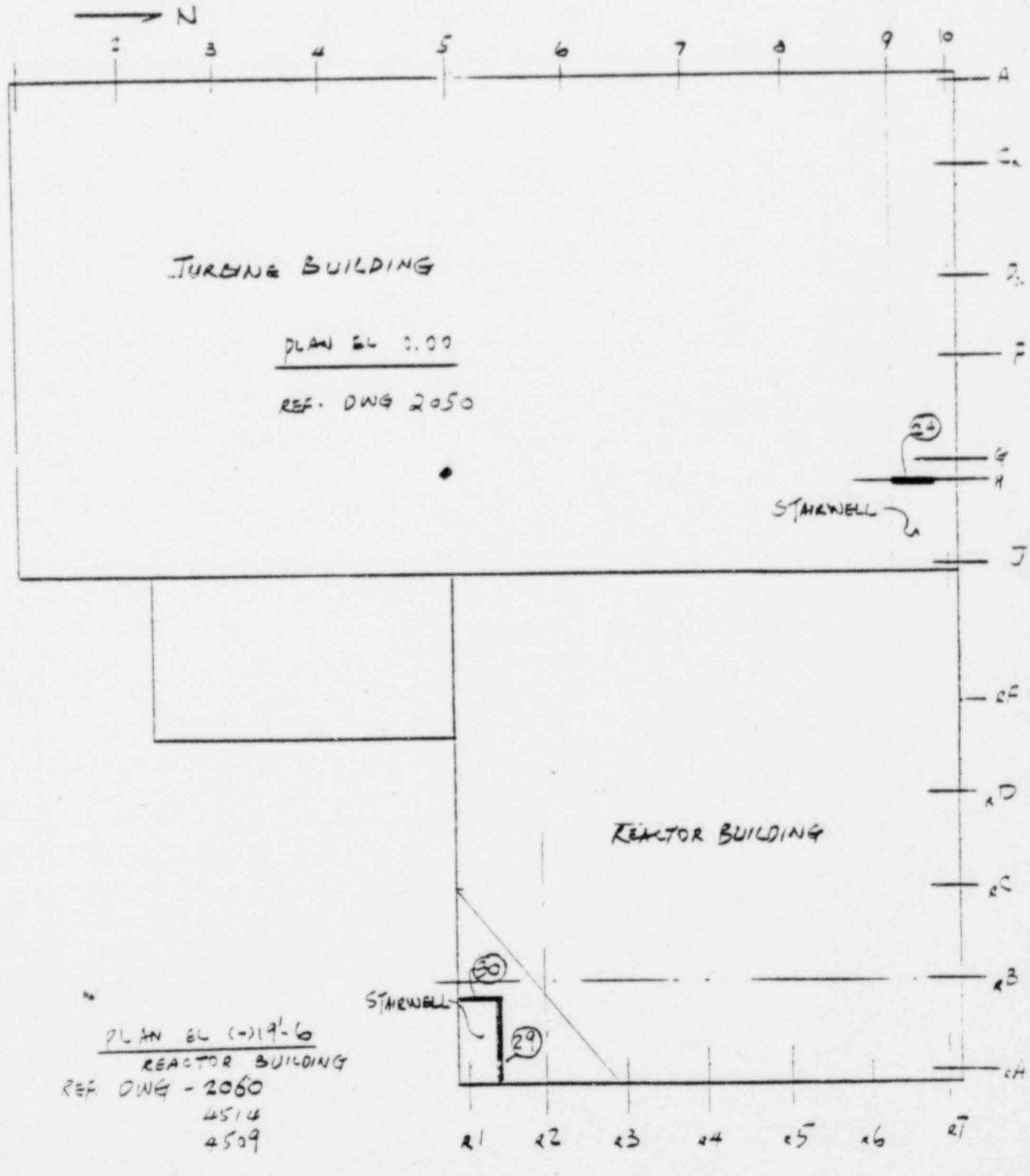
REF. DWG 2050
4510
4509
4514

POOR ORIGINAL

A000 0016

GPU Service

SUBJECT Oyster Creek Station - Unit #1	CALC. NO.
Plan - Masonry Wall Location	SHEET NO. 4 OF 5
	DATE
	COMP. BY/DATE
	CHK'D. BY/DATE A.T./10.20.80



POOR ORIGINAL

A000 0016

GPU Service

SUBJECT Oyster Creek Station - Unit #1

Plan - Masonry Wall Location

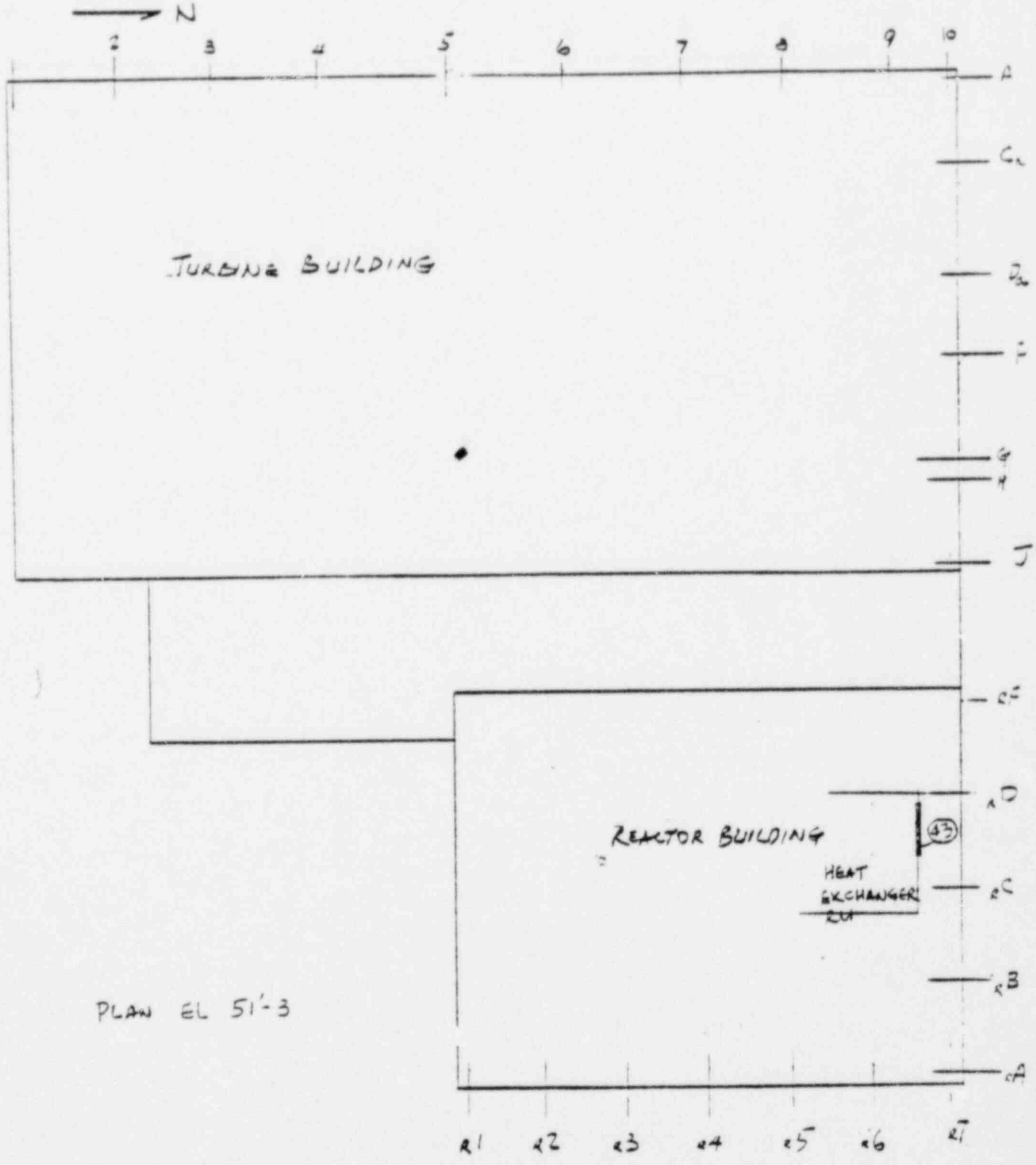
CALC. NO.

SHEET NO. 5 OF 5

DATE

COMP. BY/DATE G. Wu 10/17/80

CHK'D. BY/DATE A.T. 10.20.80



PLAN EL 51'-3

ENCLOSURE 5

GPU Service

CALC. NO.

SHEET NO. 1 OF 36

SUBJECT OYSTER CREEK STATION - UNIT No. 1

DATE

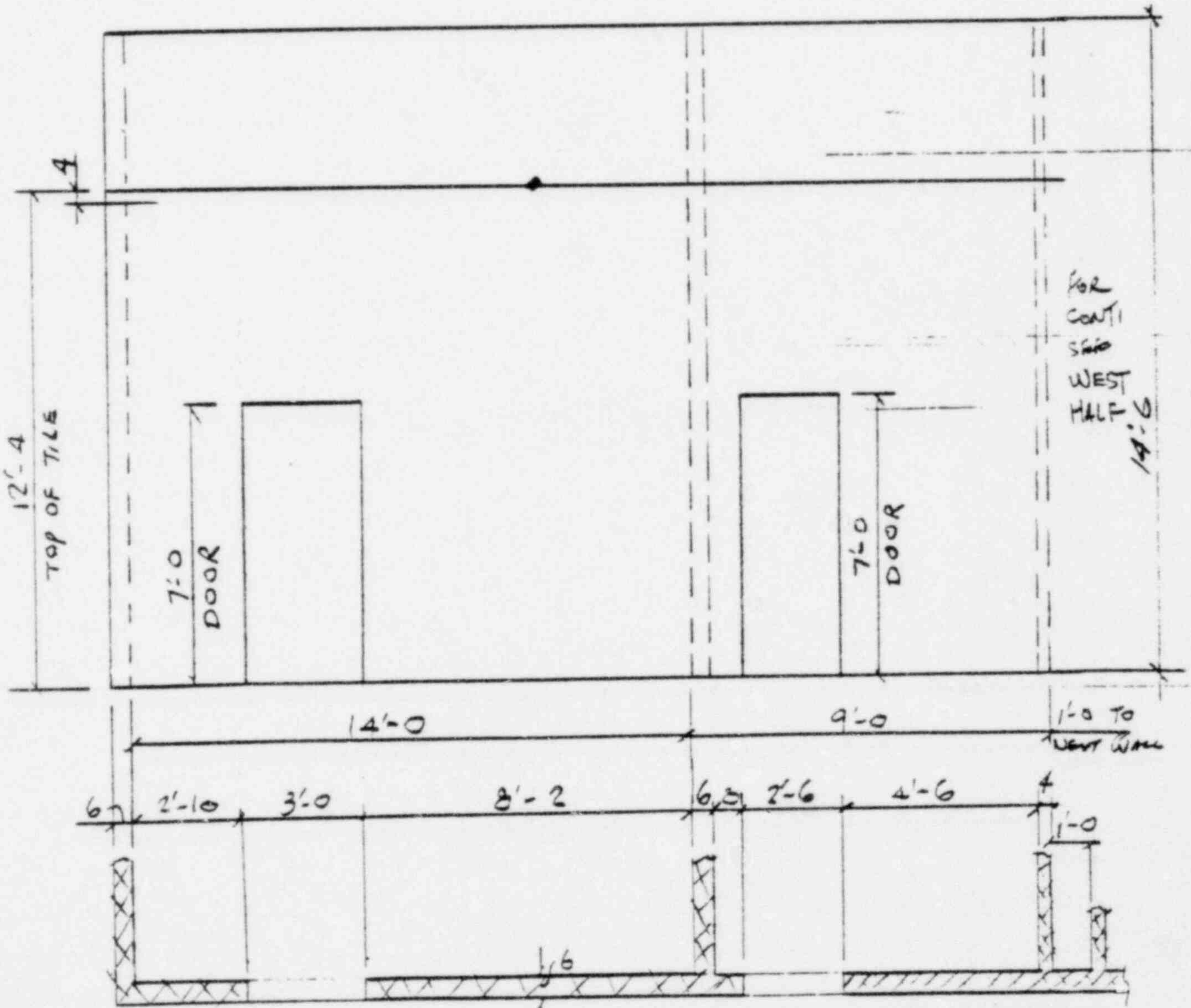
COMP. BY/DATE 10/6/80

WALL No. 2 - CONTROL RM SOUTH WALL - EAST HALF - 23'-6 LONG

CHK'D. BY/DATE AT 10.8.80

WALL THICKNESS : 6" HOLLOW GLAZED MASONRY BLOCK (STACKED)
MORTAR TYPE : M
REINFORCED : YES

REF DWG. SR 4511-6
4517-8



POOR ORIGINAL

1/4" = 1'-0

A000 0016

GPU Service

CALC. NO. ....

SHEET NO. 2 OF 36

DATE ....

SUBJECT Oyster Creek Station - Unit No. 1

COMP. BY/DATE ....

CHK'D. BY/DATE AT 10.8.80

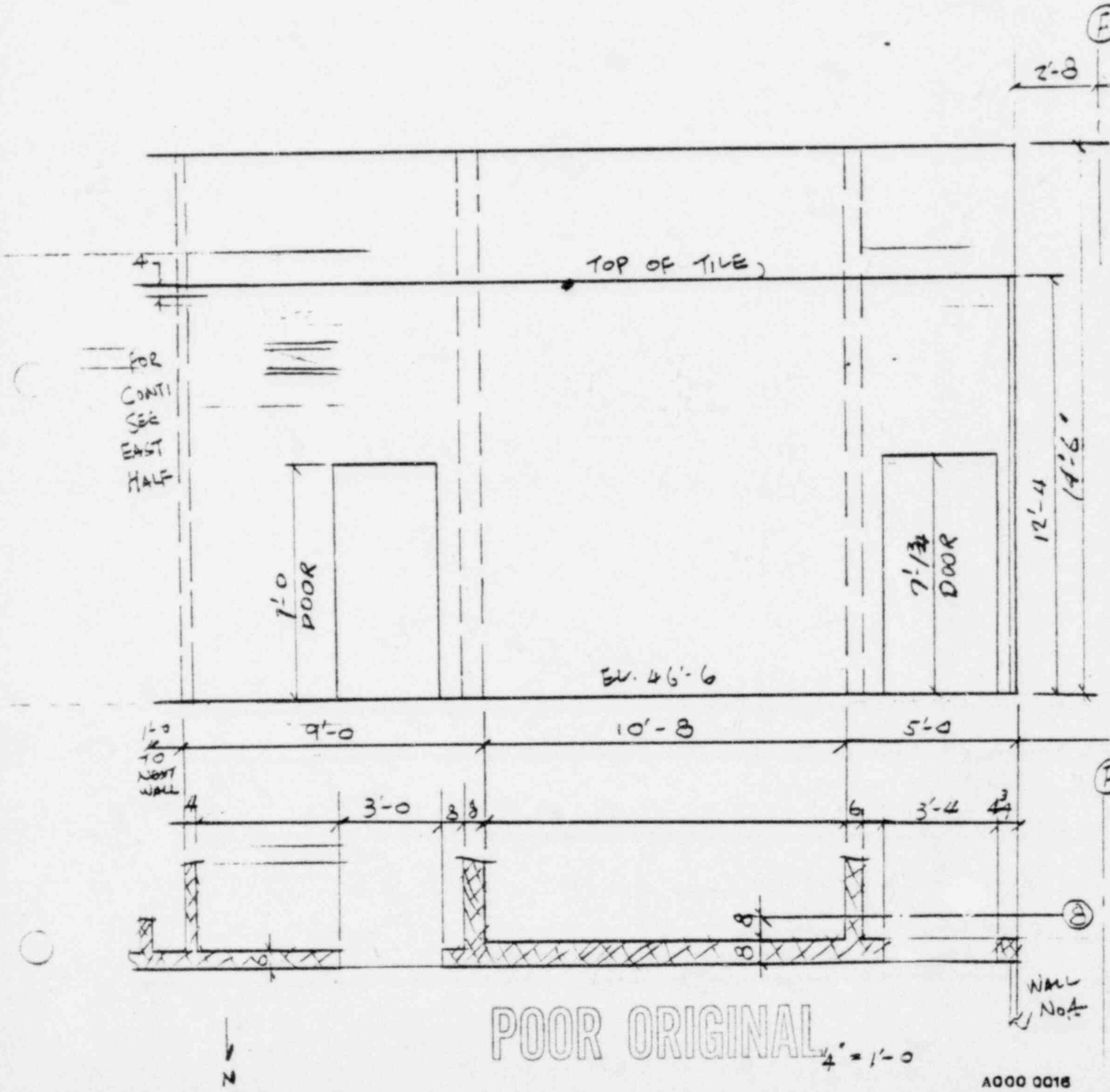
WALL NO. 2 - CONTROL RM - SOUTH WALL - WEST HALF - 25'-8 LONG

WALL THICKNESS : 6" & 8" GLAZED MASONRY BLOCK (HOLLOW)

MORTAR TYPE : M

REINFORCED : YES

REF DWG BR 4511-6, 4517-3



POOR ORIGINAL

¼" = 1'-0

A000 0016

GPU Service

CALC. NO.
SHEET NO. 3 OF 36
DATE
COMP. BY/DATE A.T. 10.7.80
CHK'D. BY/DATE 10/8/80

SUBJECT OYSTER CREEK STATION - UNIT #1

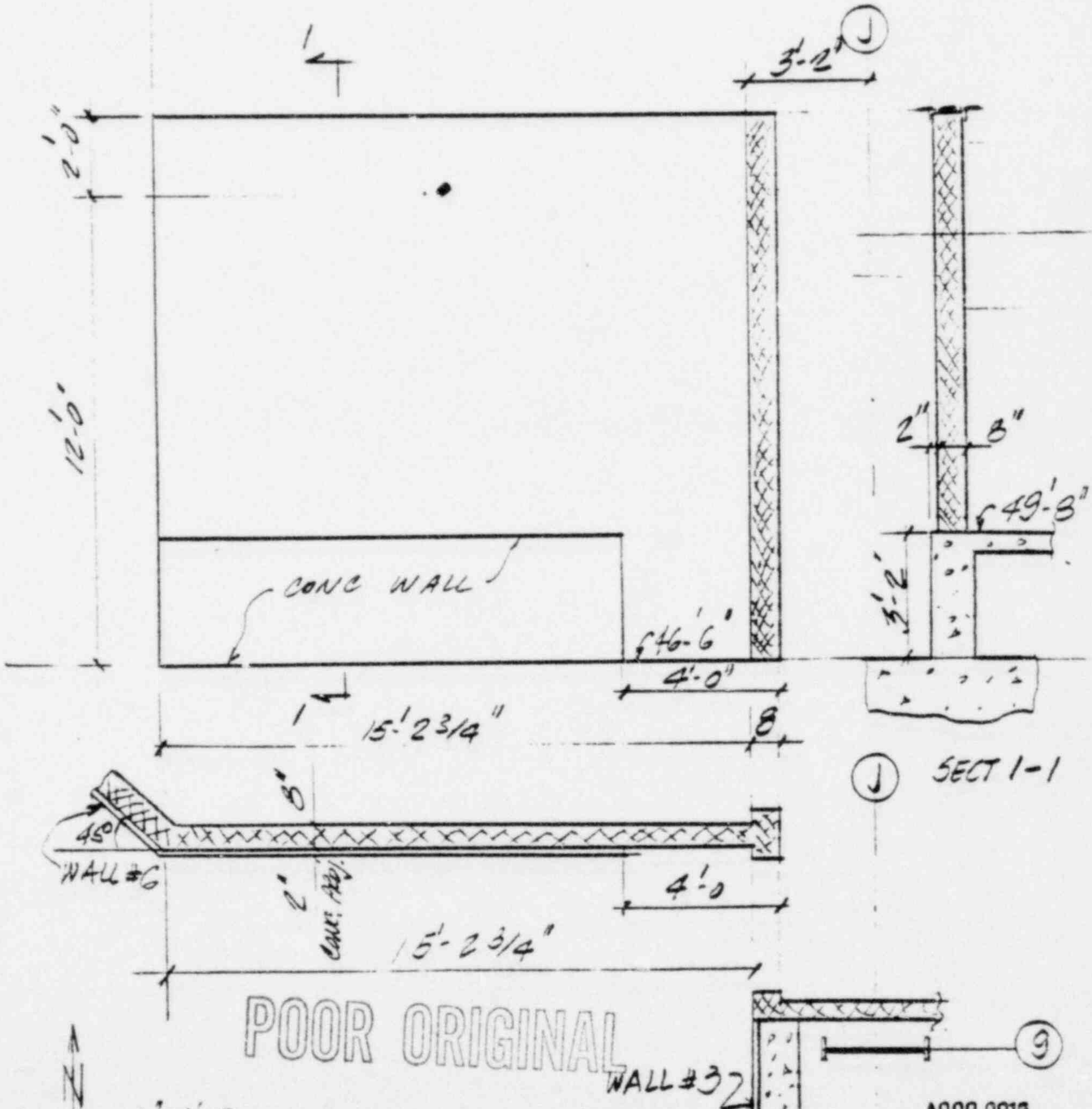
WALL # 5 - OBSERVATION RM ENCLOSURE SOUTH WALL (SECURITY / FIRE) 15'-2 3/4" LONG

WALL THICKNESS : 8" HOLLOW BLOCK WALL (STAGGERED)

MORTAR TYPE : M

REINFORCED : YES

REF. DWGS. 4511-6, 4513-3, 19430



POOR ORIGINAL

A000 0016

GPU Service

CALC. NO.

SHEET NO. 4 OF 36

SUBJECT OYSTER CREEK STATION - UNIT NO. 1

DATE

COMP. BY/DATE A.T. 10.7.80

WALL # 6 - OBSERVATION RM. ENCLOSURE S.E. TO N.W.

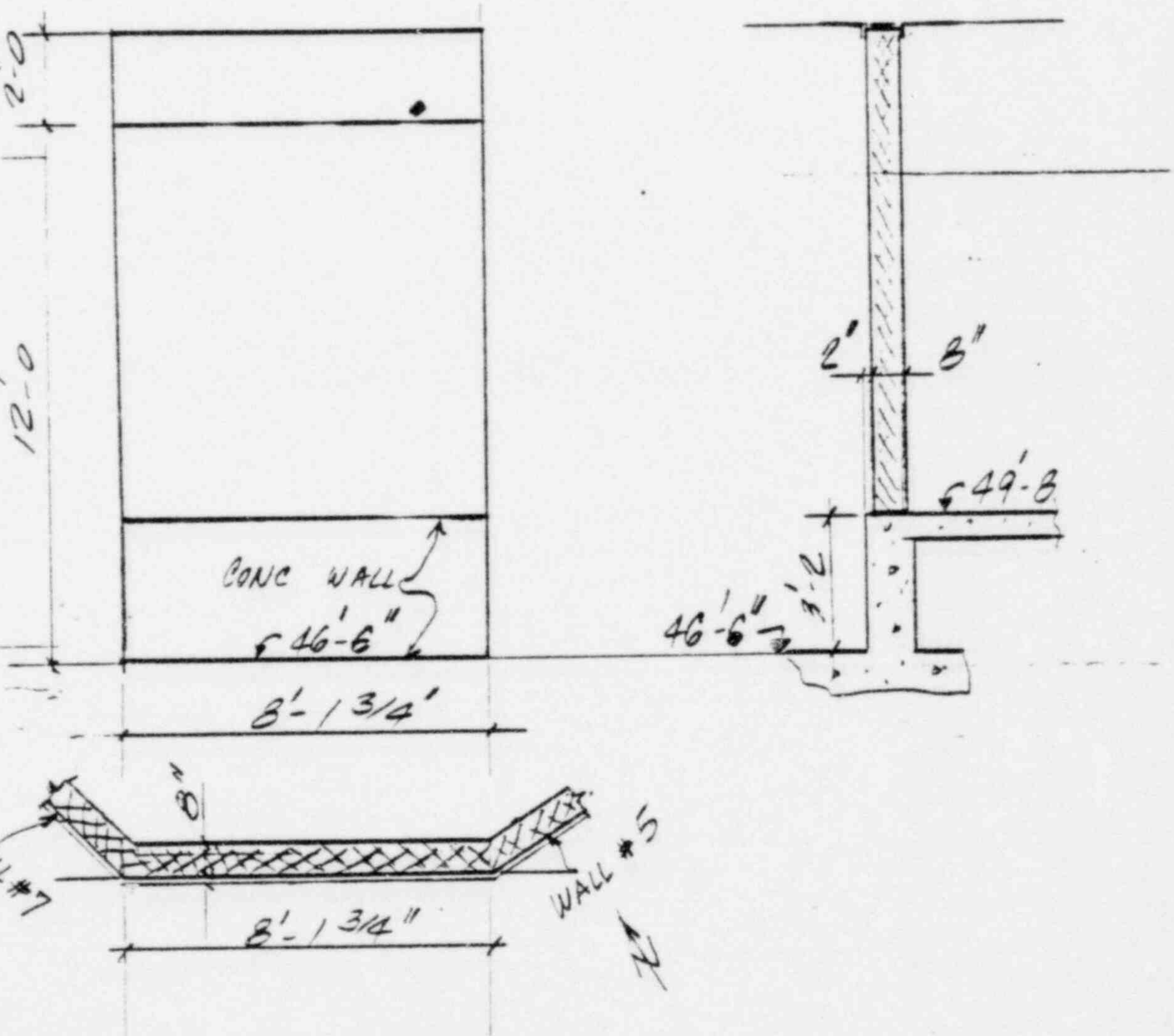
CHK'D. BY/DATE [illegible]

(SECURITY / FIRE WALL)

WALL THICKNESS : 8" HOLLOW BLOCK WALL (STAGGERED)

MORTAR TYPE : M

REINFORCED : yes. REF. DWGS. 4511-6, 4513-3, 19480



1/4" = 1'-0

POOR ORIGINAL

A000 0018

GPU Service

CALC. NO.

SHEET NO. 5 OF 36

SUBJECT OYSTER CREEK STATION - UNIT #1.

