UNITED STATES
NUCLEAR REGULATORY COMMISSION
WASHINGTON, D. C. 20555

OCTOBER 29 1979

Docket No. 50-344

LICENSEE: Portland General Electric Company (PGE)

FACILITY: Trojan Nuclear Plant

SUMMARY OF MEETING HELD ON OCTOBER 18 AND 19, 1979 WITH PORTLAND GENERAL ELECTRIC COMPANY AND BECHTEL TO DISCUSS THE TROJAN CONTROL BUILDING MODIFICATIONS

On October 18 and 19, 1979, the NRC staff met with representatives of Portland General Electric Company (PGE) and Bechtel to discuss the proposed Trojan Control Building modifications.

A list of attendees is shown in Attachment 1.

At this meeting, PGE submitted preliminary written responses to 19 of the 48 questions and requests for information propounded by the NRC staff in letters dated September 14, 20, 28 and October 2, 1979. These draft responses are shown in Attachment 2.

The status of the NRC Plant Systems Branch (PSB) questions and associated responses are as follows:

09-14-79 Letter	Status
1	Acceptable
2	Acceptable
3	Should add commitment to use of fire retardant wood
4	Discussed. No writte draft available.

09-28-79 Letter	Status
1	Acceptable
2	Conditionally acceptable. Answer makes reference to question 7 for which no response is as yet available.
3	Clarification required that differential pressure could be maintained under accident conditions or a Tech Spec waiver should be requested with appropriate basis furnished.

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4	PGE should make it clear that fire watch will be used regardless of use of fire retardant wood.
5	Acceptable
6	Acceptable
7	Discussed. No written draft available.

The following structural questions were discussed (asterisk indicates draft answer is contained in Attachment 2):

<u>09-14-79 Letter</u>: 8, 9

<u>09-20-79 Letter</u>: 2*, 3, 4, 5, 6

Note: 09-28-79 letter contained seven PSB questions - no structural questions.

<u>10-02-79 Letter</u>: 1, 2, 3, 4, 6, 7, 8, 10, 11, 12, 13, 14, 15, 17, 19 20, 21, 22, 23, 24, 25

The balance of the structural responses (in draft form) are attached, and were not discussed.

PGE indicated that formal responses to all PSB requests would be filed by October 26 or shortly thereafter.

The NRC staff indicated that comments on the draft structural responses will be made during the week of October 22.

There will probably be another meeting similar to this one to discuss written draft responses to the remaining 31 items when available.

Charles Trammell

Charles Trammell, Project Manager
Operating Reactors Branch #1, DOR

Attachments:
1. List of Attendees
2. Draft Responses

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Docket Files
NRC PDR
Local PDR
ORB1 Reading
NRR Reading
H. Denton *
E. Case *
D. Eisenhut*
B. Grimes *
R. Vollmer *
J. Miller *
L. Shao
W. Gammill *
G. Zech *
A. Schwencer
D. Ziemann *
P. Check *
G. Lainas *
D. Davis *
NRC Staff Participants
T. J. Carter *
T. Ippolito *
D. Crutchfield*
R. Reid *
V. Noonan
G. Knighton *
D. Brinkman *
P. T. Kuo *
Project Manager
OELD
OSD (3)
S. Showe, I&E*
C. Parrish/P. Kreutzer *
R. Fraley, ACRS (16)
TERA
M. Miller, ASLB

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Portland, Oregon 97232

Ronald W. Johnson, Esquire
Corporate Attorney
Portland General Electric Company
121 S.W. Salmon Street
Portland, Oregon 97204

Mr. Jack W. Lentsch, Manager
Generation Licensing and Analysis
Portland General Electric Company
121 S.W. Salmon Street
Portland, Oregon 97204

Columbia County Courthouse
Law Library, Circuit Court Room
St. Helens, Oregon 97501

Director, Oregon Department of Energy
Labor and Industries Building, Room 111
Salem, Oregon 97310

Dr. Hugh D. Paxton
1220 41st Street
Los Alamos, New Mexico 87544

Michael Malmrose
U. S. Nuclear Regulatory Commission
Trojan Nuclear Plant
P. O. Box 0
Rainier, Oregon 97048

Dr. Kenneth A. McCollom, Dean
Division of Engineering,
Architecture and Technology
Oklahoma State University
Stillwater, Oklahoma 74074

Mr. Eugene Rosolie
Coalition for Safe Power
215 S.E. 9th Avenue
Portland, Oregon 97214

Richard M. Sandvik, Esquire
Frank W. Ostrander, Jr.
Counsel for Oregon Dept. of
Energy
500 Pacific Building
520 S.W. Yamhill
Portland, Oregon 97204

Maurice Axelrad, Esquire
Lowenstein, Newman, Reis,
Axelrad and Toll
Suite 1214
1025 Connecticut Avenue, N.W.
Washington, D. C. 20036

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Ms. Nina Bell
728 S.E. 26th Street
Portland, Oregon 97214

*Less Attachment 2

ATTACHMENT 1

TROJAN CONTROL BUILDING MEETING

OCTOBER 18 AND 19, 1979

NRC Staff
C. Trammell
J. Gray
D. Persinko
V. Noonan
F. Clemenson
J. E. Knight
K. Herring
A. Hafiz

PGE
D. Broehl
T. Bushnell
R. Johnson
L. Erickson

Bechtel
W. White
B. Sarkar
K. Gross
R. Anderson

Shaw, Pittman, Potts & Trowbridge
B. Churchill
P. Harvey

Lowenstein, Reis, Neuman, Axelrad & Toll
M. Axelrad
A. Carr

Hanson, Holley & Biggs
M. Holley, Jr.

NRC Questions (9/14/79)

10/16/79

DRAFT

1. Provide a detailed description of how the equivalent diameter was determined which was used in computing the penetration of the dropped washer into the steel cover plate for cable trays.

2. Provide a drawing which illustrates the projected area used for computing the equivalent diameter.

Answer:

An evaluation of the postulated drop of a plate washer on the steel cover trays was provided in Licensee's response dated September 5, 1979 to Systems Branch Question 11. In the equation used, the term "D" is the diameter of the missile. For an irregularly shaped missile, such as the corner of the plate washer, an equivalent diameter must be used in the analysis.

The equivalent diameter is taken as the diameter of a circle with an area (A) equal to the circumscribed contact area or projected frontal area of the noncylindrical missile. (Reference: page 2-4, Bechtel Topical Report BC-TOP-9A, Rev. 2).

The contact area (A) is the plate thickness (T) times the arc length (L) of the rounded portion of the plate washer.

The arc length (L) is the length of the rounded edge, or one fourth the circumference of a circle of that radius (R).

Plate Washer thickness (T) = 2.375 in.

Radius of rounded corner (R) = 1.5 in.

$$L = \frac{2\pi R}{4} = \frac{2\pi(1.5)}{4} = 2.36 \text{ in}$$

$$A = TL = (2.375)(2.36) = 5.6 \text{ in.}^2$$

$$D = \sqrt{\frac{4A}{\pi}} \quad \sqrt{\frac{4(5.6)}{\pi}} = 2.67 \text{ in.}$$

The attached Fig. 2-1 shows the projected area used for computing the equivalent diameter of the plate washer impact.