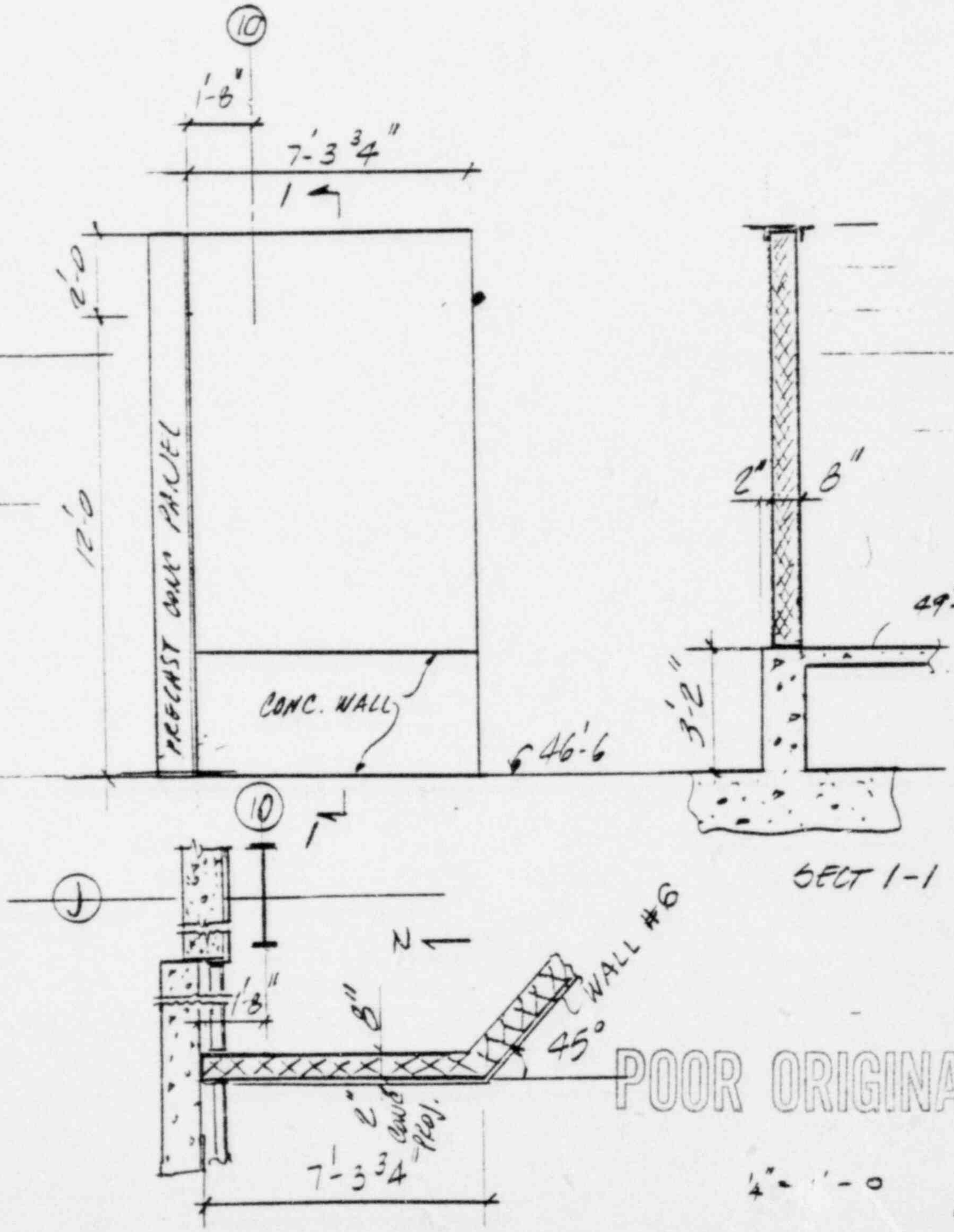
DATE

COMP. BY/DATE A.T. 10.7.80

CHK'D. BY/DATE [illegible]

WALL #7 - OBSERVATION RM ENCLOSURE - WEST WALL (SECURITY/FIRE WALL).

WALL THICKNESS : 8" HOLLOW CONC. BLOCK (STAGGERED)
MORTAR TYPE : M
REINFORCED : YES. REF DWGS. 4511-6
4513-3
19480



POOR ORIGINAL

1/4" = 1'-0

A000 0016

GPU Service

CALC. NO.

SHEET NO. 6 OF 36

DATE

COMP. BY/DATE A.T. 10.9.80

CHK'D. BY/DATE JW 10/13/80

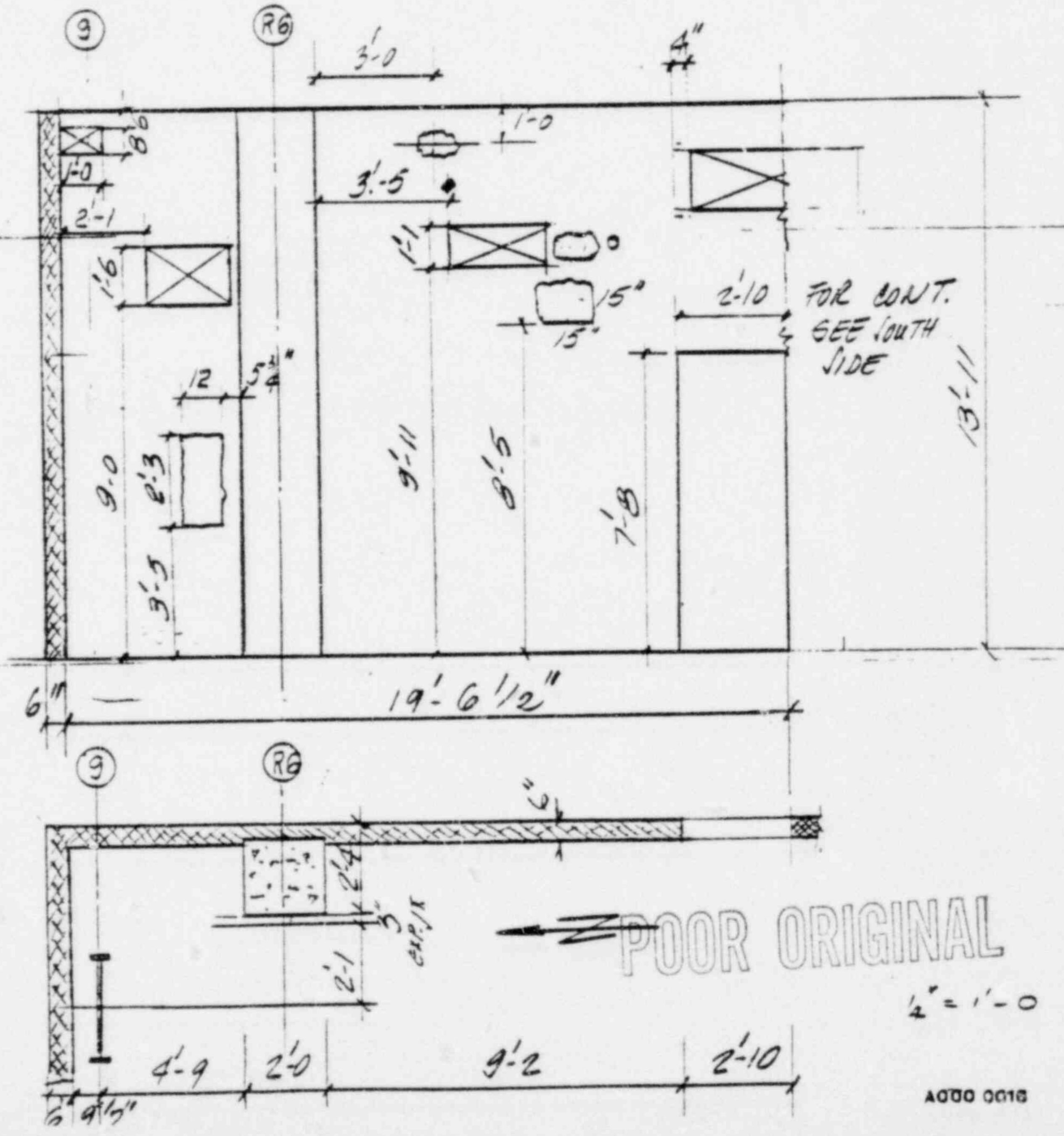
SUBJECT OYSTER CREEK STATION - UNIT #1

WALL # 8 - CABLE TRAY AREA - EAST WALL (32'-10" LONG)

WALL THICKNESS : 6" HOLLOW GLAZED CONC. MASONRY UNITS (STACKED)
MORTAR TYPE : M
REINFORCED : NO.

REF DWGS : 4511-6
4513-3

NORTH SIDE OF WALL



POOR ORIGINAL

A000 0016

GPU Service

CALC. NO. ....

SHEET NO. 7 OF 36

SUBJECT OYSTER CREEK STATION - UNIT #1.

DATE ....

COMP. BY/DATE AT 10.9.80

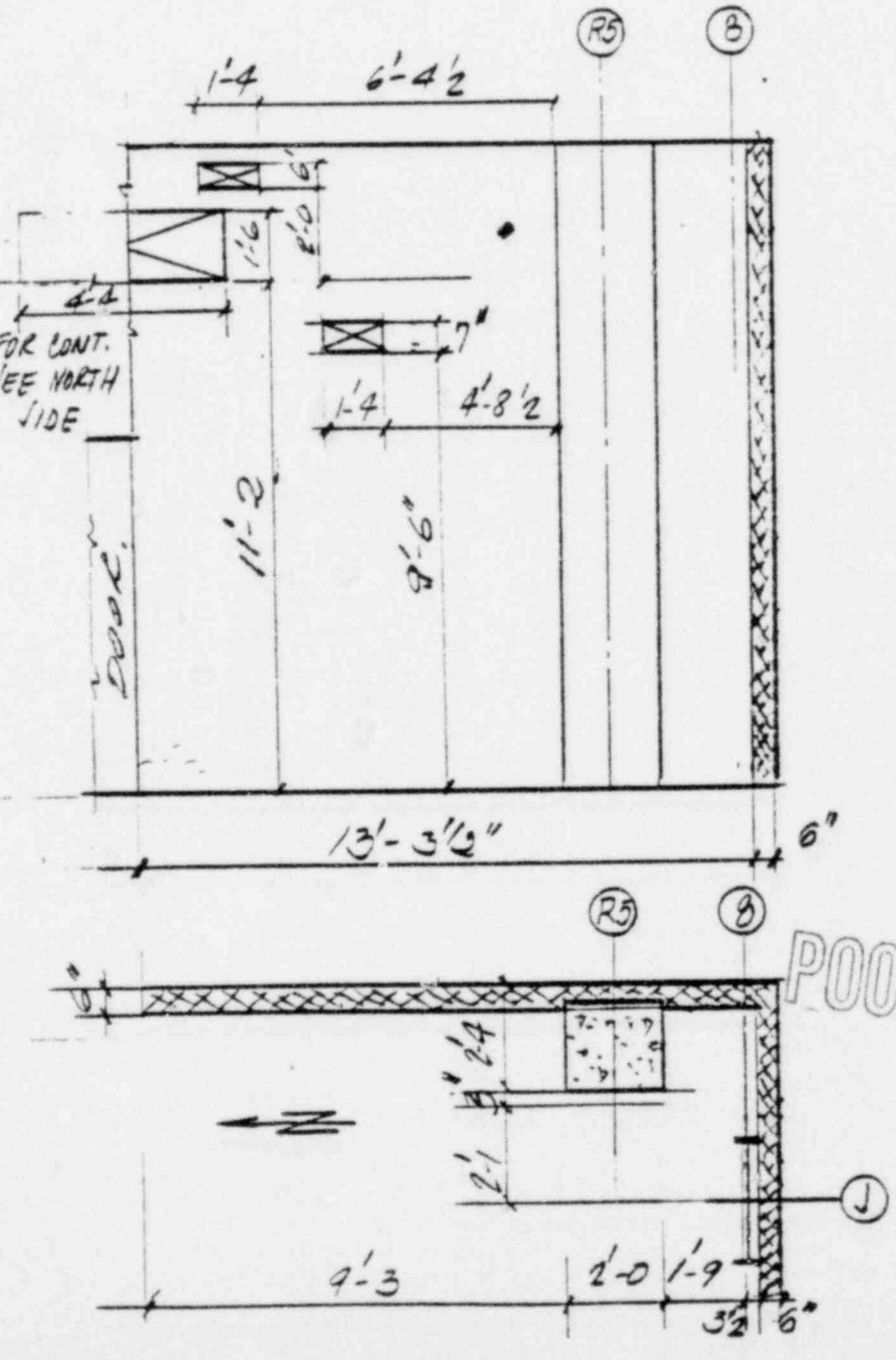
CHK'D. BY/DATE [illegible] 10.11.80

WALL # 8 - CABLE TRAY AREA - EAST WALL (32'-10" LONG)

WALL THICKNESS : 6" HOLLOW GLAZED CONC. MASONRY UNITS (STACKED)
MORTAR TYPE : M
REINFORCED : NO.

REF DWGS. [illegible]11-6
[illegible]13-3

SOUTH SIDE OF WALL



POOR ORIGINAL

¼" = 1'-0

A000 0013

GPU Service

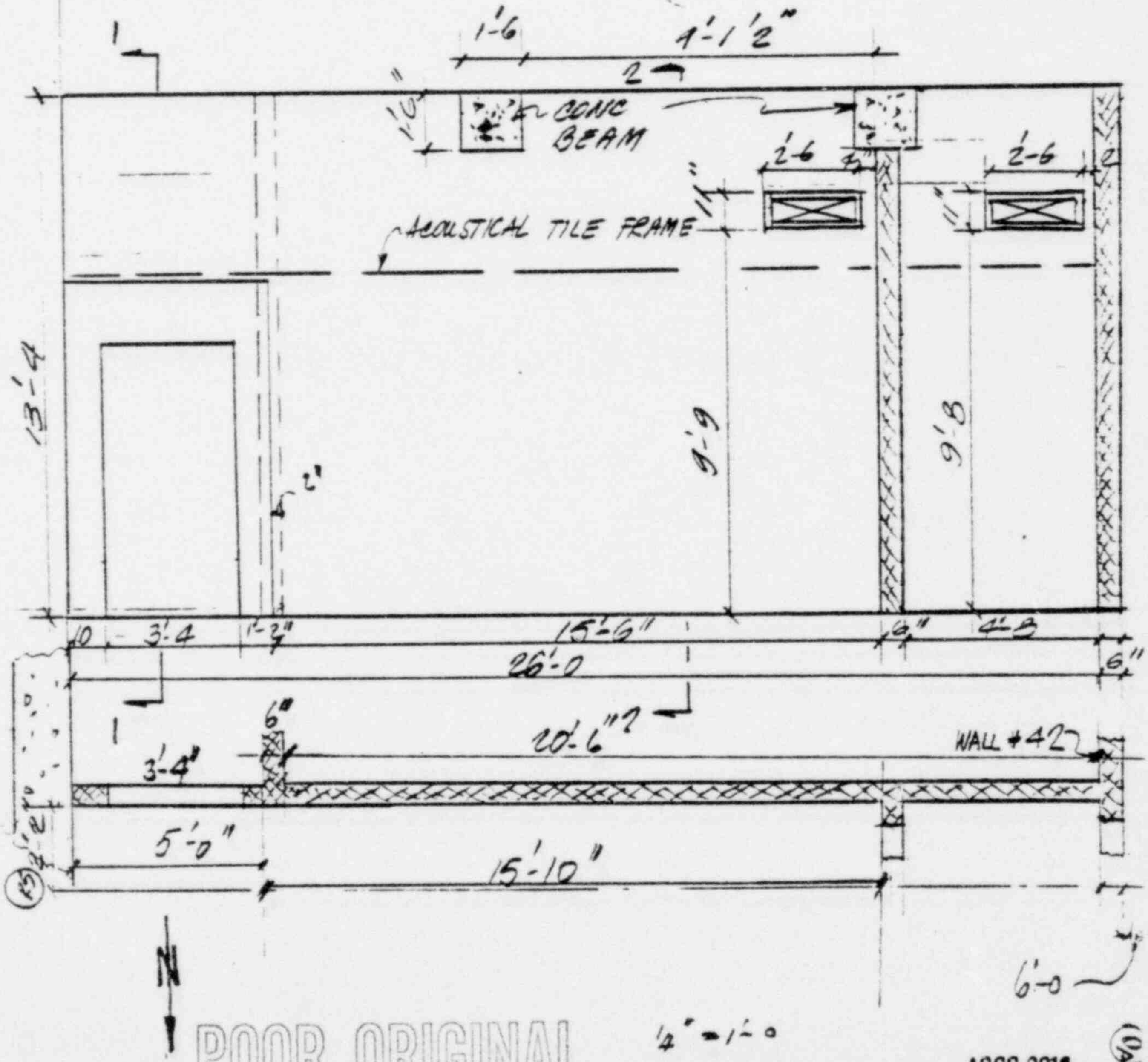
DATE
COMP. BY/DATE AT 10.9.80
CHK'D. BY/DATE 10/12/80

SUBJECT OYSTER CREEK STATION - UNIT #1

WALL # 15 - MONITOR AND CHANGE RM - SOUTH WALL 26'-0 LONG

WALL THICKNESS : 6" GLAZED MASONRY UNITS (STACKED).
MORTAR TYPE : M
REINFORCED : NO.

REF DWGS: 4511-6
4517-8



POOR ORIGINAL

1/4" = 1'-0

A000 0016

GPU Service

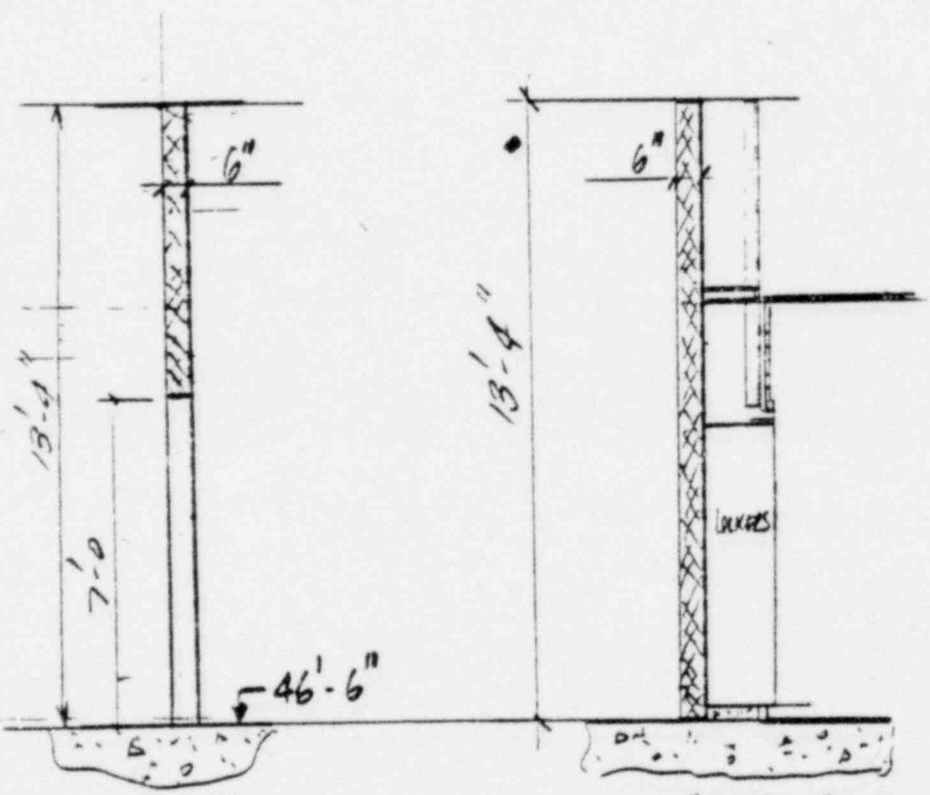
CALC. NO.
SHEET NO. 9 OF 36
DATE
COMP. BY/DATE A.T. 10.9.80
CHK'D. BY/DATE [illegible] 10/13/80

SUBJECT OYSTER CREEK STATION - UNIT #1

WALL #15 - MONITOR AND CHANGE RM. - SOUTH WALL

- WALL THICKNES 6" GLAZED HOLLOW MASONRY (STACKED)
- MORTAR TYPE M
- REINFORCED NO.

REF. DWGS: 4511-6
4517-8



SECT 1-1

SECT 2-2

$\frac{1}{4}$" = 1'-0

POOR ORIGINAL

A000 0016

GPU Service

SUBJECT OYSTER CREEK STATION - UNIT #1

WALL # 17 - BATTERY ROOM (A & B)

SOUTH WALL (WEST SECTION).

CALC. NO.

SHEET NO. 10 OF 36

DATE

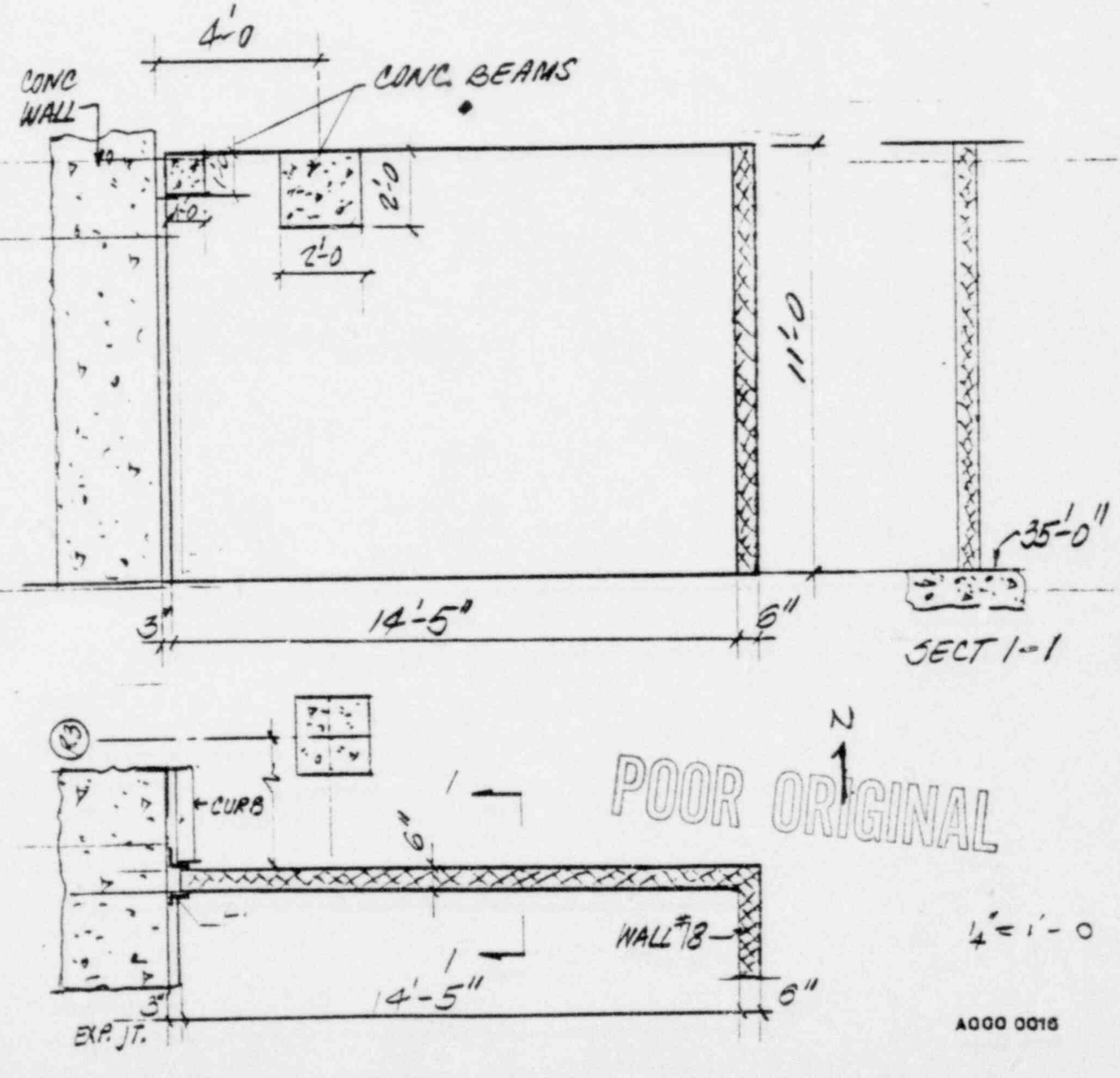
COMP. BY/DATE A.T. 10.9.80

CHK'D. BY/DATE

WALL-THICKNESS : 6" HOLLOW CONC MASONRY UNITS (STAGGERED.)

MORTAR TYPE : M

REINFORCED : NO. REF DWG. 4510-3



POOR ORIGINAL

A000 0016

GPU Service

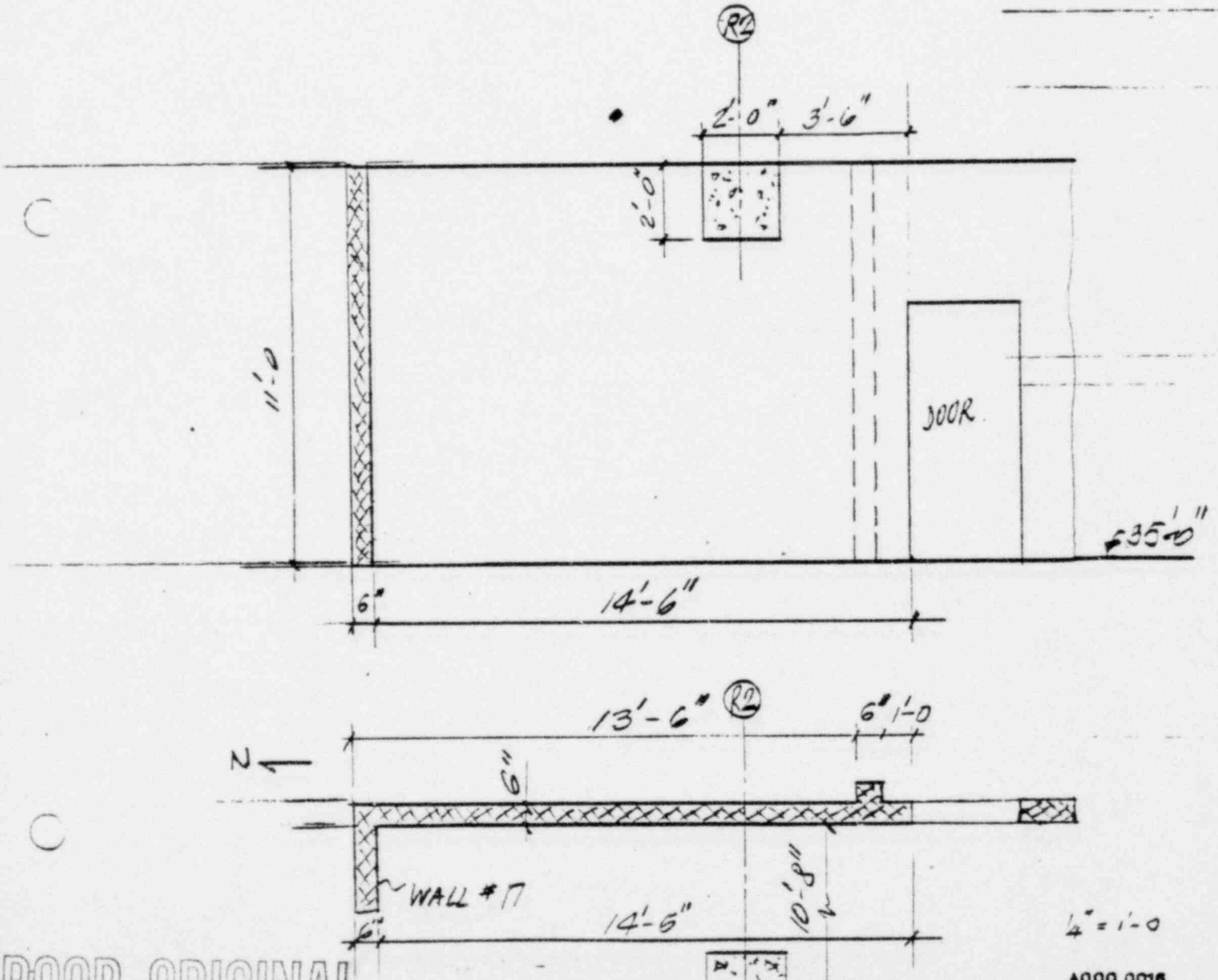
CALC. NO.
SHEET NO. 11 OF 36
DATE
COMP. BY/DATE AT 10.10.80
CHK'D BY/DATE [illegible] 10/1?/80

SUBJECT OYSTER CREEK STATION — UNIT #1

WALL #18 - BATTERY ROOM (A & B)
WEST WALL

14'-6" LONG

- WALL THICKNESS : 6" HOLLOW CONC. MASONRY ~~UNITS~~ (STAGGERED)
- MORTAR TYPE : M
- REINFORCED : NO. REF DWG: 4510-3.



A000 0016

GPU Service

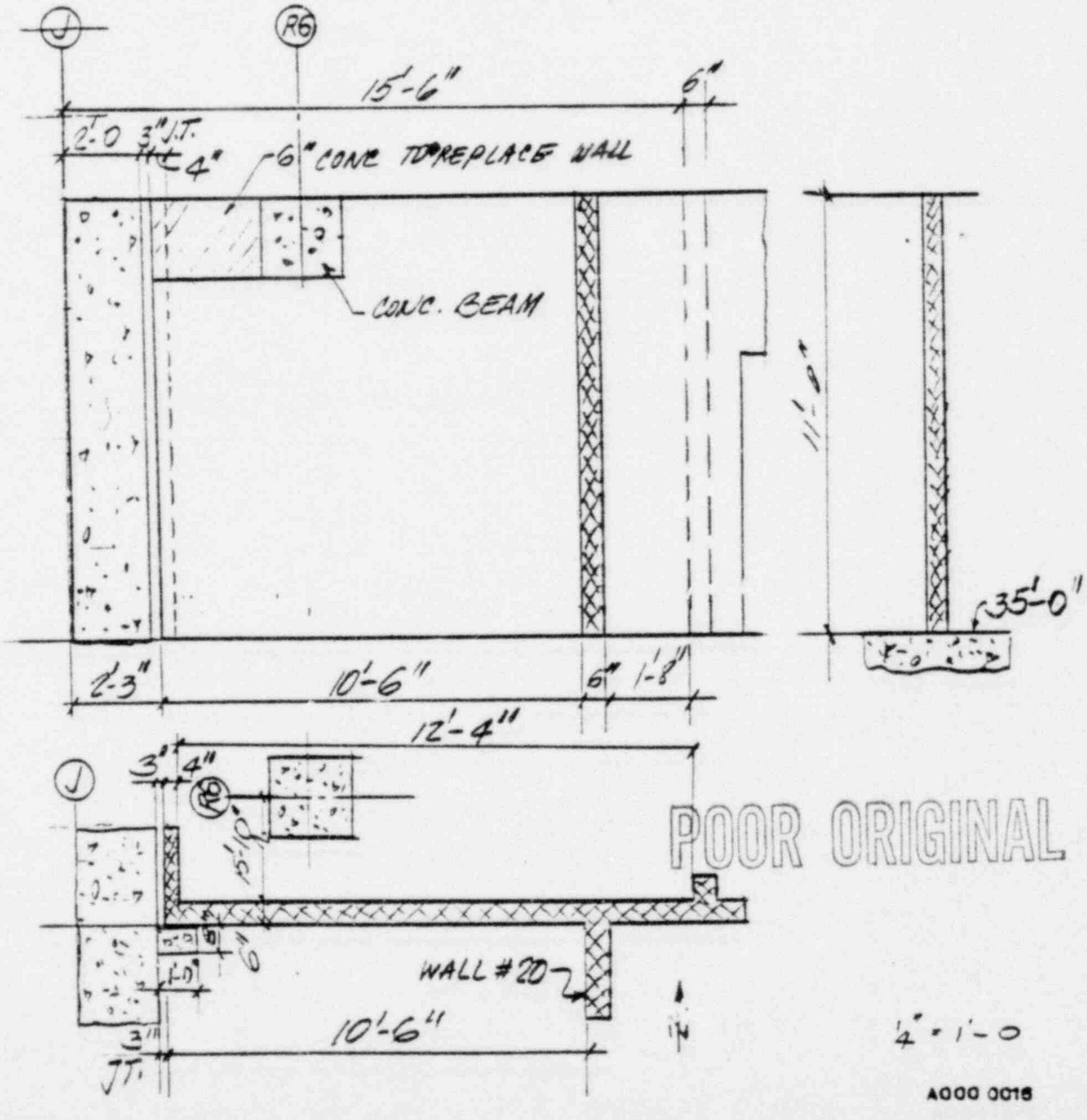
CALC. NO.
SHEET NO. 12 OF 36
DATE
COMP. BY/DATE A.T. 10.10.80
CHK'D. BY/DATE [illegible]

SUBJECT OYSTER CREEK STATION - UNIT #1

WALL # 19 - ELECTRIC TRAY RM. (NORTH WALL)
10'-6" LONG

WALL THICKNESS : 6" HOLLOW CONC. MASONRY UNITS (STACKED)
MORTAR TYPE : M
REINFORCED : NO.

REF DWGS. : 4510-3
4515-2



POOR ORIGINAL

1/4" = 1'-0

A000 0016

GPU Service

SUBJECT OYSTER CREEK STATION - UNIT #1

WALL #20 - ELECTRIC TRAY RM. (EAST WALL)
16'-2" LONG

CALC. NO.

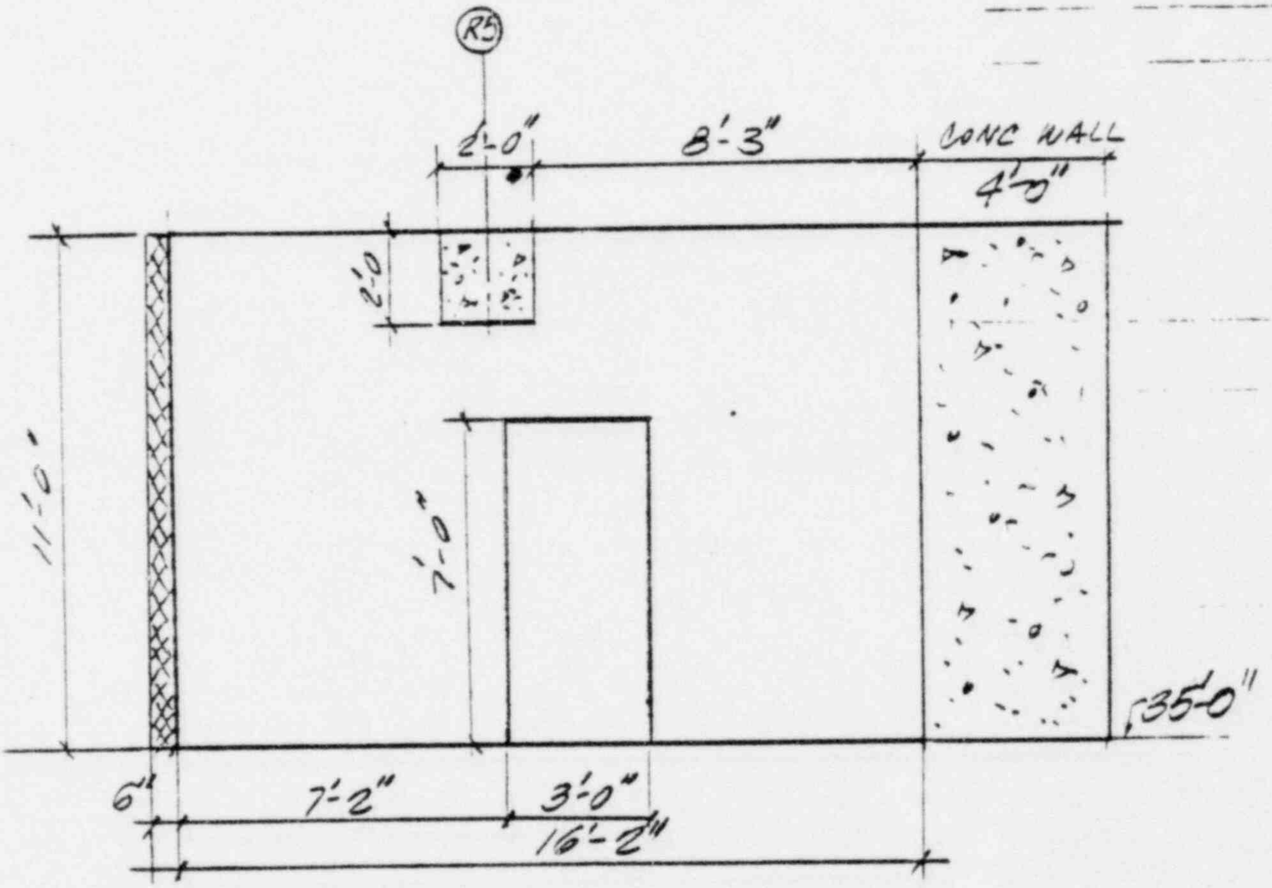
SHEET NO. 13 OF 36

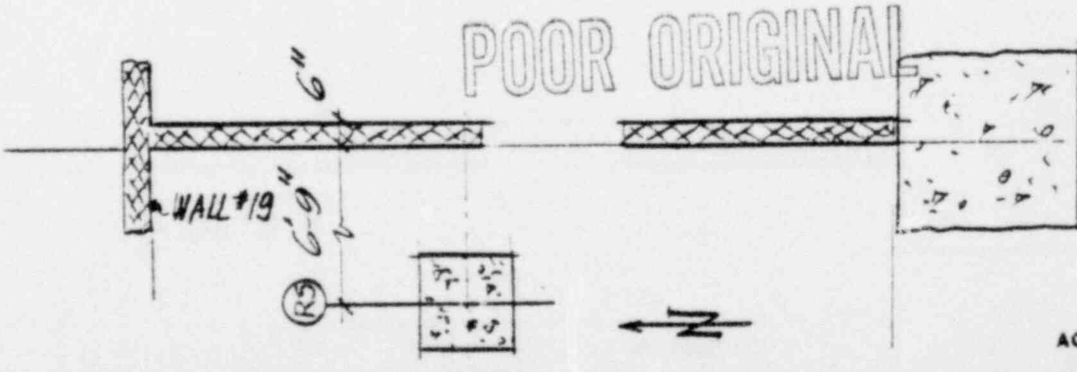
DATE

COMP. BY/DATE AT 10.10.80

CHK'D. BY/DATE

WALL THICKNESS : 6" HOLLOW CONC MASONRY UNITS (STACKED)
MORTAR TYPE : M
REINFORCED : NO. REF. DRWGS.: 4510-3





1/4" = 1'-0

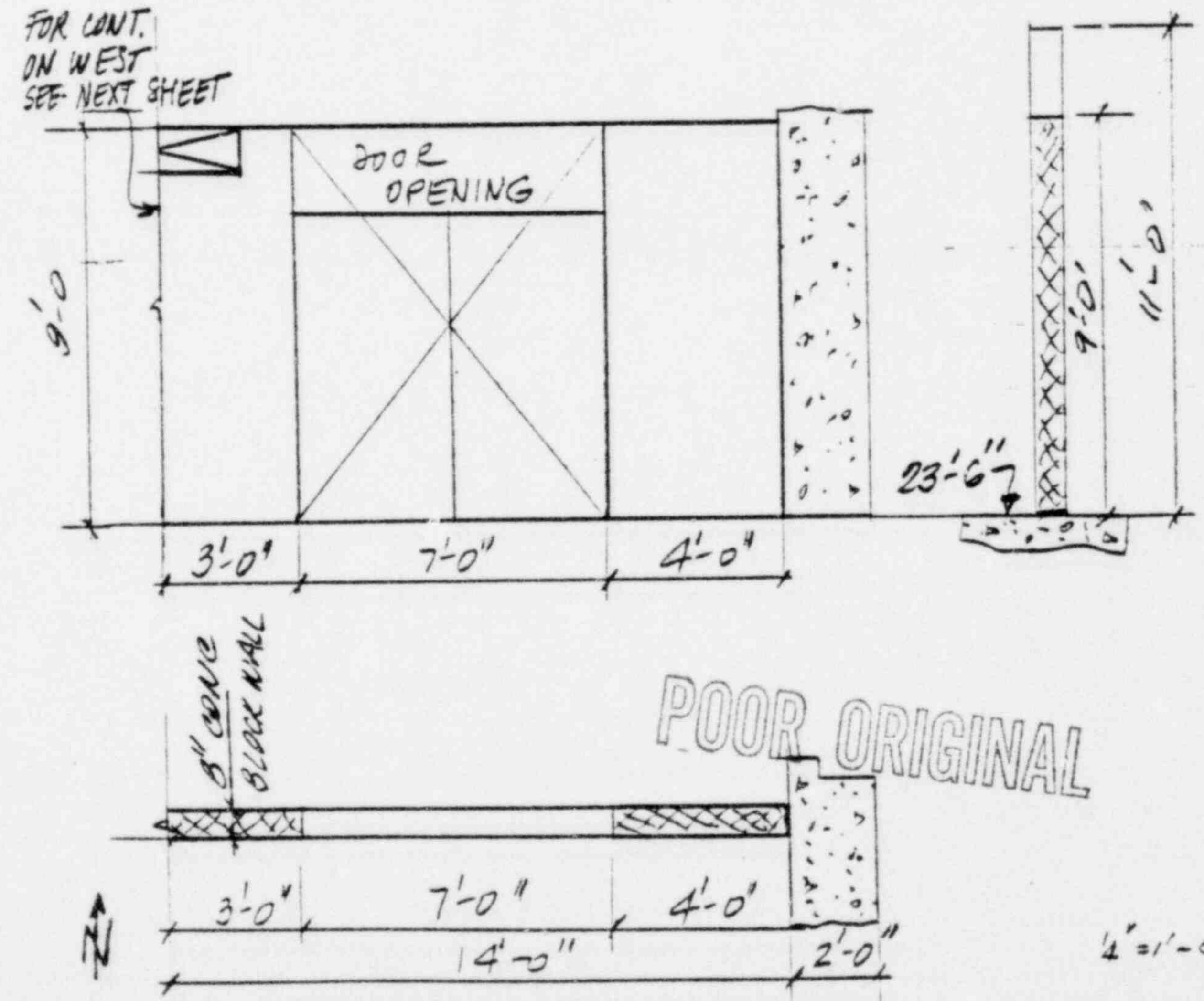
A000 0016

GPU Service

CALC. NO.
SHEET NO. 14 OF 36
DATE
COMP. BY/DATE A.T. 10.10.80
CHK'D. BY/DATE [illegible] 10/21/80

SUBJECT OYSTER CREEK STATION - UNIT #1
WALL # 21 480 SWICHTGEAR ROOM
(NORTH WALL) 34'-0 LONG.

WALL THICKNESS : 8" HOLLOW CONC MASONRY UNITS (STAGGERED).
- MORTAR TYPE : M
REINFORCED : NO. REF DWG : 4510-3



1/4" = 1'-0

A000 0016

GPU Service

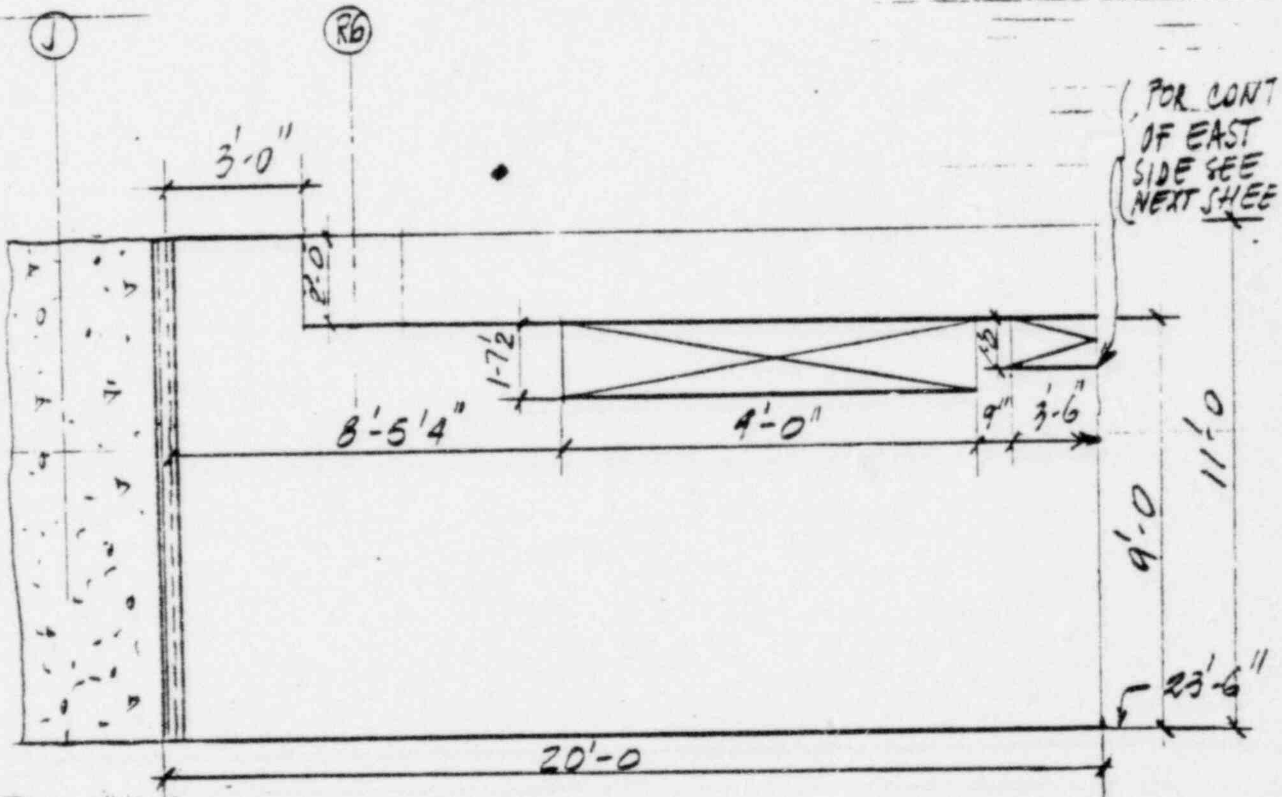
CALC. NO.

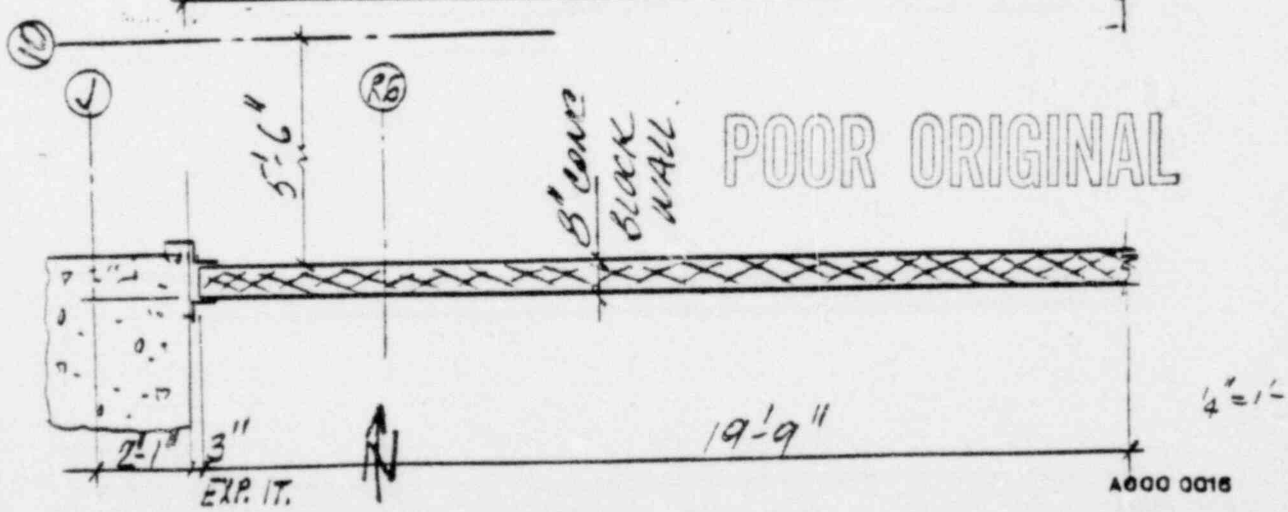
DATE
COMP. BY/DATE AT 10.10.80
CHK'D. BY/DATE

SUBJECT OYSTER CREEK STATION - UNIT #1

WALL #21 - 480 SWITCHGEAR ROOM (NORTH WALL). (34'-0 LG. WALL)

WALL THICKNESS : 8" HOLLOW CONC MASONRY UNITS (STAGGERED).
MORTAR TYPE : M
REINFORCED : NO. REF. DWGS: 4510-3.





POOR ORIGINAL

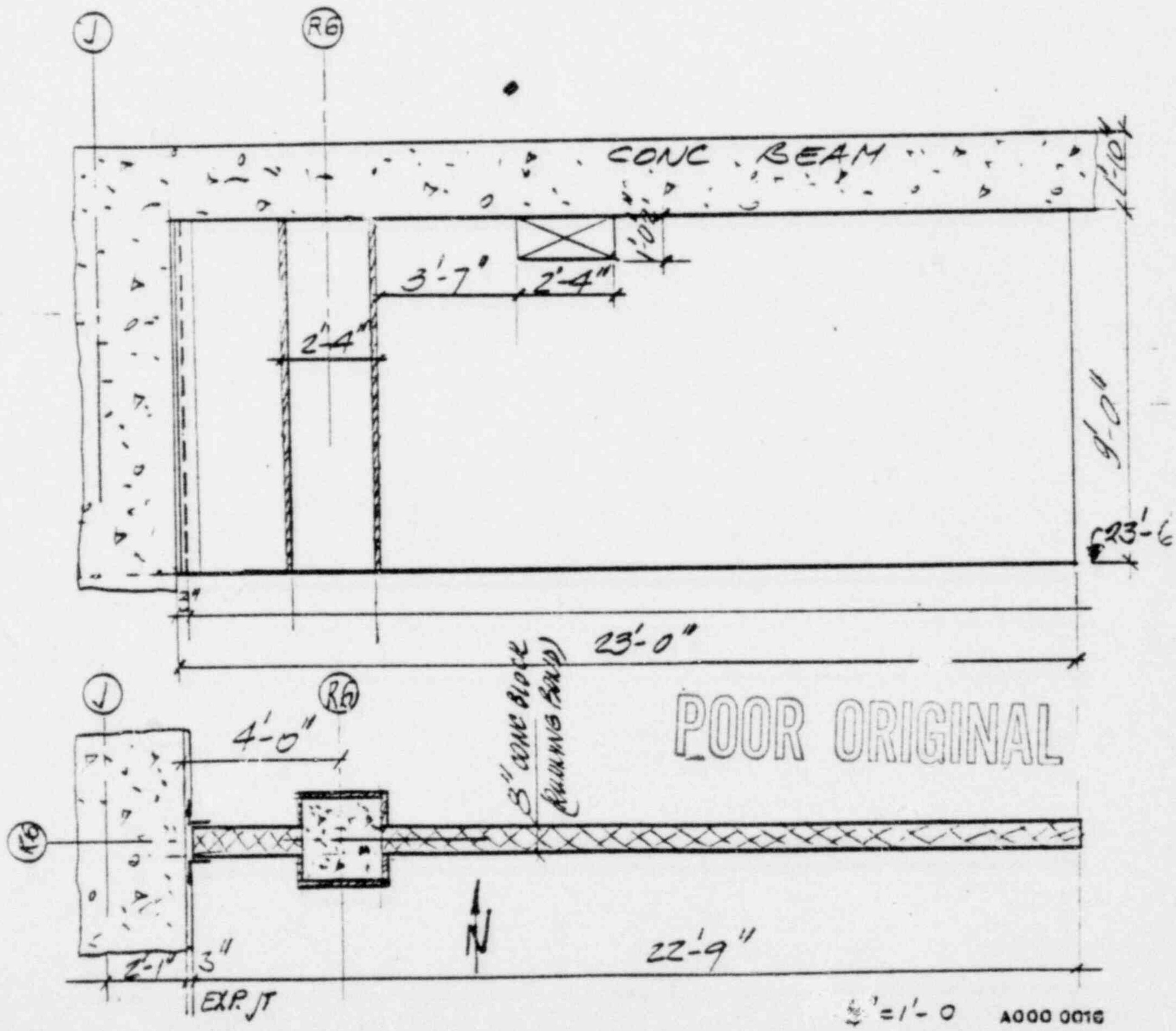
A000 0016

GPU Service

SUBJECT OYSTER CREEK STATION - UNIT #1
WALL #22 - 480 SWITCHGEAR ROOM
(CENTER BLOCK WALL) 23'-0" LONG

CALC. NO.
SHEET NO. 16 OF 36
DATE
COMP. BY/DATE AT 10.10.80
CHK'D. BY/DATE

WALL THICKNESS : 8" Hollow CONC MASONRY UNITS (STAGGERED).
MORTAR TYPE : M
REINFORCED : NO. REF. DWG: 4510-3



$\frac{1}{4}$"=1'-0

A000 0016

GPU Service

CALC. NO.

SHEET NO. 17 OF 36

DATE

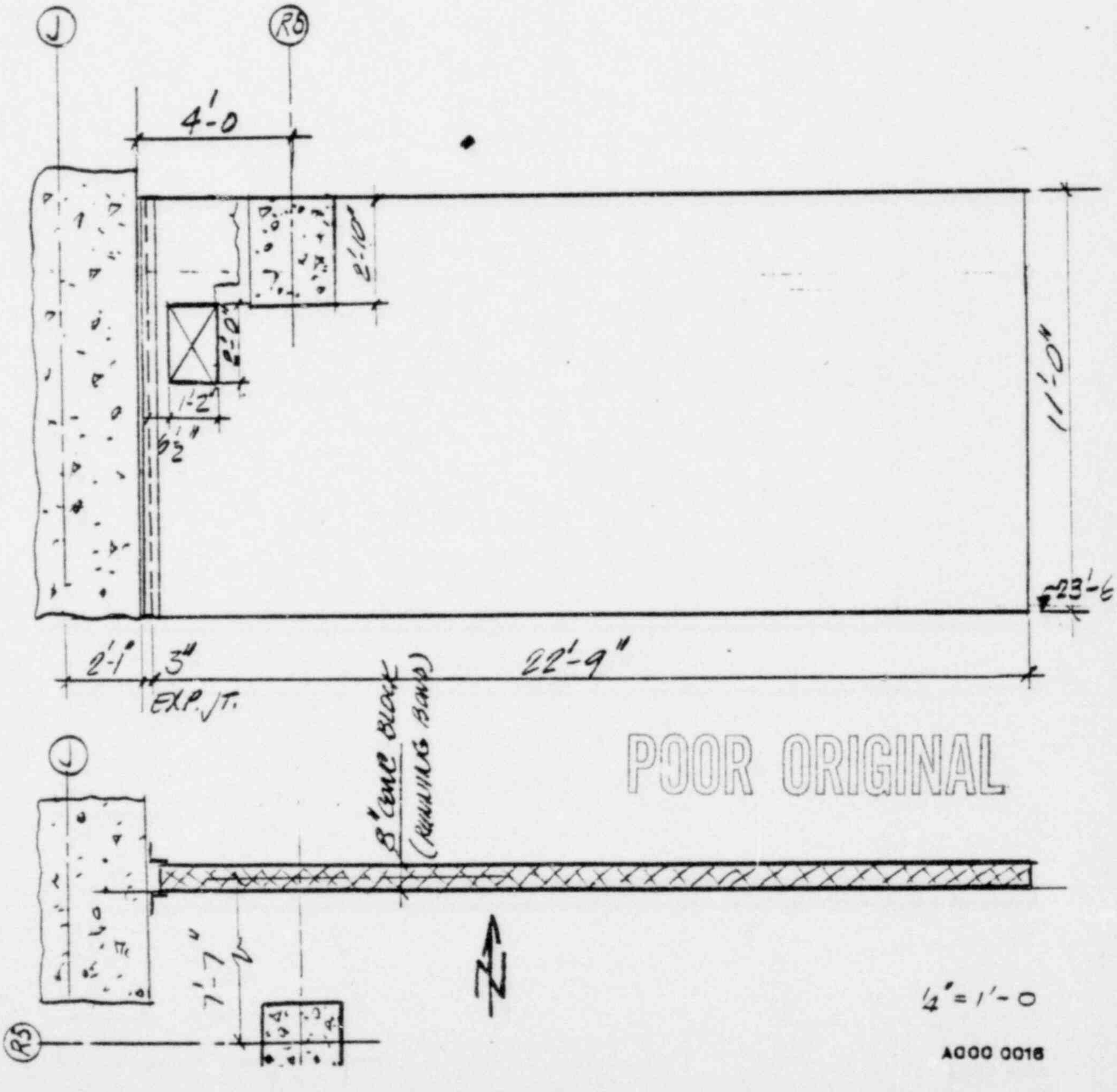
SUBJECT OYSTER CREEK STATION - UNIT #1

COMP. BY/DATE AT 10.13.80

CHK'D. BY/DATE [illegible] 10/17/80

WALL # 23 - 480 SWITCHGEAR ROOM (SOUTH BLOCK WALL) 22'-9" LONG.

WALL THICKNESS : 8" HOLLOW CONC MASONRY UNITS (STAGGERED)
MORTAR TYPE : M
REINFORCED : NO. REF DWG : 4510-3



A000 0016

GPU Service

CALC. NO.

SHEET NO. 18 OF 36

DATE

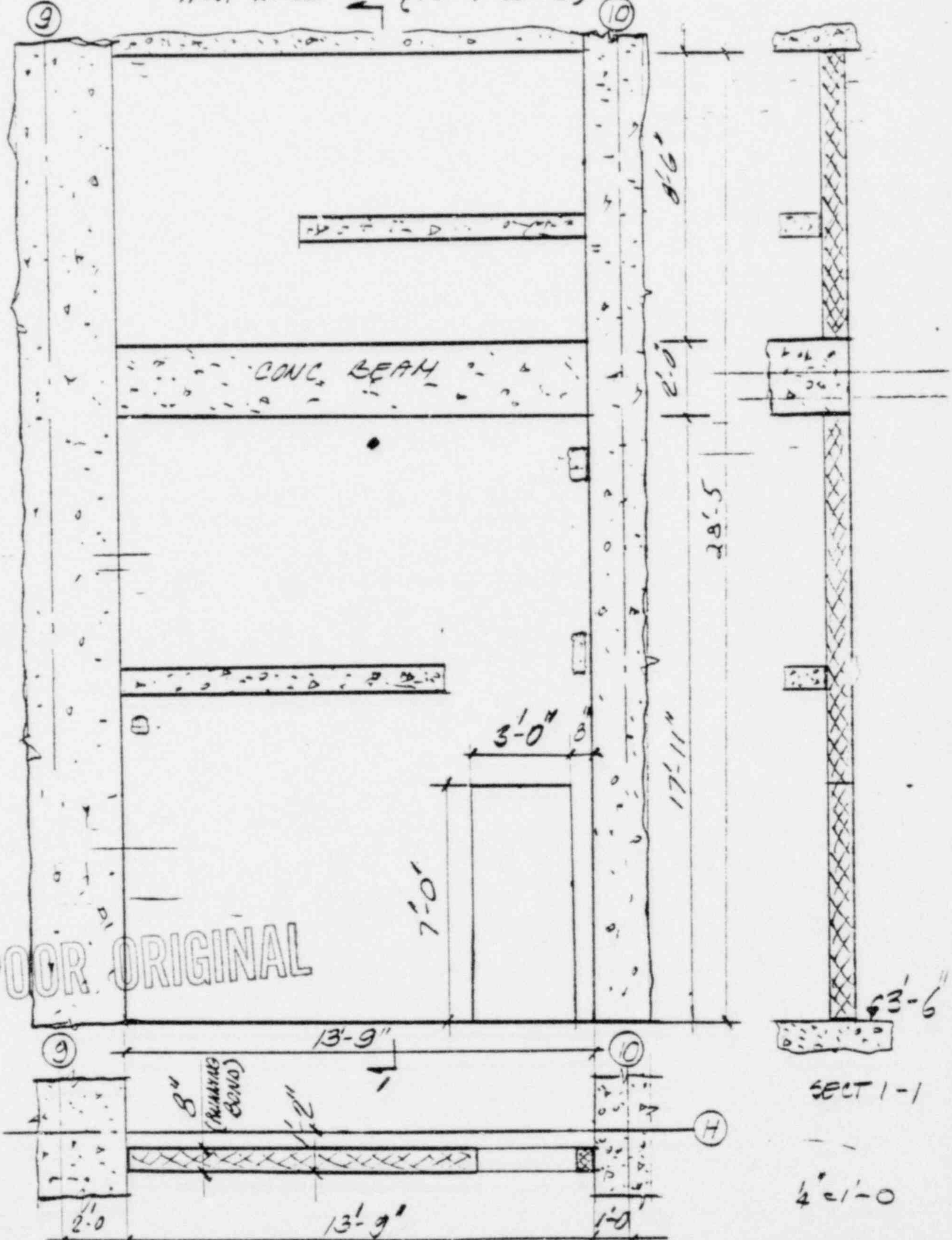
COMP. BY/DATE A.T. 10.13.80

CHK'D. BY/DATE [illegible]

SUBJECT OYSTER CREEK STATION — UNIT #1

WALL #24 – TURBINE BUILDING NORTH EAST STAIRWELL

WEST WALL 1 (13'-9 LONG)



GPU Service

CALC. NO. ...

SHEET NO. 19 OF 36

DATE ...

COMP. BY/DATE A.T. 10.13.80

CHK'D. BY/DATE [illegible]

SUBJECT OYSTER CREEK STATION — UNIT #1

WALL #24 - TURBINE BUILDING NORTHEAST STAIR.

WEST WALL 13'-9" LONG.

WALL THICKNESS : 8" Hollow CONC MASONRY UNITS (STAGGERED)

MORTAR TYPE : M

REINFORCED : NO. REF DWGS: 2050-3

POOR ORIGINAL

A000 0016

GPU Service

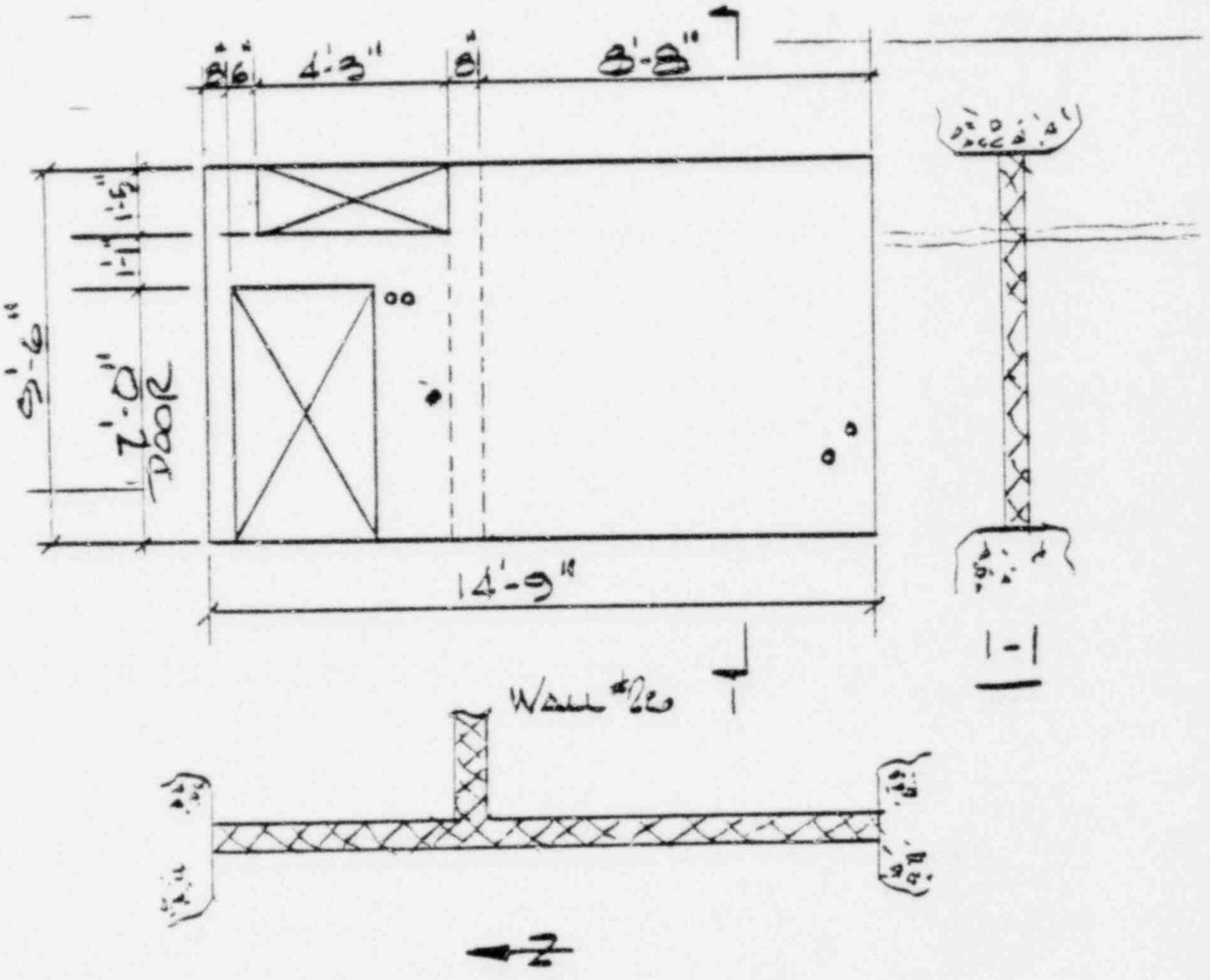
CALC. NO.
SHEET NO. 20 OF 36
DATE
COMP. BY/DATE RLF 10-31-80
CHK'D. BY/DATE VF 10-31-80

SUBJECT OYSTER CREEK STATION - UNIT #1

WALL #26 - CABLE SPREADING ROOM - WEST WALL 14'-9" LONG

WALL THICKNESS = 8" HOLLOW BLOCK
MORTAR TYPE : M
REINFORCED : YES W/UNISTRUT

REF DWG - 2051-1



POOR ORIGINAL

A000 0016

GPU Service

CALC. NO.

DATE

SUBJECT OYSTER CREEK STATION - UNIT #1

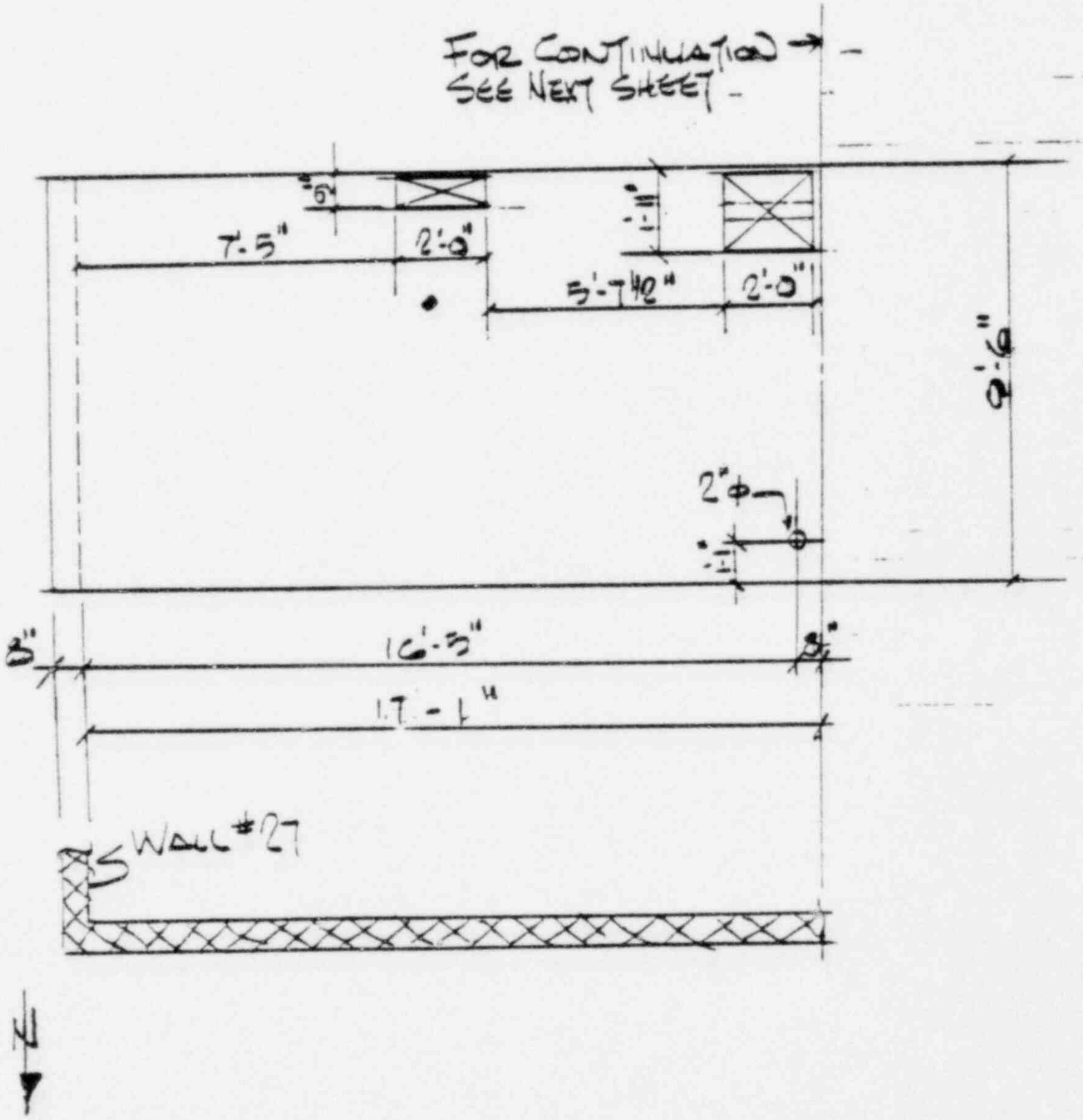
COMP. BY/DATE LB 10-31-80

CHK'D. BY/DATE VF 10-31-80

WALL #26 - CABLE SPREADING RM. - WEST SECT. NORTH WALL 30'-0" LONG

WALL THICKNESS : 8" HOLLOW BLOCK
MORTAR TYPE : M
REINFORCED : YES W/ UNISTRUT

REF DWG : 2051-1



POOR ORIGINAL

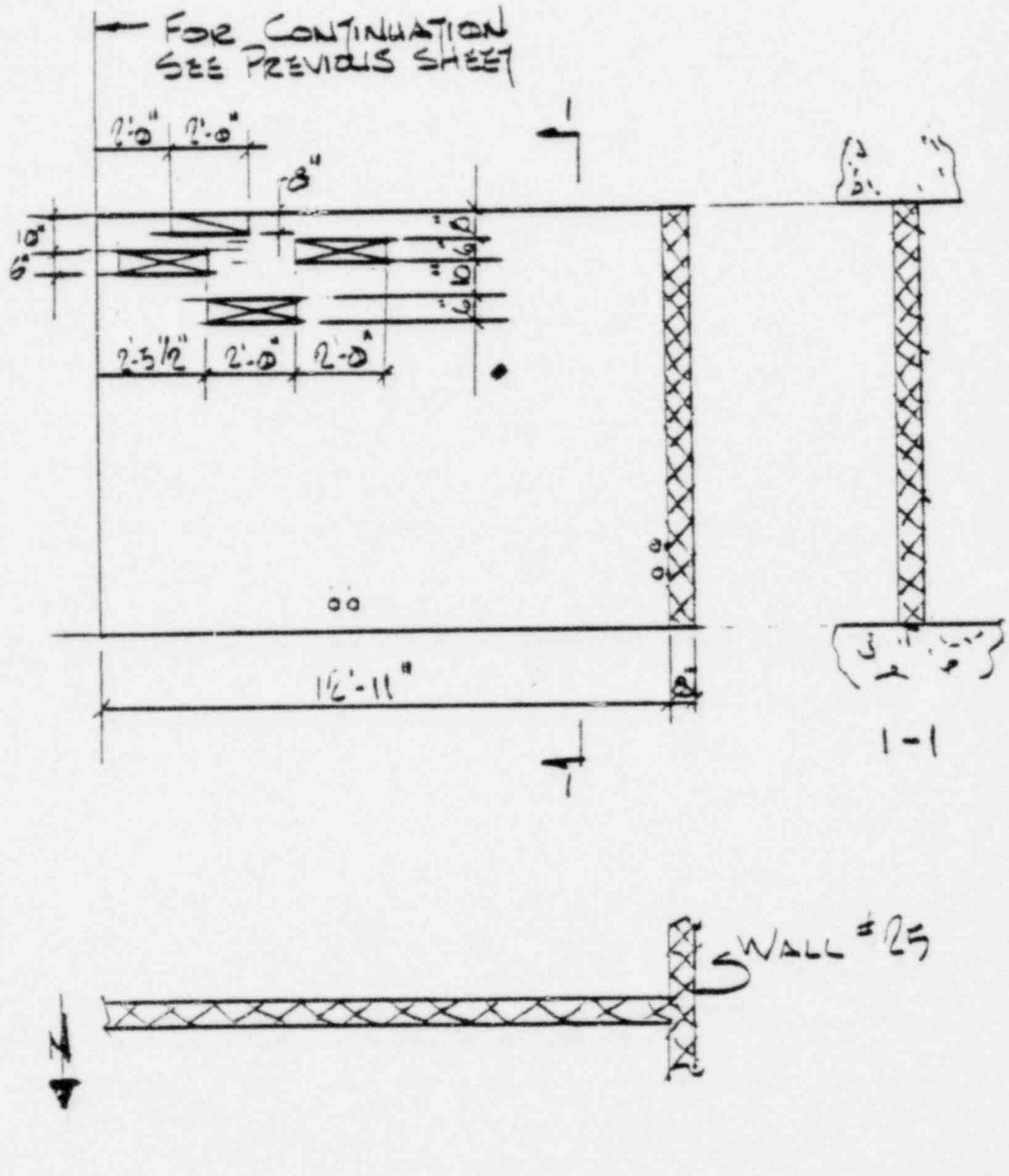
AGOO 0016

GPU Service

CALC. NO.
SHEET NO. 22 OF 36
DATE
COMP. BY/DATE [illegible] 10-31-80
CHK'D. BY/DATE [illegible] 10-31-80

SUBJECT OYSTER CREEK STATION - UNIT #1

WALL #26 - CABLE SPREADING RM - WEST SECTION OF NORTH WALL



POOR ORIGINAL

A000 0016

GPU Service

CALC. NO.

SHEET NO. 23 OF 36

SUBJECT OYSTER CREEK STATION - UNIT #1

DATE

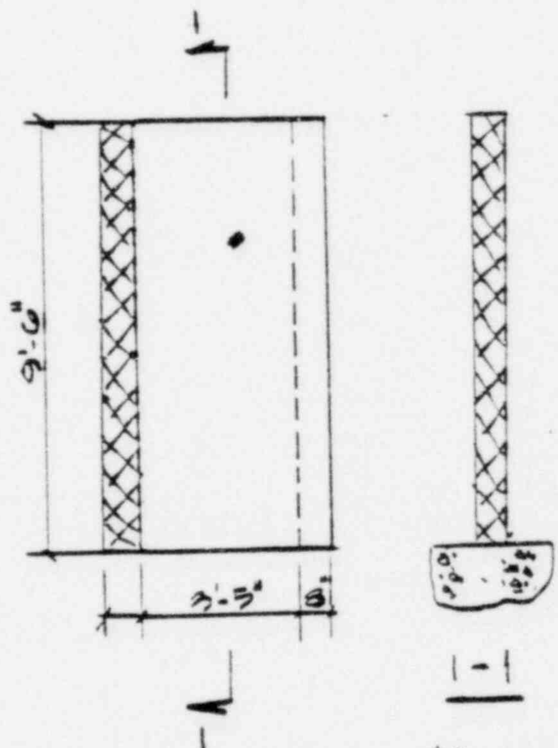
COMP. BY/DATE LH 10-31-80

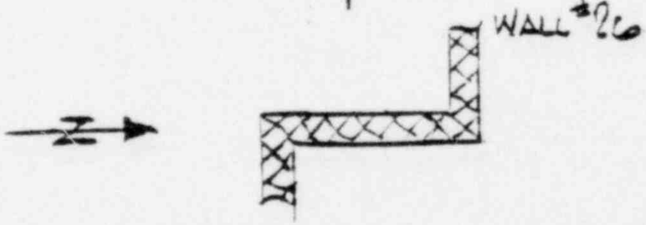
WALL #27 - CABLE SPREADING RM - North-South WALL

CHK'D. BY/DATE [illegible] 10-31-80

WALL THICKNESS : 8" HOLLOW BLOCK 3'-5" LONG
MORTAR TYPE : M
REINFORCED : YES - UNISTRUT

REF DWGS: 2031-4





POOR ORIGINAL

A000 0016

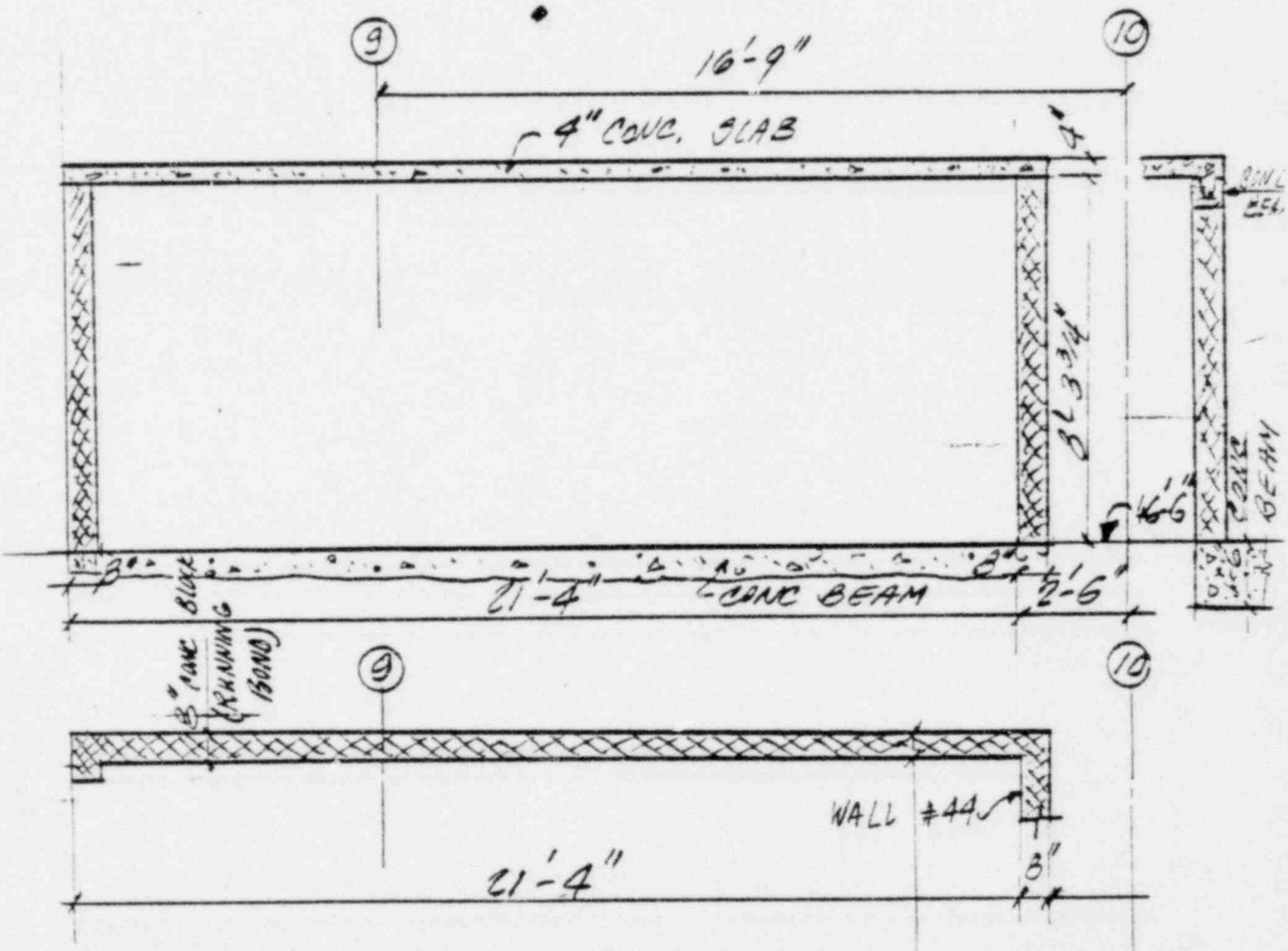
GPU Service

SUBJECT OYSTER CREEK STATION - UNIT #1

WALL # 28 NORTH EAST STAIRWELL FROM TURBINE OPERATING FLOOR WEST WALL. (22'-0 LONG).

CALC. NO.
SHEET NO. 24 OF 36
DATE
COMP. BY/DATE AT 10.13.80
CHK'D. BY/DATE JH 10/17/80

WALL THICKNESS : 8" HOLLOW CONC. MASONRY UNITS (STAGGERED).
MORTAR TYPE : M
REINFORCED : NO. REF DWG: 4514



1/4" = 1'-0

POOR ORIGINAL

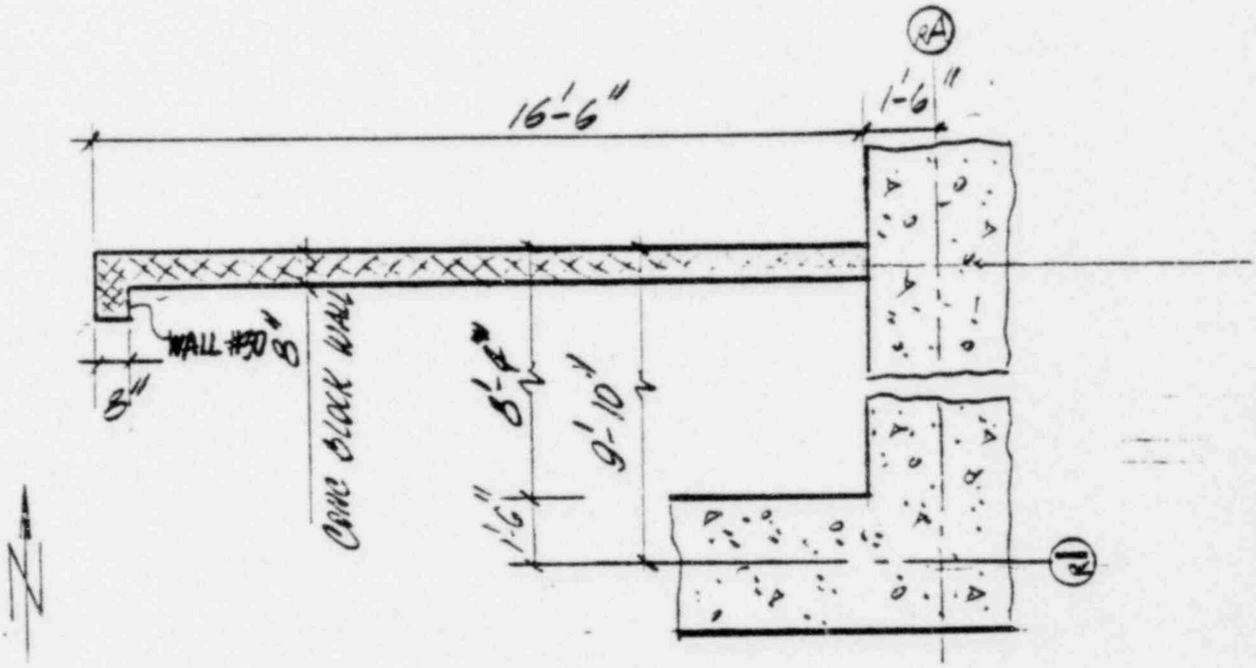
GPU Service

CALC. NO.
SHEET NO. 25 OF 36
DATE
COMP. BY/DATE FT 10.13.80
CHK'D. BY/DATE 1/17/81

SUBJECT OYSTER CREEK STATION — UNIT #1

WALL # 29 - REACTOR BUILDING S/E STAIRWELL TO -19'-6" NORTH WALL.
(15'-10" LONG WALL)

WALL THICKNESS : 8" Hollow CONC MASONRY UNITS (STACKED).
MORTAR TYPE : M
REINFORCED : YES REF DWG. 4509

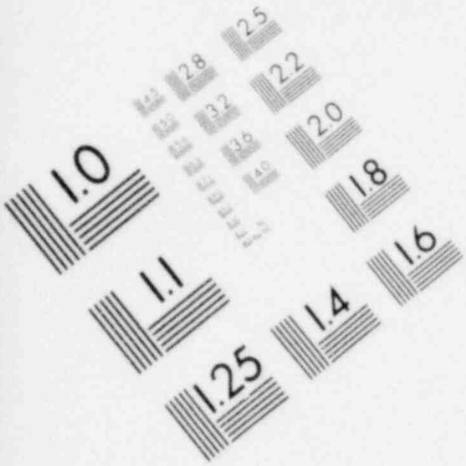


SECT 2-2

1/4" = 1'-0

POOR ORIGINAL

A000 0016



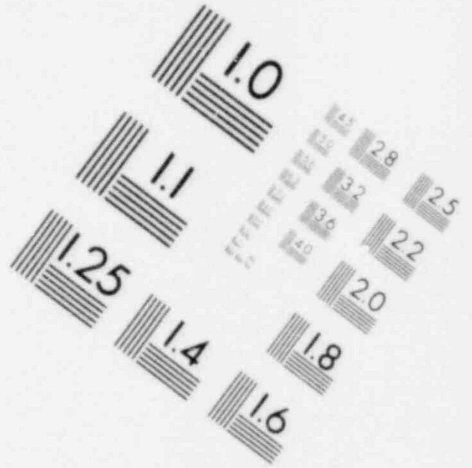
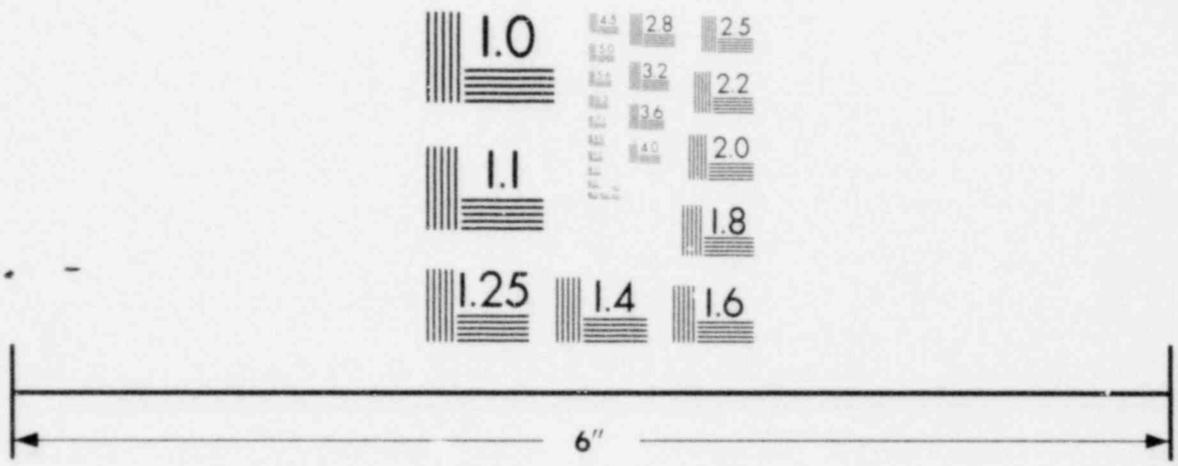
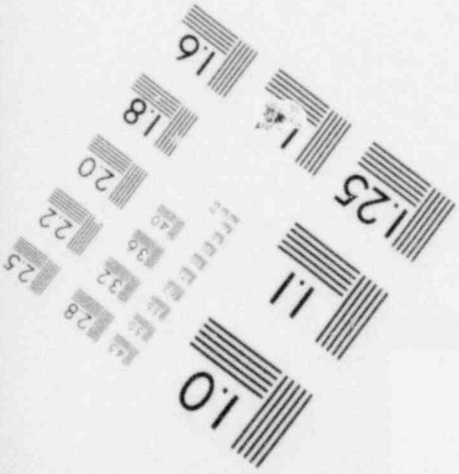


IMAGE EVALUATION TEST TARGET (MT-3)





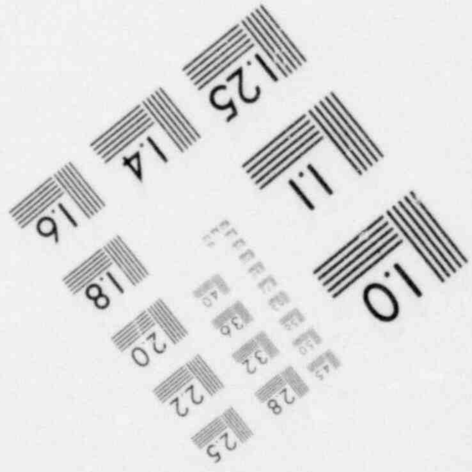


IMAGE EVALUATION
TEST TARGET (MT-3)

6"

GPU Service

CALC. NO. ....

SHEET NO. 26 OF 36

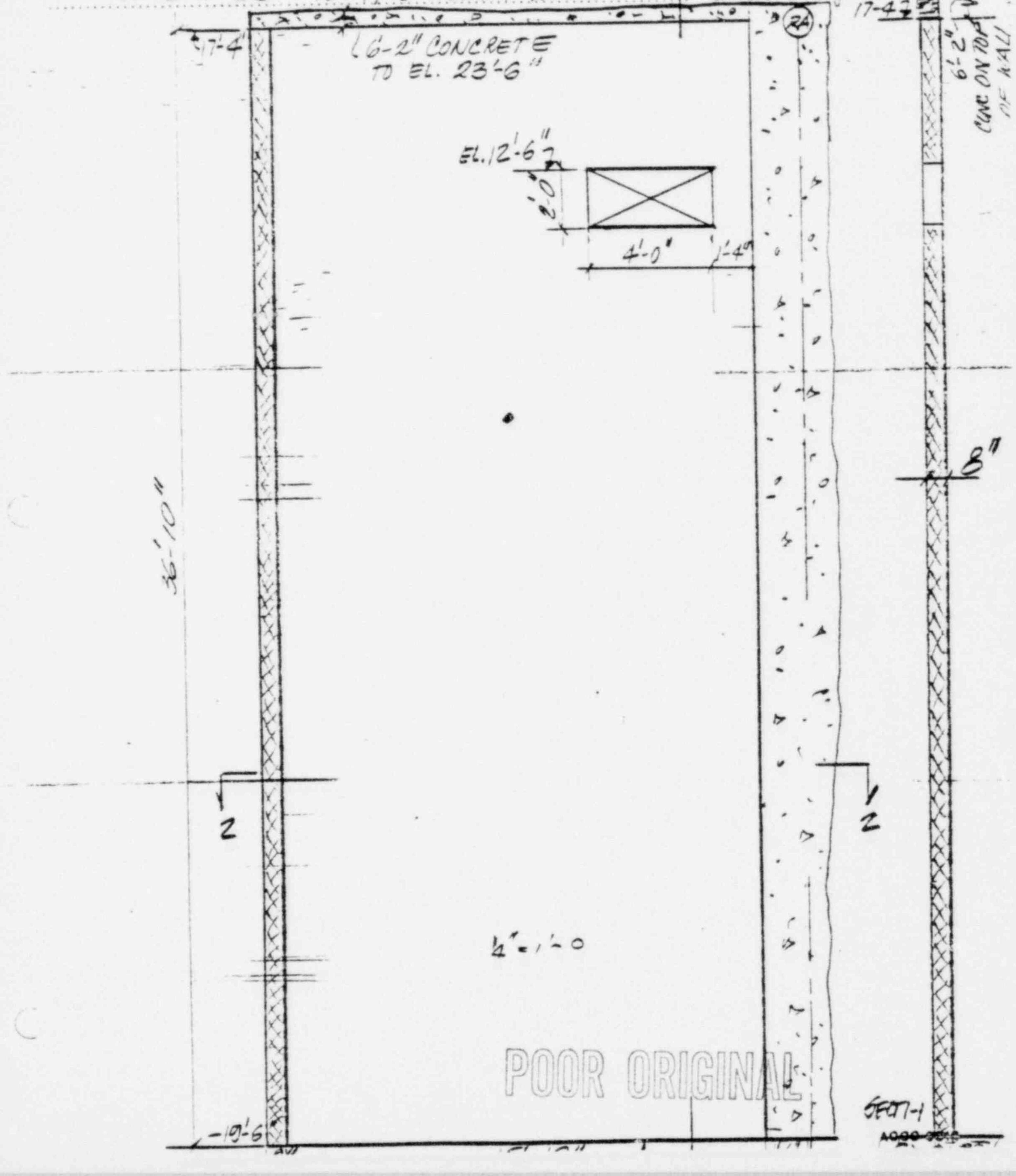
DATE ....

COMP. BY/DATE AT 10.13.80

CHK'D. BY/DATE

SUBJECT OYSTER CREEK STATION — UNIT #1

WALL # 29 - REACTOR BUILDING S/E STAIR WELL TO -19'-6" NORTH WALL



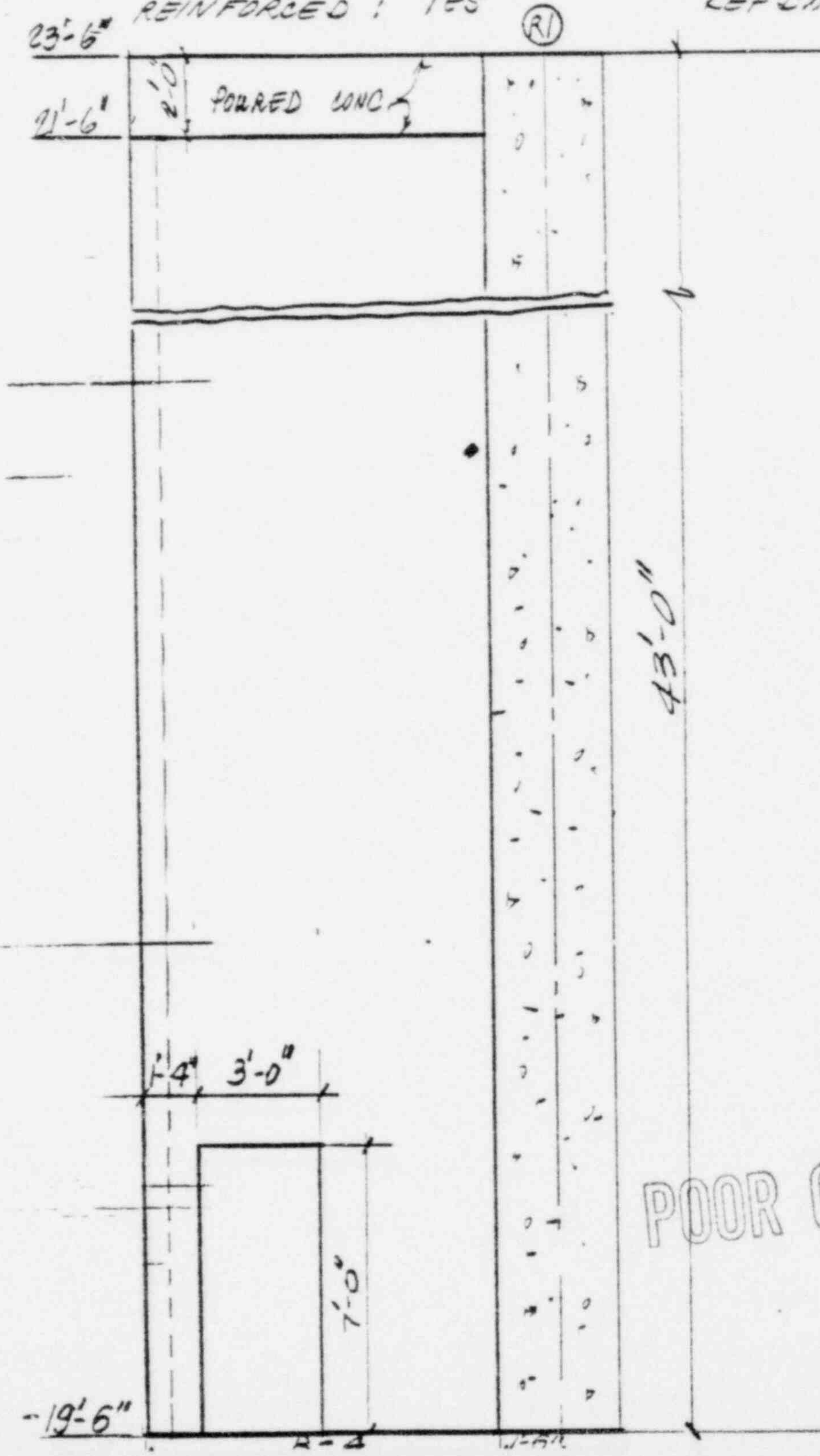
GPU Service

CALC. NO.
SHEET NO. 27 OF 36
DATE
COMP. BY/DATE AT 10.13.80
CHK'D. BY/DATE [illegible]

SUBJECT OYSTER CREEK STATION - UNIT #1

WALL #30 REACTOR BUILDING S/E STAIRWELL TO -19'-6" -- WEST WALL. 8'-4" LONG.

WALL THICKNESS ; 8" Hollow CONC MASONRY UNITS (STACKED)
MORTAR TYPE : M
REINFORCED : YES
REF DWG: 4509-3



POOR ORIGINAL

1/4" = 1'-0

A000 0016

GPU Service

CALC. NO.

SHEET NO. 28 OF 36

SUBJECT OYSTER CREEK STATION - UNIT #1

DATE

COMP. BY/DATE. AT 10.13.80

WALL #30 REACTOR BUILDING S/E STAIRWELL

CHK'D. BY/DATE

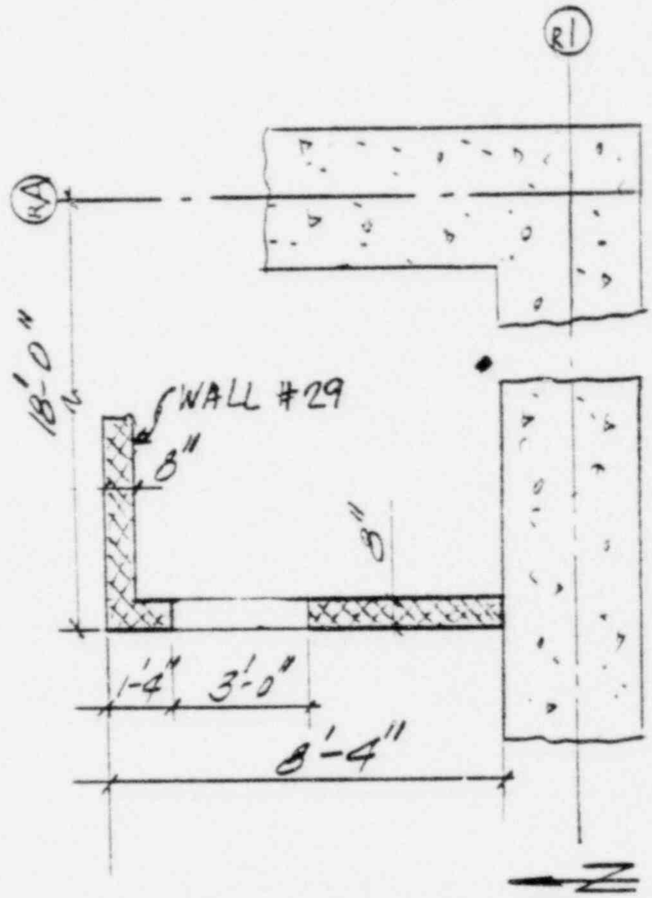
TO - 19'-6" - WEST WALL 8'-4" LONG

WALL THICKNESS : 8" HOLLOW MASONRY UNITS (STACKED)

MORTAR TYPE : M

REINFORCED : YES

REF. DWG. 4509-3



1/4" = 1'-0

POOR ORIGINAL

GPU Service

CALC. NO.

SHEET NO. 29 OF 36

SUBJECT OYSTER CREEK STATION - UNIT #1

DATE

COMP. BY/DATE AT 10.14.80

WALL #31 REACTOR BUILDING ELEVATOR

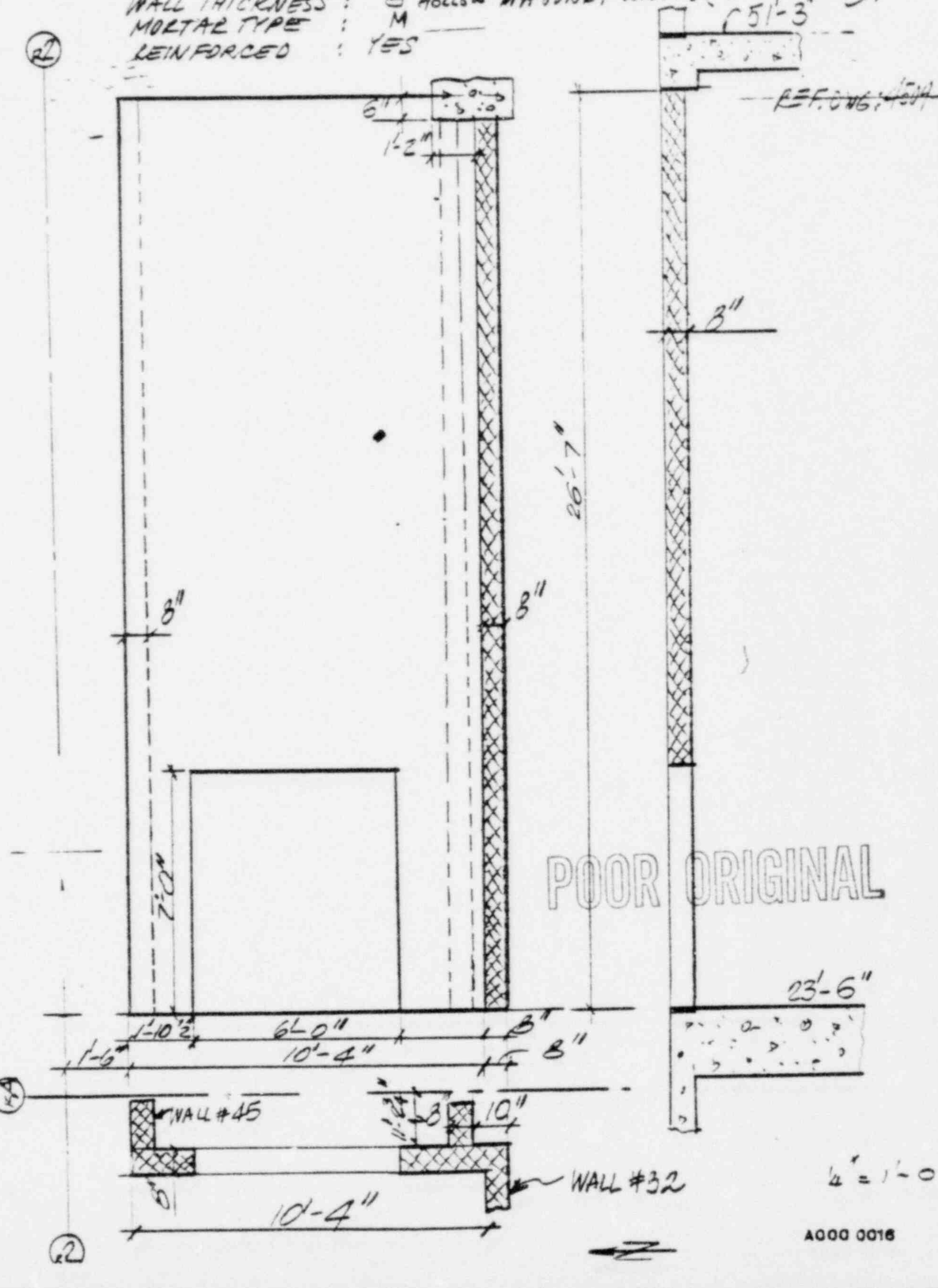
CHK'D/BY/DATE 1/17/80

23'-6" TO 51'-3" - WEST WALL 10-4 LONG

WALL THICKNESS : 8" Hollow MASONRY UNITS (STACKED).

MORTAR TYPE : M

REINFORCED : YES



¼" = 1'-0

POOR ORIGINAL

A000 0016

GPU Service

CALC. NO.

SHEET NO. 30 OF 36

DATE

COMP. BY/DATE AT 10.14.80

CHK'D. BY/DATE

SUBJECT OYSTER CREEK STATION - UNIT #1

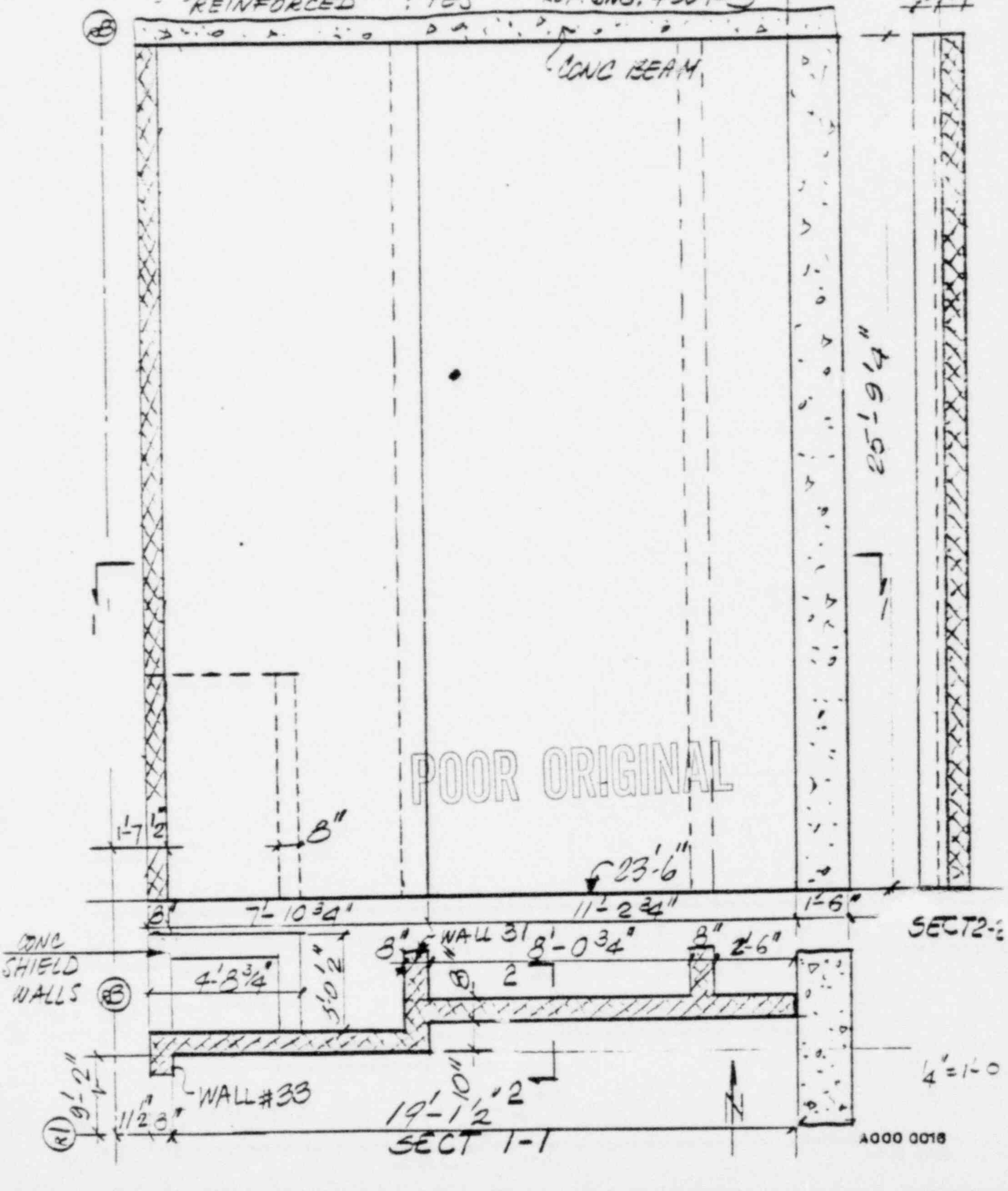
WALL #32 - RX. BUILDING STAIRWELL

23'-6" TO 51'-3" NORTH WALL 19'-1/2" LONG

WALL THICKNESS : 8" Hollow MASONRY UNITS (STACKED)

MORTAR TYPE : M

REINFORCED : YES REF. DWG. 4509-3



A000 0016

GPU Service

CALC. NO.

SHEET NO. 31 OF 36

SUBJECT OYSTER CREEK STATION - UNIT #1

DATE

COMP. BY/DATE AT 10.14.80

WALL #33 RX BUILDING STAIRWELL

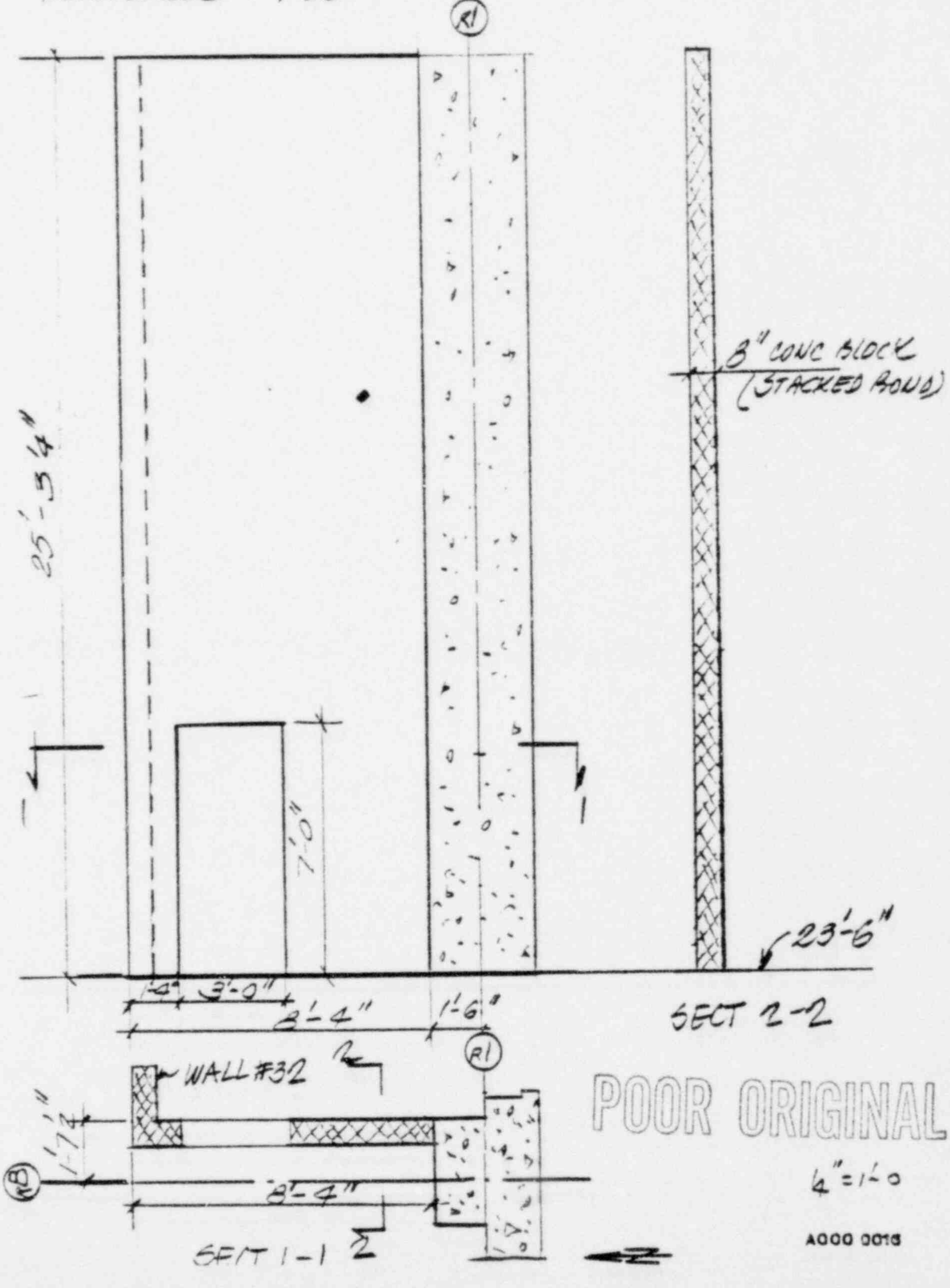
CHK'D. BY/DATE

23'-6" TO 51'-3" WEST WALL 8'-4" LONG

WALL THICKNESS : 8" HOLLOW MASONRY UNITS (STACKED)

MORTAR TYPE : M REF DWG: 4509-3

REINFORCED : YES



POOR ORIGINAL

A000 0016

GPU Service

CALC. NO.

DATE

COMP. BY/DATE AT 10/14/80

CHK'D. BY/DATE [illegible]

SUBJECT OYSTER CREEK STATION - UNIT #1.

WALL #42 CORRIDOR NO. 5 - EAST WALL (49'-10" LONG)

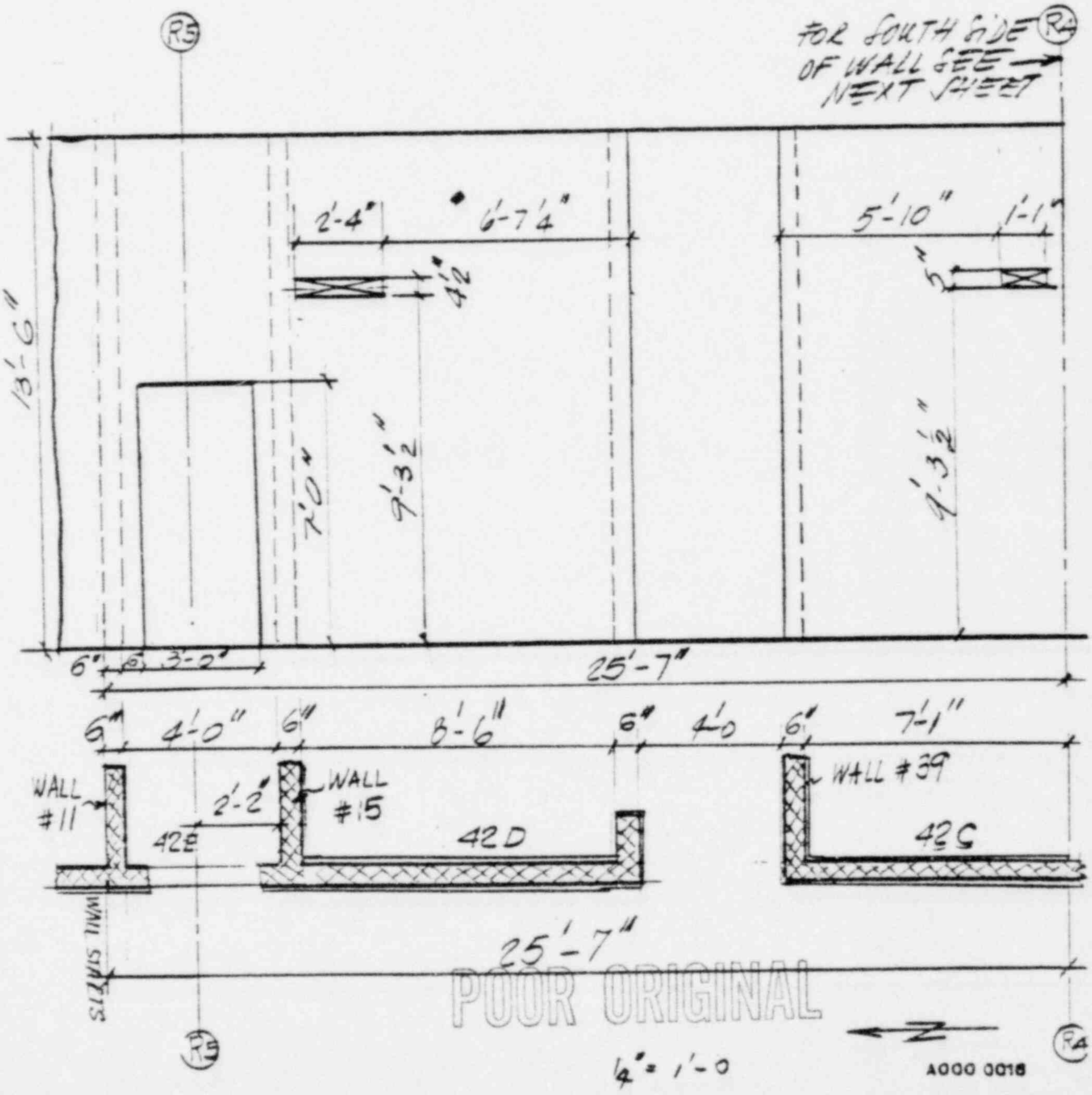
WALL THICKNESS : 6" Hollow GLAZED MASONRY UNITS (STAGGERED). WITH TILE (1/2")

MORTAR TYPE : M

REINFORCED : NO

REF. DWG. 4517-3
4511-3

(NORTH SIDE OF WALL LOOKING EAST)



A000 0016

GPU Service

CALC. NO.

SHEET NO. 33 OF 36

SUBJECT OYSTER CREEK STATION - UNIT #1

DATE

COMP. BY/DATE AT 10.14.80

WALL # 42 CORRIDOR NO. 5 - EAST WALL

(49'-10" LONG)

CHK'D. BY/DATE

WALL THICKNESS : 6" Hollow GLAZED MASONRY UNITS (STAGGERED) WITH TILE 1/2"

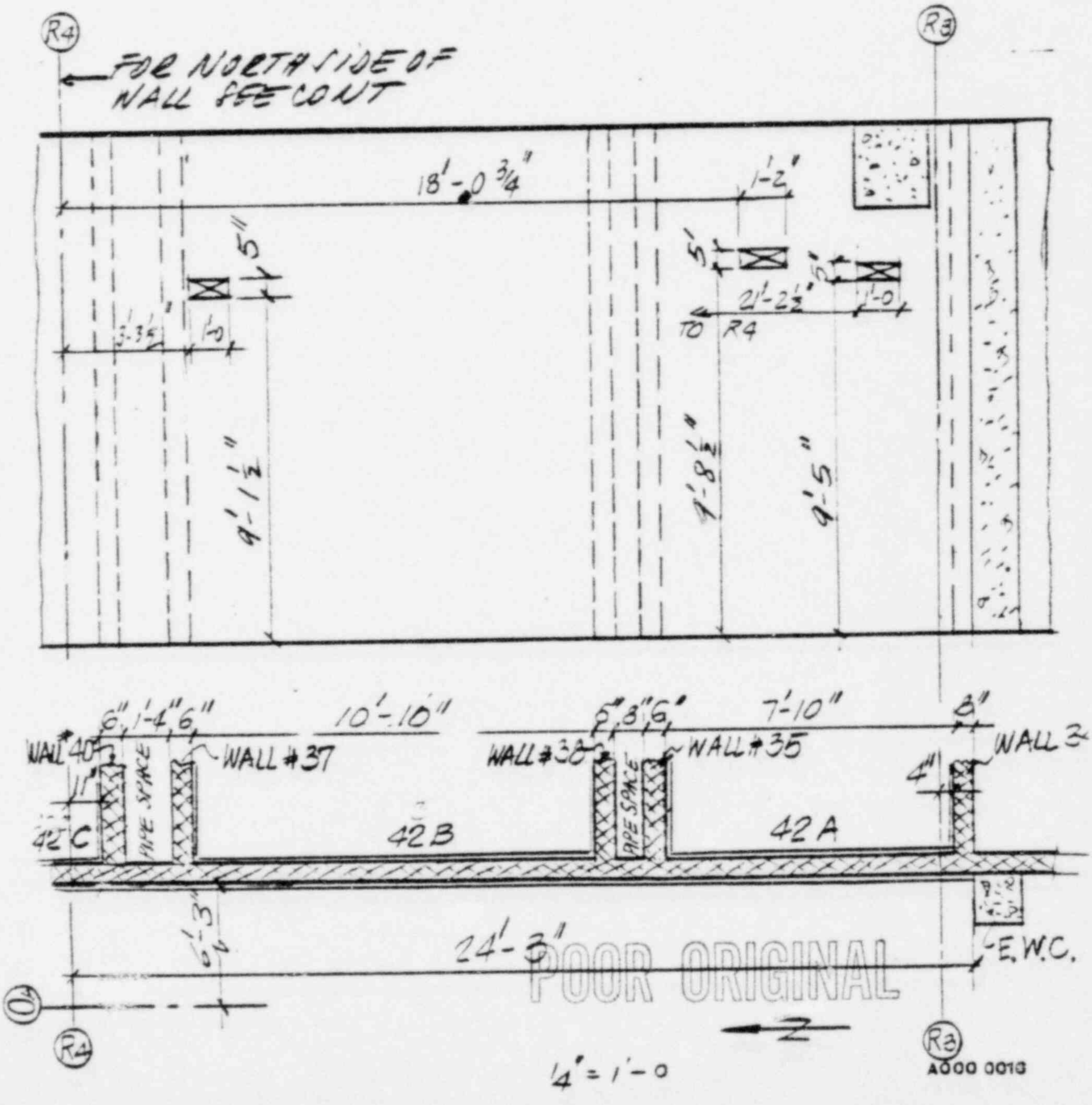
MORTAR TYPE : M

REINFORCED : NO.

REF. DWG. : 4517-8

4511-3.

(SOUTH SIDE OF WALL LOOKING WEST)



POOR ORIGINAL

1/4" = 1'-0

A000 0013

GPU Service

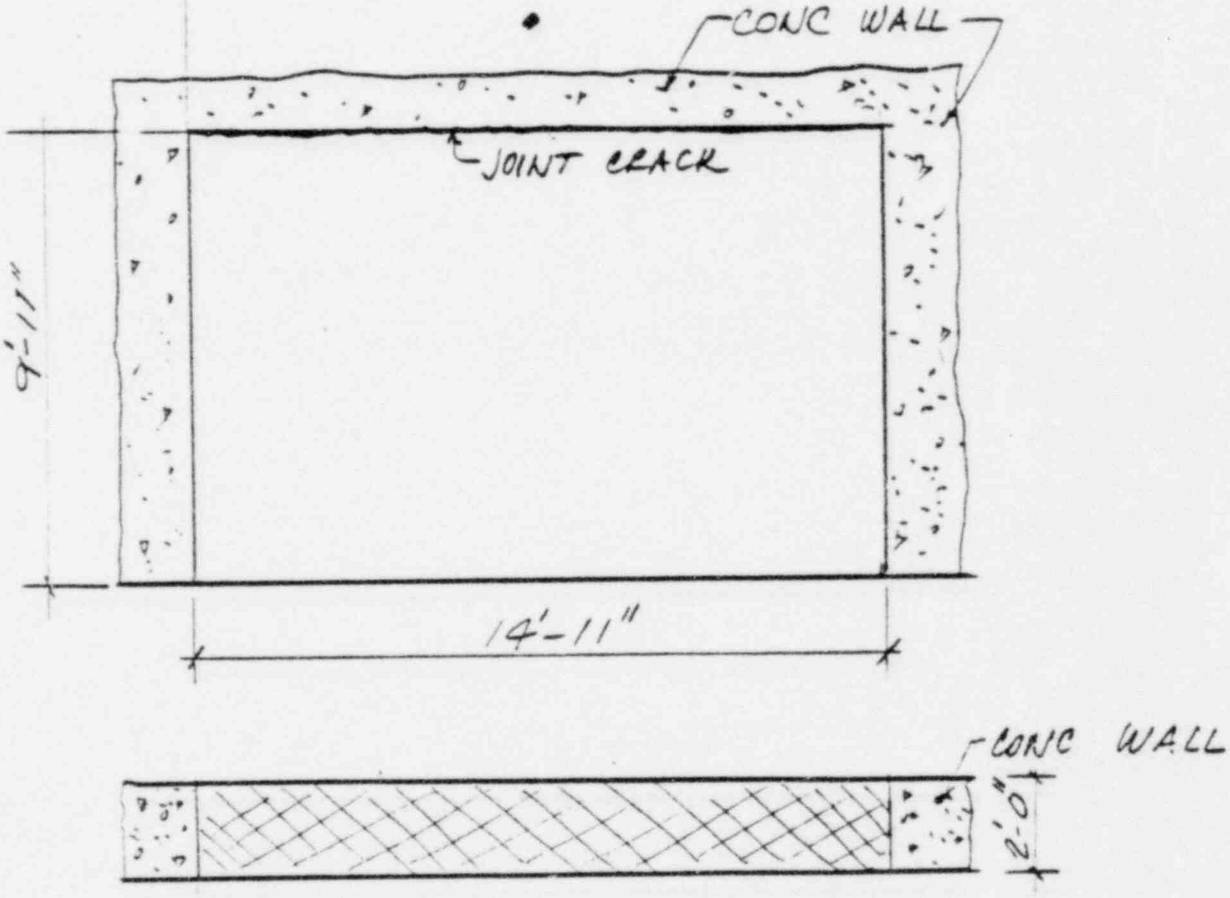
CALC. NO.
SHEET NO. 34 OF 36
DATE
COMP. BY/DATE
CHK'D. BY/DATE

SUBJECT OYSTER CREEK STATION - UNIT #1

WALL #43 SHUTDOWN HEAT EXCHANGER ROOM - NORTH WALL (KNOCKOUT SECTION).

WALL THICKNESS : 24" (MULTI-WYTHES STACKED WALL)
MORTAR TYPE : M
REINFORCED : No
REF. DWG. : 2061-4

24' WALL REMOVABLE



POOR ORIGINAL

1/4"=1'-0

A000 0016

GPU Service

CALC. NO.

SHEET NO. 35 OF 36

DATE

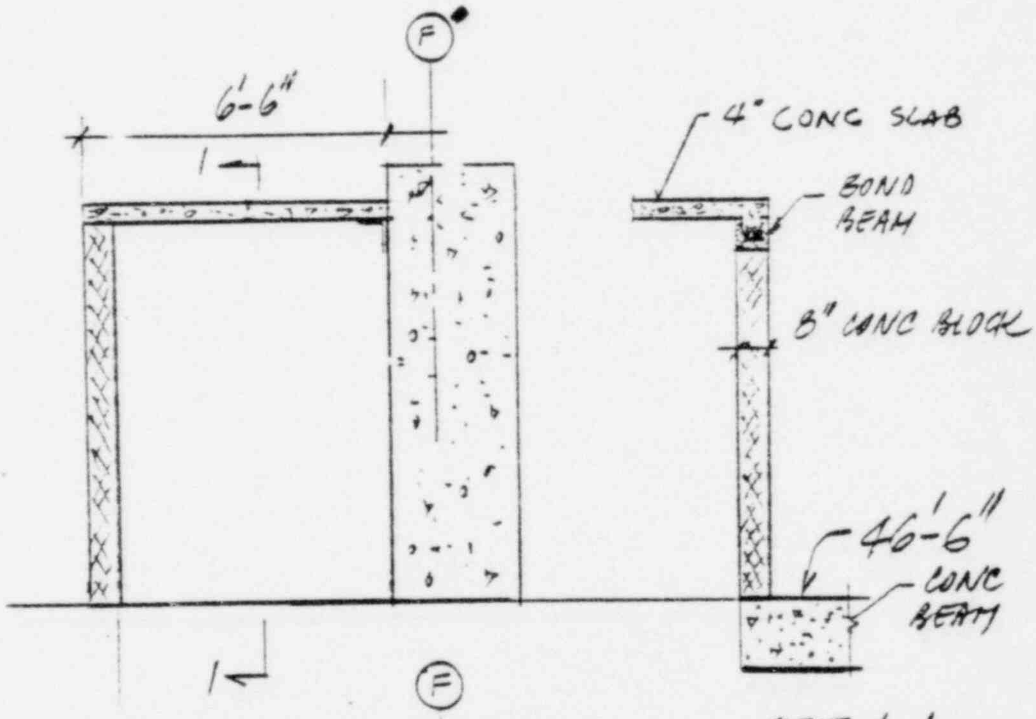
COMP. BY/DATE HT 10.15.80

CHK'D. BY/DATE [illegible] 10.20.80

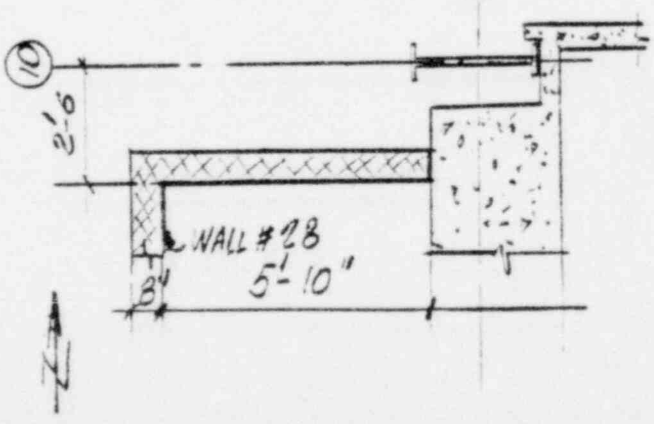
SUBJECT OYSTER CREEK STATION — UNIT #1

WALL #44 - NORTH EAST STAIRWELL FROM TURBINE OPERATING FLOOR NORTH WALL. (6'-6" LONG).

WALL THICKNESS : 8" hollow CONC MASONRY UNITS (STAGGERED).
MORTAR TYPE : M
REINFORCED : NO. REF. DWG: 4514-2



SECT 1-1



POOR ORIGINAL

1/4" = 1'-0

GPU Service

CALC. NO.

SHEET NO. 36 OF 36

DATE

COMP. BY/DATE AT 10.15.80

CHK'D BY/DATE

SUBJECT OYSTER CREEK STATION — UNIT #1

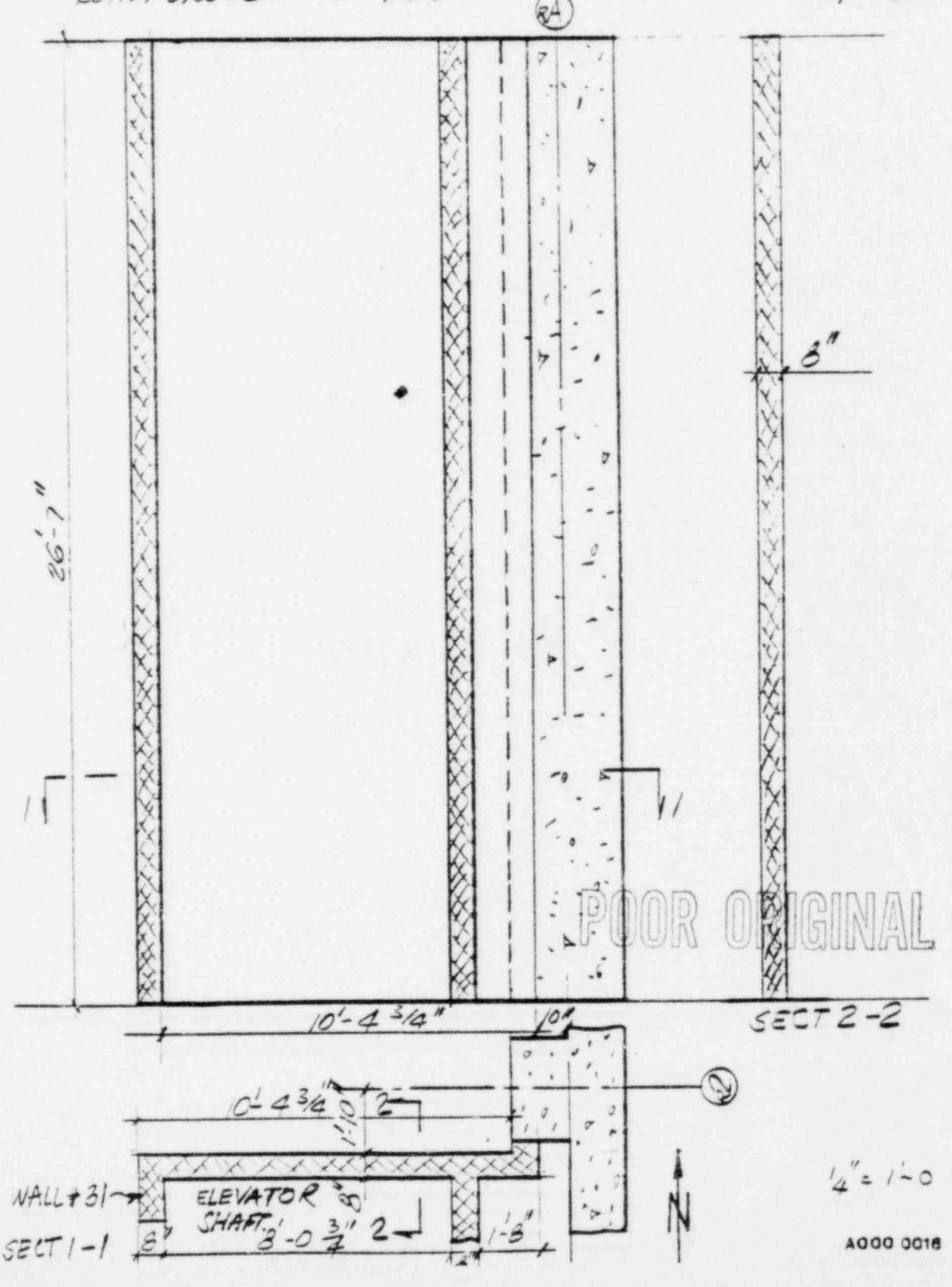
WALL #45 - REACTOR BUILDING ELEVATOR

23'-6" TO 51'-3" NORTH WALL 10'-4 3/4" LONG.

WALL THICKNESS : 8" HOLLOW MASONRY UNITS (STACKED)

MORTAR TYPE : M

REINFORCED : YES REF DWG: 4509-3



1/4" = 1'-0

A000 0016

ENCLOSURE 6

OYSTER CREEK NUCLEAR GENERATING STATION

NRC IE BULLETIN 80-11

CONCRETE BLOCK WALLS

WALL NO.	WALL LOCATION	WALL FUNCTION	REEVALUATION STATUS (SEE NOTE)
1*	Control Room North Wall	D	IV
2	Control Room South Wall	F, S	I
3*	Control Room East Wall	D	I.
4*	Control Room West Wall	D	IV
5	Observation Room Enclosure - South Wall	S, F	II
6	Observation Room, S. E. to N.W.	S, F	II
7	Observat.on Room West Wall	S, F	II
8	Cable Tray Area East Wall	F (partial)	II
9*	Vestibule No. 2 Computer Room, East Wall	PP	III
10*	Health Physics Office - North Wall	PP	III
11*	Health Physics Office - South Wall	PP	III
12*	Health Physics Office - East Wall	PP	III
13*	Contaminated Clothing Area, HP Tech Office South Wall	PP	III
14*	Contaminated Clothing Area -East Wall	PP	III
15	Monitor and Change - South Wall	F	I
16*	Control Room Center Column Block Facing	D	IV
17	Battery Room (A&B) South Wall (West Section)	S,ES ,F	I
18	Battery Room (A&B) West Wall (South Section)	S,F	COMPLETED
19	Electric Tray Room - North Wall	ES	I
20	Electric Tray Room - East Wall	ES	I
21	480V Switchgear Room - North Wall	S, F	I
22	480V Switchgear Room - Center Block Wall	F	I
23	480V Switchgear Room - South Block Wall	F	I

WALL NO.	WALL LOCATION	WALL FUNCTION	REEVALUATION STATUS (SEE NOTE)
24	Turbine Building North East Stairwell - West Wall	F	II
25	Cable Spread Room - West Wall	F, SS	II
26	Cable Spread Room - West section of North Wall	F, SS	II
27	Cable Spread Room - North-South Wall on col. line H	F, SS	II
28	Northeast stairwell from turbine operating floor - West Wall	F, CC	II
29	Reactor Building S/E stairwell to 19'-6" North Wall	F,SS,FC	I
30	Reactor Building S/E stairwell to 19'-6" West Wall	F,ES,FC	I
31	Peactor Building Elevator 23'-6" to 51'-3" West Wall	F, FC	I
32	Rx Building Stairwell 23'-6" to 51'-3" North Wall	F, FC	I
33	Rx Building Stairwell 23'-6" to 51'-3" West Wall	F, FC	I
34*	Office Building 3rd Floor locker room North Wall	PP	III
35*	Shower Room North Wall	PP	III
36	Shower Room West Wall	Incorporate in Wall No. 42	
37*	Toilet No. 4 - North Wall	PP	III
38*	Toilet No. 4 - South Wall	PP	III
39*	Supervisor Shower - North Wall	PP	III
40*	Supervisor Shower - South Wall	PP	III
41*	Supervisor Shower - East Wall	PP	III
42	Corridor No. 5 - East Wall	PP	I
43	Shutdown Heat Exchanger Room - North Wall (knockout section)	SW,F	II
44	Northeast Stairwell from turbine operating floor - North Wall	F, CC	II
45	Rx Building Elevator 23'-6" to 51'-3" North Wall	F, FC	I

WALL NO.	WALL LOCATION	WALL FUNCTION	REEVALUATION STATUS (SEE NOTE)
46*	Monitor & change Room - East Wall	D	IV
47*	Duct chase - South Wall	PP	III

KEY:

* = Walls to be deleted from IE-80-11 Scope (see notes _ and IV)

SW = Shield wall

D = Decorative

F = Fire barrier

S = Security

PP = Personnel partition only

SS = Safety equipment support

ES = Equipment Support (non-safety) other than conduit

FC = Flood control to protect safety equipment

CC = Contamination Control

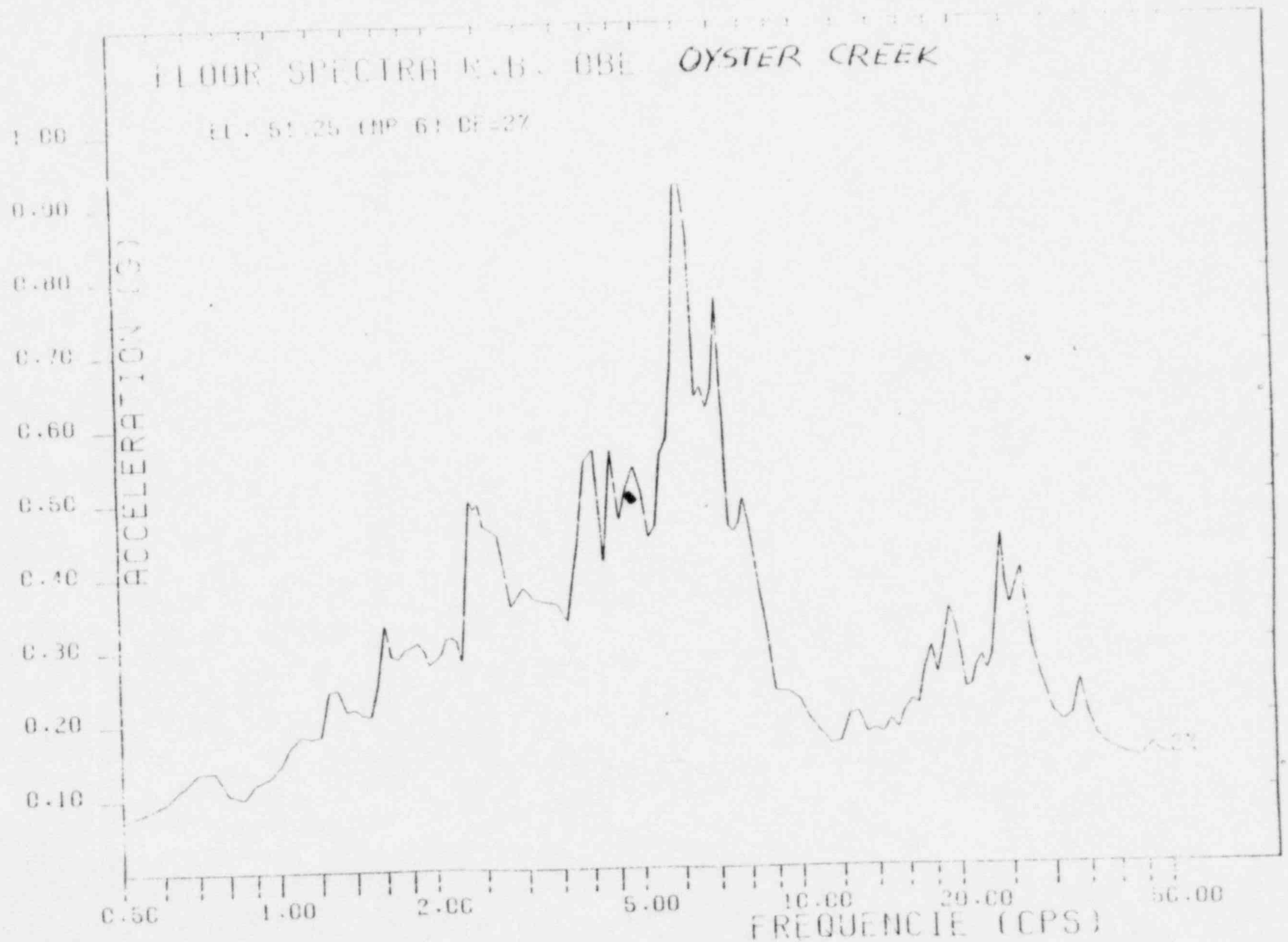
NOTE I: Field survey, sketches and sections, Preliminary Reevaluation, Mathematical Models, Frequency Calculations completed.

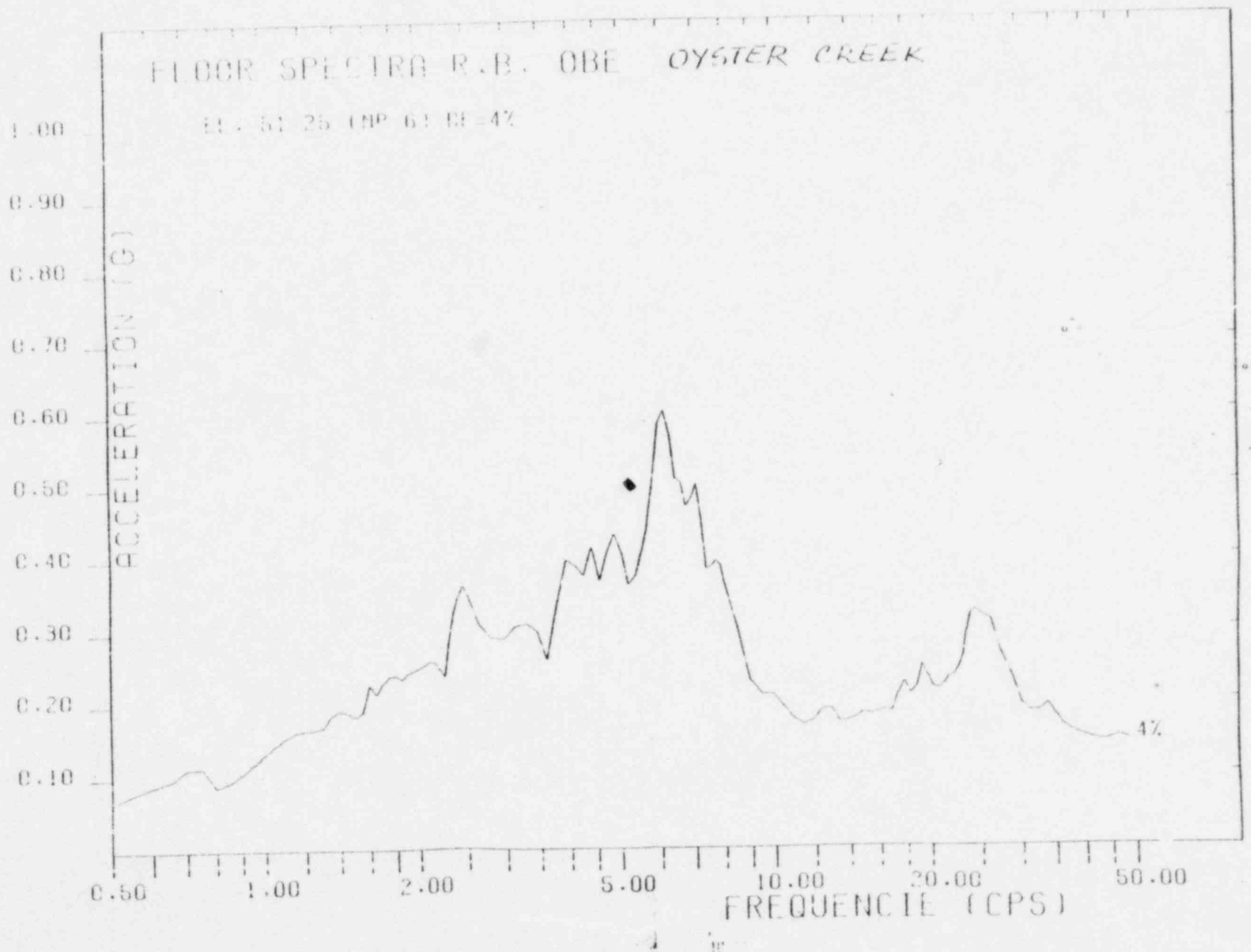
NOTE II: Field survey, sketches and sections, Preliminary Reevaluation completed.

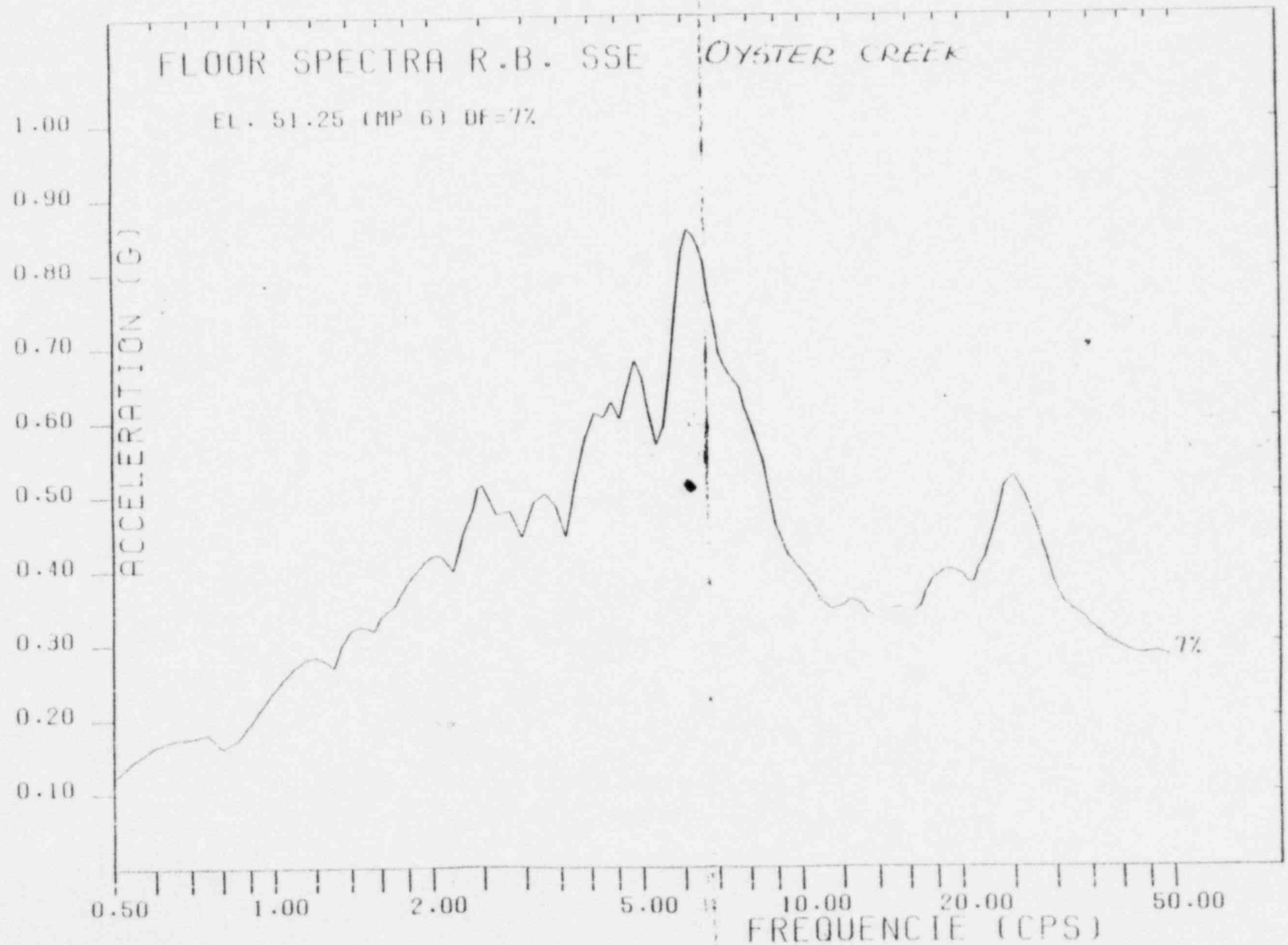
NOTE III: Top portion of these walls that was initially considered to be missile hazard for safety related equipment is scheduled to be removed.

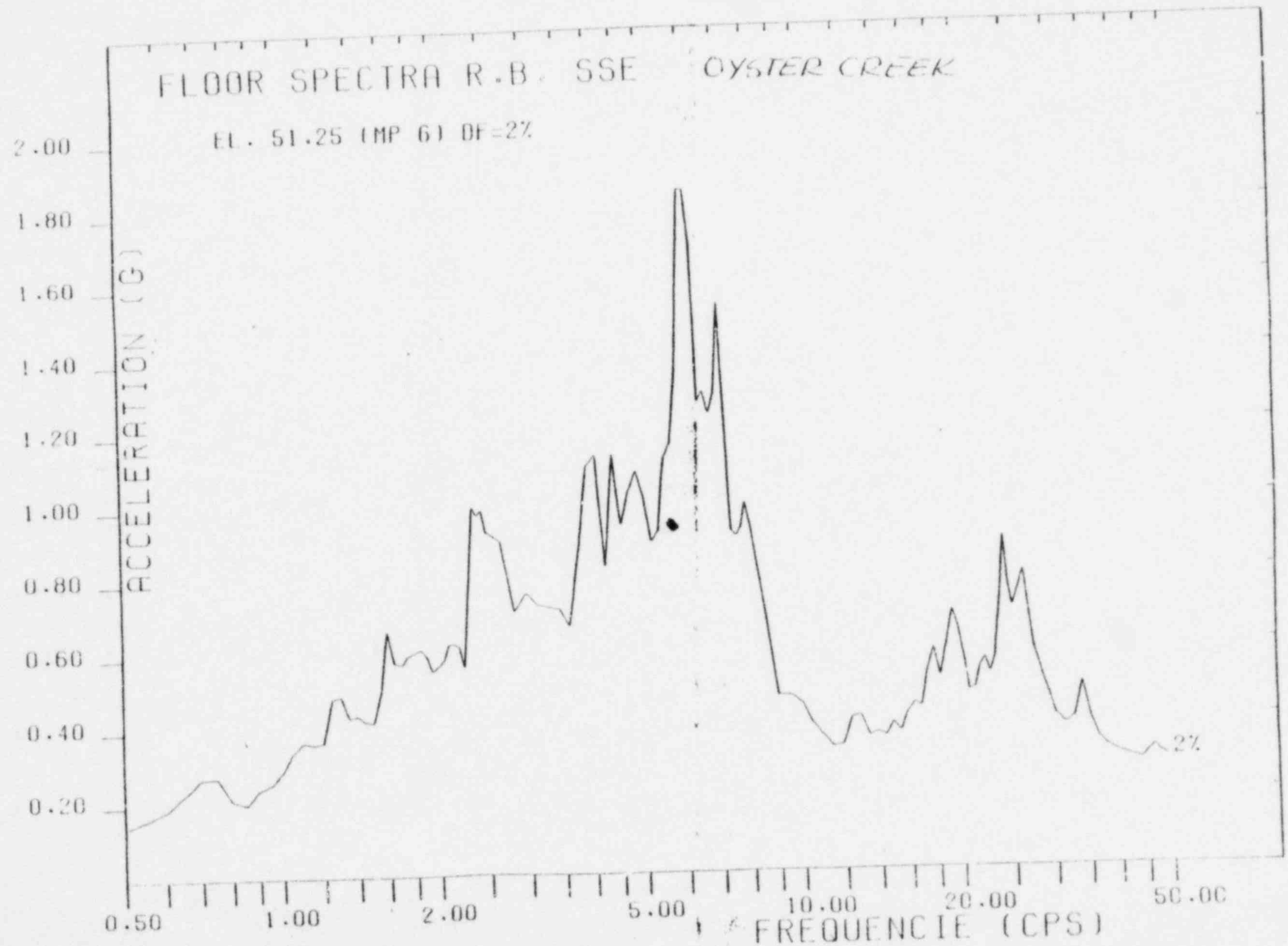
NOTE IV: Decorative tile to be removed.

ENCLOSURE 7

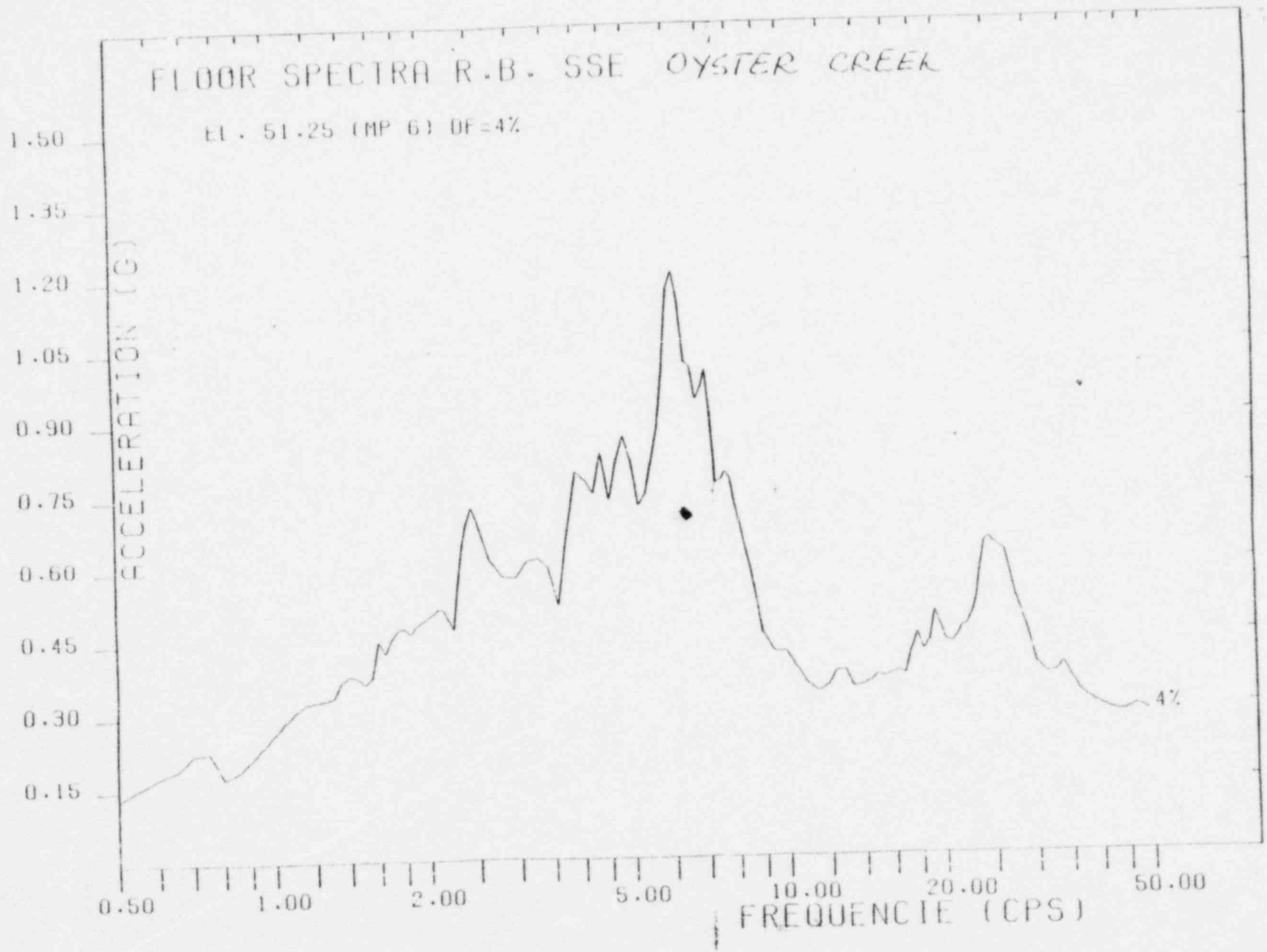

FLOOR SPECTRA R.B. OBE OYSTER CREEK
ACCELERATION (G)
1.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.50
1.00
2.00
5.00
10.00
20.00
50.00
FREQUENCIE (CPS)

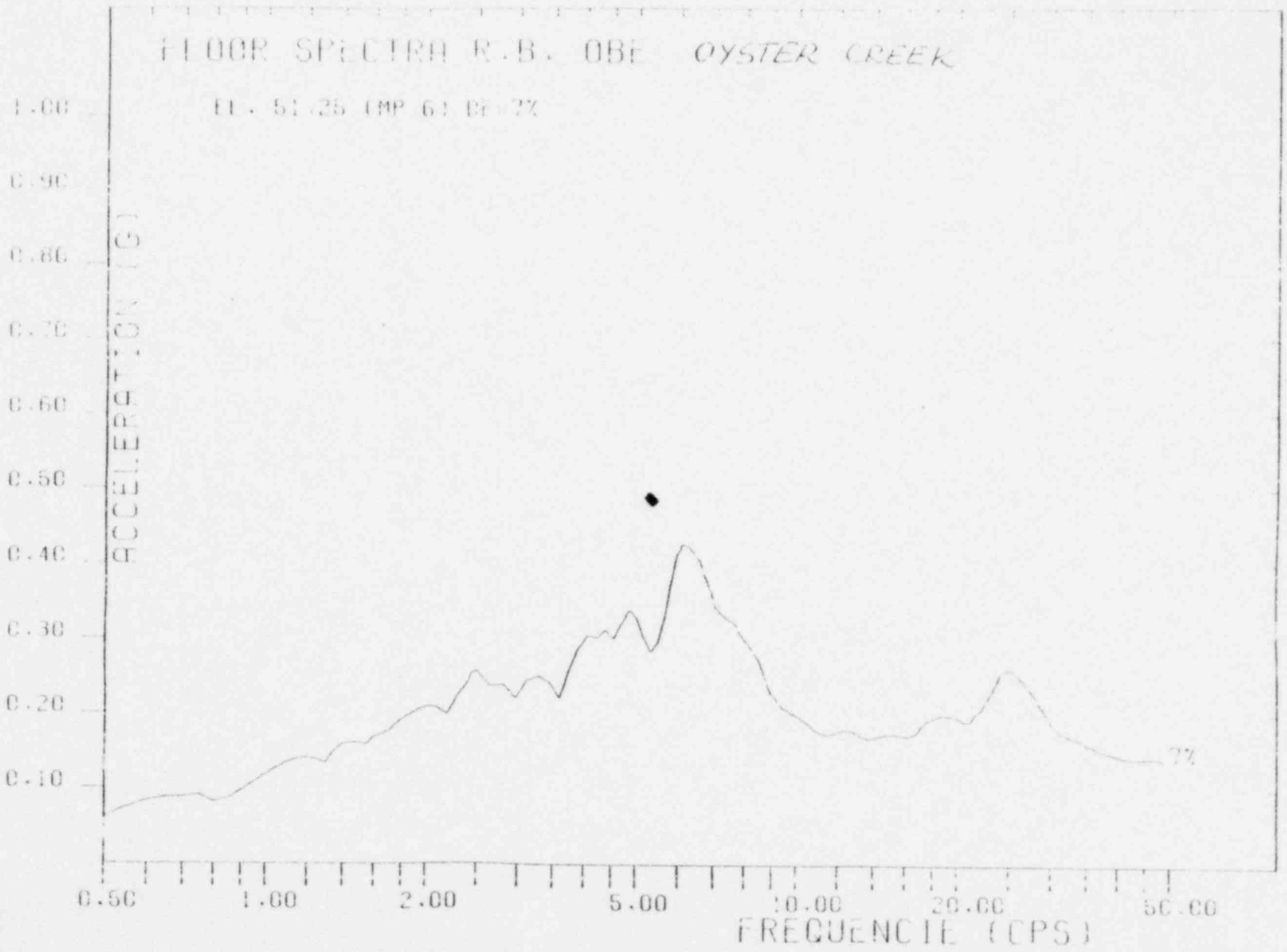

FLOOR SPECTRA R.B. OBE
OYSTER CREEK
ACCELERATION (G)
1.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.50
1.00
2.00
5.00
10.00
20.00
50.00
FREQUENCIE (CPS)
4%


FLOOR SPECTRA R.B. SSE
OYSTER CREEK
EL. 51.25 (MP 6) DF=7%
ACCELERATION (G)
1.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
7%
0.50
1.00
2.00
5.00
10.00
20.00
50.00
FREQUENCIE (CPS)



FLOOR SPECTRA R.B. SSE
OYSTER CREEK
EL. 51.25 (MP 6) DF=2%
ACCELERATION (G)
2.00
1.80
1.60
1.40
1.20
1.00
0.80
0.60
0.40
0.20
2%
0.50
1.00
2.00
5.00
10.00
20.00
50.00
FREQUENCIE (CPS)


FLOOR SPECTRA R.B. SSE OYSTER CREEK
EL. 51.25 (MP 6) DF=4%
ACCELERATION (G)
1.50
1.35
1.20
1.05
0.90
0.75
0.60
0.45
0.30
0.15
4%
0.50
1.00
2.00
5.00
10.00
20.00
50.00
FREQUENCIE (CPS)


FLOOR SPECTRA R.B. OBE OYSTER CREEK
EL. 51.25 (MP 6) DF 7%
1.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
ACCELERATION (G)
0.50
1.00
2.00
5.00
10.00
20.00
50.00
FREQUENCIE (CPS)
7%

ENCLOSURE 8

EVALUATION OF

CONCRETE MASONRY WALL FOR THE

OYSTER CREEK GENERATING STATION

October 27, 1980

Evaluation Techniques

Calculations were based on the following procedures and assumptions:

1. The wall was modeled by Finite Element Model for both Frequency and Static analysis
2. Boundary Condition for the Beam Model was assumed pinned at both top and bottom of the wall
3. Boundary condition for the plate model was assumed pinned at all four sides of the wall
4. The masonry details for the wall shown on J.C.P.&L Drawing 4514-2 were used to determine block dimensions and reinforcing
5. The reinforcing used at the intersecting block was wire mesh at every other course with a minimum projection length of 12 inches
6. Analysis procedure for the wall was based on "Criteria for the re-evaluation of Conc e' Masonry Walls" by Computech Engineering Services, Inc.
7. The wall was evaluated with the following load and load combination

   Service loads D + E

   Factored loads D + E'
8. All attached loads on wall such as piping, conduits and panels are transferred into the wall at the supporting point

Results:

The results of the evaluation are summarized in the following table. Stresses of wall when modeled as plate and simply supported on all 4 edges are within the allowables as given in Table 2 of the Specification.

SUMMARY OF EVALUATION --- WALL No. 18

Wall Location	Battery Room (A&B) West Wall (South Section)
Dimension	Height x Width = 11'-0 x 14'-6
Type of Construction	6" unreinforced running bond ASTM C-90 Block, Type M mortar
Frequency Range, Hz Beam[1] Plate[2]	 10.23 - 13.21 16.73 - 21.60
Response Acceleration - g Beam[1] Plate[2]	 0.41 (SSE) ; 0.22 (OBE) 0.60 (SSE) ; 0.33 (OBE)
Stress - psi (Tension- flexural) Beam[1] Plate[2] - Normal to bed jt. parallel to bed jt. Allowable	 70.8 (SSE) ; 41.1 (OBE) 37.8 (SSE) ; 21.8 (OBE) 20.2 (SSE) ; 11.9 (OBE) 41.5 (SSE) ; 25.0 (OBE)

Notes: 1. Based on wall modeled as a simply supported beam.
2. Based on wall modeled as a simply supported plate.
3. Max shearing stress — 11.6 psi

ENCLOSURE 9

TECHNICAL SPECIFICATION

MASONRY

SECTION 4A

W.O. 2299 Specification
Oyster Creek Nuclear Station

4A. 1. Scope

This subsection covers interior and exterior concrete masonry, and glazed concrete masonry, com plete.

4A. 2. Specification References

Reference to Federal and A.S.T.M. specifications shall mean the latest revision thereto.

4A. 3. Materials

a. Aggregate

Aggregate used in making concrete masonry units shall conform to ASTM Specification C-33 or C-331, except as modified hereinafter. Grading of aggregates as stipulated in Section 7 in ASTM Specification C-33 and testing of lightweight aggregates for drying shrinkage as stipulated in Section 6 (a) in ASTM Specification C-331, will not be required.

b. Anchors and Ties

Anchors and ties shall be of approved design and shall be zinc-coated ferrous metal, of the types noted below. Zinc coating of anchors and ties shall conform to ASTM Specification A-153, Class B1, B2 or B3 as required.

(1) Wire mesh ties for anchorage of concrete-masonry-unit partitions to exterior walls shall be made of steel wire not lighter than 0.0625 inch (16 gage) in diameter and shall be a minimum of 3 inches wide, of lengths as indicated and have a maximum 1/2 inch mesh.

(2) Rigid steel strap anchors shall be 1 inch by 1/4 inch with ends turned down not less than 2 inches of lengths as indicated.

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(3) Cavity wall ties shall be rectangular in shape at the required length not less than 4 inches wide made of nominal 3/16 inch diameter wire. The wire shall be commercial bronze or copper clad steel, the copper constituting at least 25% of the copper clad steel wire, and shall be crimped in the center to provide an effective moisture drip.

(4) Concrete masonry walls abutting dovetail slots located in concrete walls or column shall have dovetail anchors spaced in alternate concrete masonary courses. Anchors shall be galvanized, 16 gauge, 1-1/2 inches wide, and shall be bent down to project into the first block cell.

(5) Furred walls ties shall be 16 gauge, by 1-1/4 inch galvanized steel, 1-1/2 inches long plus 1-3/4 inch bends.

(6) Masonry nails for fastening steel corrugated wall ties to face of concrete or masonry shall be 1/4 inch by 2 inches Parker-Kalon galvanized hardened masonry nails or equal approved by the Engineer.

c. Joint Reinforcement

Joint reinforcement for concrete masonry units shall be made of zinc-coated wire, conforming to ASTM Specification A-82. Longitudinal wires may be smooth or deformed and shall be not lighter than 3/16 inch in diameter. Cross-wires shall be not lighter than 0.1483 inch (9 gage) in diameter. The distance between contacts of cross-wires with longitudinal wire, shall not exceed 6 inches for smooth longitudinal wires and 16 inches for deformed longitudinal wires. The spacing of the longitudinal wires in joint reinforcement shall be 1-1/2 inches less than the nominal width of the block. Cross-wires may be placed between and in the same plane as, deformed longitudinal wires, but shall intersect above or below plain longitudinal wires, with ends of cross-wires extending not more than 1/8 inch beyond the outer sides of the longitudinal wires. Joint reinforcement shall be furnished in flat sections ranging from 10 to 20 or more feet in length. Reinforcement shall not be furnished in rolls.

d. Vertical Reinforcement

Vertical reinforcement steel in concrete masonry shall be placed as indicated on the drawings and shall comply with the requirements specified under the Concrete Section of this specification. Vertical cells to be filled shall have alignment maintaining a clear unobstructed continuous vertical cell. Vertical reinforcement shall be continuous and rigidly secured at top and bottom and at intervals as necessary to hold in proper position. Splices where used, shall be 30 diameters in length. Splices shall be centered in blocks. Units containing reinforcement shall be solidly filled with class A [illegible] concrete, as specified under Section, "Concrete".

e. Horizontal Reinforcement

Where shown on the drawings, horizontal reinforcement shall be extra heaby "Dur-O-Wall" or equal as approved by the Engineer.

f. Concrete Masonry Units.

Concrete masonry units shall be of modular dimensions and shall include all closers, jamb units, headers and special shapes and sizes required to complete the work as indicated. Vertical external corners shall be bullnose. Units exposed to view shall be of the same appearance and shall be cured by the same process.

Units shall be free of deleterious matter that will stain, paint or corrode metal. Concrete masonry units that have been subjected to a saturated steam pressure of not less than 120 pounds per square inch for 5 hours or more, may be used as soon as cooled. Units that have not been subjected to such steam pressure shall be cured for 28 days or more. Units shall be delivered to the job site in an air-dry condition. The mositure content at time of delivery shall not exceed 30% of maximum possible moisture content.

(1) Load bearing concrete masonry units conforming to ASTM Specification C90, except as modified herein, shall be used in exterior walls and all interior walls containing vertical reinforcing steel.

(2) Non-load bearing concrete masonry units, conforming to ASTM Specification C129, except as modified herein, shall be used in interior non-load bearing interior work, except that load bearing units may be used.

(3) Fire-resistant concrete masonry units for stair enclosures and fire walls shall be of the type that will give 2 hours fire rating, and compressive strenght of block and web and shell thickness shall conform to the requirements of the Underwriters' Laboratories for 2 hours fire rated block. To secure fire resistance rating, cells of blocks may be filled with concrete as the blocks are laid.

g. Glazed Concrete Masonry Units

Glazed concrete masonry units shall be Spectra-Glaze units as manufactured by United Glazed Products, Inc., Baltimore, Maryland or equal approved by the Engineer. The finished and exposed surfaces shall be covered at the point of manufacture with a

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thermosetting resinous compound and graded silica sand, which is cast onto the base block by an external heat-polymerizing process. The block shall be manufactured as specified herein before.

(1) The facing material, after polymerization, shall conform to the requirements of ASTM Specification C-126 Grade G. This facing shall have a flame spread not in excess of 30 as determined by the tunnel test in accordance with ASTM Specification E-84. The glazed facing shall be uniformly smooth and shall not show the irregular contours of the base block on the surface or along the edges of the glazing. The facing shall retu… 3/8 inch over the ends of the block, forming a lip not less than 1/16 inch thick, thus making possible the use of a 1/4 inch exposed mortar joint.

(2) Color shall be as selected by the Engineer.

(3) The facing of glazed masonry units shall show no effect when subjected to the following chemicals for the period of time indicated:

Glacial Acetic Acid	24 hours
Citric Acid (10%)	24 hours
Sulphuric Acid (10%)	24 hours
Tannic Acid (5%)	24 hours
Trisodium Phosphate (5%)	24 hours
Hydrogen Peroxide (3%)	24 hours
Detergent "Tide" (10%)	24 hours
Vegetable Oil	24 hours
Undiluted NH^4OH	12 hours
Ethyl Alcohol	3 hours
Blue Black Ink	1 hour
Tincutre of Iodine	1 hour

(4) There shall be no visible change of color, gloss or texture when glazed masonry units are exposed to 500 hours of strong ultra violet radiation, consisting of cycles of 102 minutes of ultra violet rays alone, followed by 18 minutes of ultra violet plus water spray.

(5) The glazed block facing shall be uniformly smooth and not show the irregular contours of the base block surface or the base block edge. The facing shall be free from chips, cracks, crazes, blisters, blooming, crawling, peeling, holes or other imperfections detracting from the appearance of the wall when viewed from a distance of five (5) feet at right angles to the wall. All exposed external corners shall be bullnose.

(6) Glazed block units shall be manufactured with a maximum face dimension tolerance of 1/16th of an inch, thus permitting units to be laid with 1/4 inch exposed bed joints on the glazed block surface while maintaining standard 3/8 inch bed.

(7) As defined in Federal Standard 141, Method 6192, the wear factor shall not exceed 130, using standard Taber Abraser Research Model with CS-17 calibrase wheel and a one thousand gram weight for 500 wear cycles.

(8) Glazed Block shall demonstrate the glazed surface resistance to impact by permitting a one inch diameter steel ball to be dropped a distance of two feet without cracking or shattering.

(9) The units shall be delivered to the site in individual cartons, stored under cover until placed in the wall and handled in such manner as to preclude damage to finished faces, ends or edges.

(10) The glazed block supplier shall submit a notarized certificate stating that the units supplied for this project fully conforms to all the specification requirements set forth herein.

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h. Mortar

Mortar shall comply with the proportion specification for Type M as set forth in ASTM Specification C270, except that when tested for water retention the mortar shall have a flow, after suction, of 75% or more when mixed to an initial flow of 125 to 140%. When tested for compressive strength, the water retention requirements for mortar stipulated in ASTM Specification C270 shall apply. The Subcontractor shall furnish a certified copy of laboratory tests as evidence that the mortar used in the work meets the requirements of the proportion specification.

4A.4. SAMPLES:

The following samples of materials proposed for use shall be submitted to the Engineer and his approval received before materials represented by the samples are delivered to the job site.

1. Anchors and Ties

   Two (2) of each type proposed for use.

2. Glazed Concrete Masonry Units

   Two (2) color samples of each type proposed for use.

3. Joint Reinforcement

   One (1) piece of eacy type of reinforcement, 18 inches long, showing at least two (2) cross-joints.

4A.5. SAMPLES FOR TESTING:

a. A sample of five (5) individual and whole concrete masonry units representative of the manufacturer's product, whose units are proposed for use, shall be selected after cooling and/or curing, at the point of manufacture, and tested to verify compliance with applicable ASTM requirements.

b. Sample units shall prove under test to be free from cracks or other structural defects and to have been manufactured with the same type and quality of aggregate and cured and dried by the same procedures as those to be employed in producing units for use in the work.

c. Shop Drawings

Shop drawings of the Glazed Concrete Masonry Unit installation shall be submitted for approval. Drawings shall show spec[illegible]l shapes, shape numbers, applicable dimensions.

4A.6. TESTS

Tests shall be made by an approved commercial testing laboratory at the expense of the Subcontractor.

4A.7. ERECTION

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a. General

Masonry shall be plumb, true to line, courses level, and even. Masonry shall not be erected when the ambient temperature is below 35°F, except by written permission of the Engineer. No frozen work shall be built upon. No unit having a film of water or frost on its surface shall be laid in the walls. Masonry shall be protected from freezing for 48 hours after being laid. Masonry erected during arid weather when the ambient air has a temperature of more than 99°F in the shade and relative humidity of less than 50% shall be protected from direct exposure to wind and sun for 48 hours after installation. Spaces around metal door frames and other built-in items shall be solidly filled with mortar. Anchors, wall plugs, accessories, flashings and other items required to be built in with masonry required to accommodate the work of others shall be done by masonry mechanics. When directed by the Engineer, all exposed concrete masonry units shall be kept continuously moist by applying a light water fog for a period of 24 to 48 hours after erection. Units shall be free from chipped edges or other imperfections detracting from the appearance of the finished work.

b. Laying Concrete Masonry Units

Masonry shall be laid plumb, true to line, with level courses accureately spaced and with each course breaking joints with the course next below. Or in stack bond so that vertical joints between units will be over the joints of the units in the course immediately below and vertical joints between units will be in alignment from top to bottom of the wall. Each unit shall be adjusted to its final position in the wall while mortar is still soft and plastic. Any unit that is disturbed after mortar has stiffened shall be removed and relaid with fresh mortar. Bond pattern shall be kept plumb throughout. Corners and reveals shall be plumb and true. Walls or partitions abutting columns or walls shall be anchored thereto. Concrete masonry units shall be wetted before laying. Mortar joints shall be approximately 3/8 inch wide. Bed joints shall be such that 1 unit plus joint shall be 8 inches. Mortar joints in piers, columns and pilasters and starting courses on concrete slabs, on solid foundation walls, and on beams shall be full bedded under both face shells and webs. Other joints shall have full mortar coverage on horizontal and vertical face shells, but mortar shall not extend through the unit on the web edges. Each course shall be bonded at corners. Jamb units shall be of the shapes and sizes required to bond with wall units. No cells shall be left open in face surfaces. Masonry unit walls or partitions supporting plumbing, heating or other fixtures and voids at door and window jambs and other spaces requiring grout fill shall be full t..ed in mortar to prevent leakage and filled solid with mortar mixed to pouring consistency.

1. Partitions

Partitions shall be continuous from floor to underside of floor or roof construction above, except as indicated on the drawings.

2. Lintels in Masonry Unit Partitions

Lintels in masonry unit partitions shall be constructed of specially formed lintel or U-shaped units filled with Class A concrete, as specified under section "Concrete", using coarse aggregate of 1/2 inch to No. 4 nominal size, and shall be reinforced as indicated. However, not less than two (2) No. 4 bars, the full length of the lintel, shall be provided. Lintels shall extend at least 8 inches beyond each side of the opening.

3. Furring

Furring shall be 2" concrete masonry units. Unless otherwise indicated, the furring units shall be returned on the jambs and other reveals, and around projections that are indicated on the drawings as furring with masonry. Furring shall be suitably anchored to masonry walls, concrete or steel work with built-in metal anchors spaced not over 2 feet on centers horizontally and vertically and placed so as to coincide with the joints in masonry furring. United furring shall be spot bedded in mortar against the backing masonry. Furring at columns and for pipe-space enclosures shall not be bonded to other masonry but shall be anchored thereto by metal ties.

5. Cavity-Wall Construction

In walls of cavity construction, unless otherwise indicated, the facing and backing masonry shall be completely separated by a continuous air space not less than 2 or more than 3 inches wide, except for returns at jambs of openings. The inner wythes shall be securely tied together by cavity-wall ties staggered and spaced not to exceed 36 inches apart horizontally and 16 inches apart vertically. Additional cavity-wall ties shall be placed within 8 inches of the jambs of all openings except where the wythes are bonded together with masonry returns at jambs. The air space between the facing and backing wythes shall be kept clear and clean of mortar droppings by temporary wood strips laid on the wall ties and carefully lifted out before the next row of ties or anchors is placed.

a. Weep holes shall be provided 32 inches apart on center in mortar joints of exterior wythe along the bottom of the cavity over foundations, bond beams, and other water stops in wall, by placing short lengths of well greased No. 10, 5/16 inch nominal diameter braided cotton sash cord in the mortar and withdrawing these pieces of cord after the wall has been completed. Other methods of providing such weep holes may be used subject to approval of the Engineer.

b. Laying Glazed Concrete Masonry

Unless otherwise indicated, units shall be laid in courses accurately spaced, with a story pole in stack bond so that vertical joints between units will be located over the joints of the units in the course immediately below and vertical joints between units will be in alignment from botton to top of wall.

c. Chases and Raked-Out Joints

Chases and raked-out joints shall be kept free from mortar or other debris.

d. Cutting at Job

In order to insure true, sharp corners and edges, all cutting of masonry units exposed to view shall be performed with a high speed saw.

e. Surfaces to be Painted

Workmanship shall be neat, with joints well arranged to produce a proper appearance. Remove any surplus mortar from exposed face of block and leave surface ready for painting.

f. Cold Weather Installation

(1) When the outside air temperature is between 35°F and 32°F, masonry units shall be kept completely covered and free from ice and snow. The mixing water or sand for mortar shall be heated to a temperature between 70°F and 160°F. The air temperature on both sides of the masonry shall be maintained above 40°F for a period of at least 72 hours. The Bidder shall submit for approval the methods he proposes to use for protecting the masonry against low temperatures. Building upon frozen work is prohibited.

g. Joints

Joints, except those to be filled with sealant or otherwise indicated, shall be tooled slightly concave as soon as possible after mortar has attained its initial set, with a device of as long length as practicable so that the mortar will be thoroughly compacted and pressed against the edges of the units. Tooling shall not be done until after the mortar has taken its initial set. Joints for exterior glazed masonry units at entrance lobby shall be raked out 3/8 inch and tooled slightly concave. The following joints on the weather side of exterior masonry walls shall be raked out 3/4 inch and left ready for sealant:

(1) Joints between metal frames and masonry.

(2) Other joints where indicated on the drawings.

h. Unfinished Work

Unfinished work shall be stepped back for joining with new work. Toothing may be resorted to only when specifically approved. All loose mortar shall be removed and exposed joints shall be thoroughly cleaned before laying new work. Surfaces of masonry not being worked on shall be properly protected at all times during construction operations. When rain or snow is imminent and the work is discontinued, the tops of exposed masonry walls shall be covered with a strong, waterproof membrane well secured in place. Adequate provisions shall be made during construction to prevent damage by wind.

i. Mortar

Mortar that has stiffened because of chemical reaction, due to hydration, shall not be used. Except as specified below, mortar shall be used and placed in final position within 2-1/[illegible] [illegible]urs after mixing, where air tempe[illegible]re is 80°F or higher, and within 3-[illegible] hours after mixing where air temper[illegible]e is less than 80°F. Mortar not [illegible] within these time intervals shall [illegible] discarded. When cement or cements used in the mortar have been tested and the observ[illegible] time of initial set, as determined un[illegible] ASTM Specification C266 has been ascer[illegible]tained, the time interval during which the mortar must be placed in final position may be determined as follows:

Air temperature in Degrees F.	Time Interval After Mixing
80 or higher	Time of initial set minus 1 hour
Less than 80	Time of initial set minus 1/2 hour

Mortars that have stiffened within the time interval as determined above, because of evaporation of moisture from the mortar, may be retempered by adding water as frequently as needed to restore workable consistency.

4A. 9. Shrinkage Cracking Control

Shrinkage cracking in concrete masonry unit construction shall be controlled by combinations of control joints and joint reinforcing in strict accordance with the details indicated on the drawings and as specified hereinafter.

a. Joint Reinforcement

Joint reinforcement shall be installed in every other course. Joint reinforcement at openings shall extend not less than 24 inches beyond the end of sills and lintels. Reinforcement shall be lapped 6 inches or more and the lap shall contain at least one cross wire of each piece of reinforcement. Joint reinforcement shall be accurately formed around corners and at wall intersections.

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b. Control Joints

Control joints provided at the locations indicated in the concrete masonry work shall be constructed by using open end stretcher units. On the exterior face of the wall, the control joint shall be raked to a depth of 3/4 inch and left ready for calking. Control joints on exposed to view or painted interior walls or partitions shall be raked to a depth of 1/4 inch and shall not be calked. Sealing of control joints is included under Section "CAULKING AND SEALING section of these specifications.

c. Flashings

Flashings built into masonry walls shall be pressed down into a bed of fresh mortar and be covered with a full bed of mortar to receive the next course of masonry.

d. Jambs

Three (3) cells of masonry units at door jambs shall be filled with mortar full height.

4A. 10. Pointing and Cleaning

Before completion of the work, all masonry surfaces shall be left clean, free of mortar daubs and with tight mortar joints throughout. Metal cleaning tools and metal brushes shall not be used. All joints with imperfections shall be raked back 1/2 inch and pointed and tooled to match existing joints. Glazed Concrete Masonry units which are damaged, marked or chipped shall be removed and replaced with undamaged units.

After pointing mortar has set and hardened, all exposed face brickwork shall be wetted and then cleaned with a solution of 10% by volume of commercial muriatic acid, applied with stiff brushes and immediately after cleaning the surfaces shall be thoroughly rinsed down with clean water.

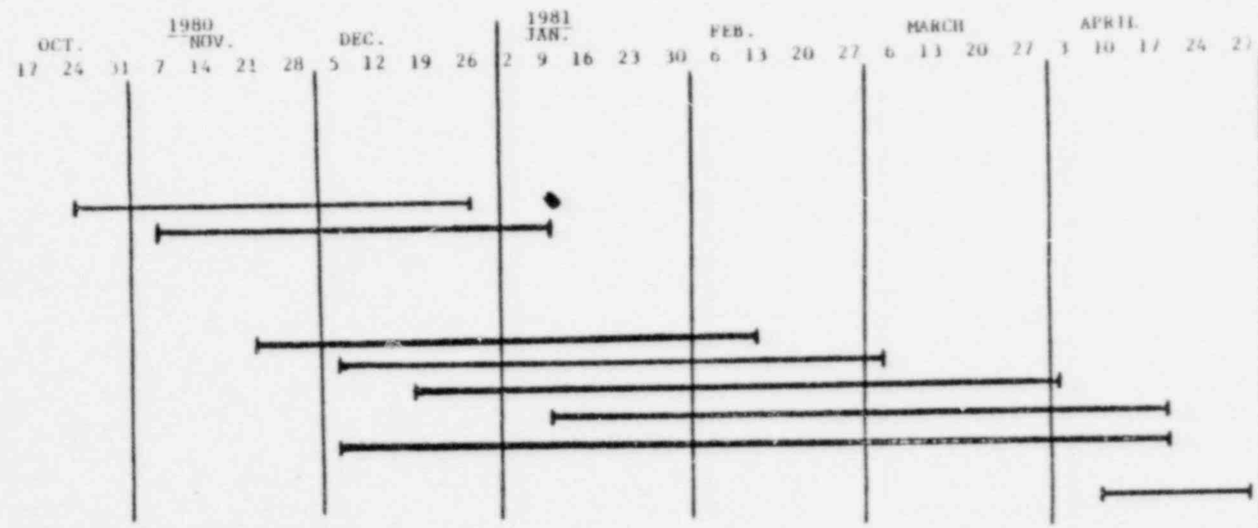
Glazed Concrete Masonry units shall be cleaned with a soap powder and clean water solution. Acid solutions shall not be used.

Excess mortar shall be removed with wooden paddles. When cleaning has been completed, units shall be washed down with clean water.

ENCLOSURE 10

OYSTER CREEK NUCLEAR STATION

SCHEDULE FOR CONCRETE MASONRY WALL REEVALUATION



I. FREQUENCY ANALYSIS

1. Modeling of Wall
2. Input Data

II. WALL ANALYSIS

1. Refined Model
2. Input Data
3. Stress Calculation
4. Wall Analysis
5. Checking

III. Final Report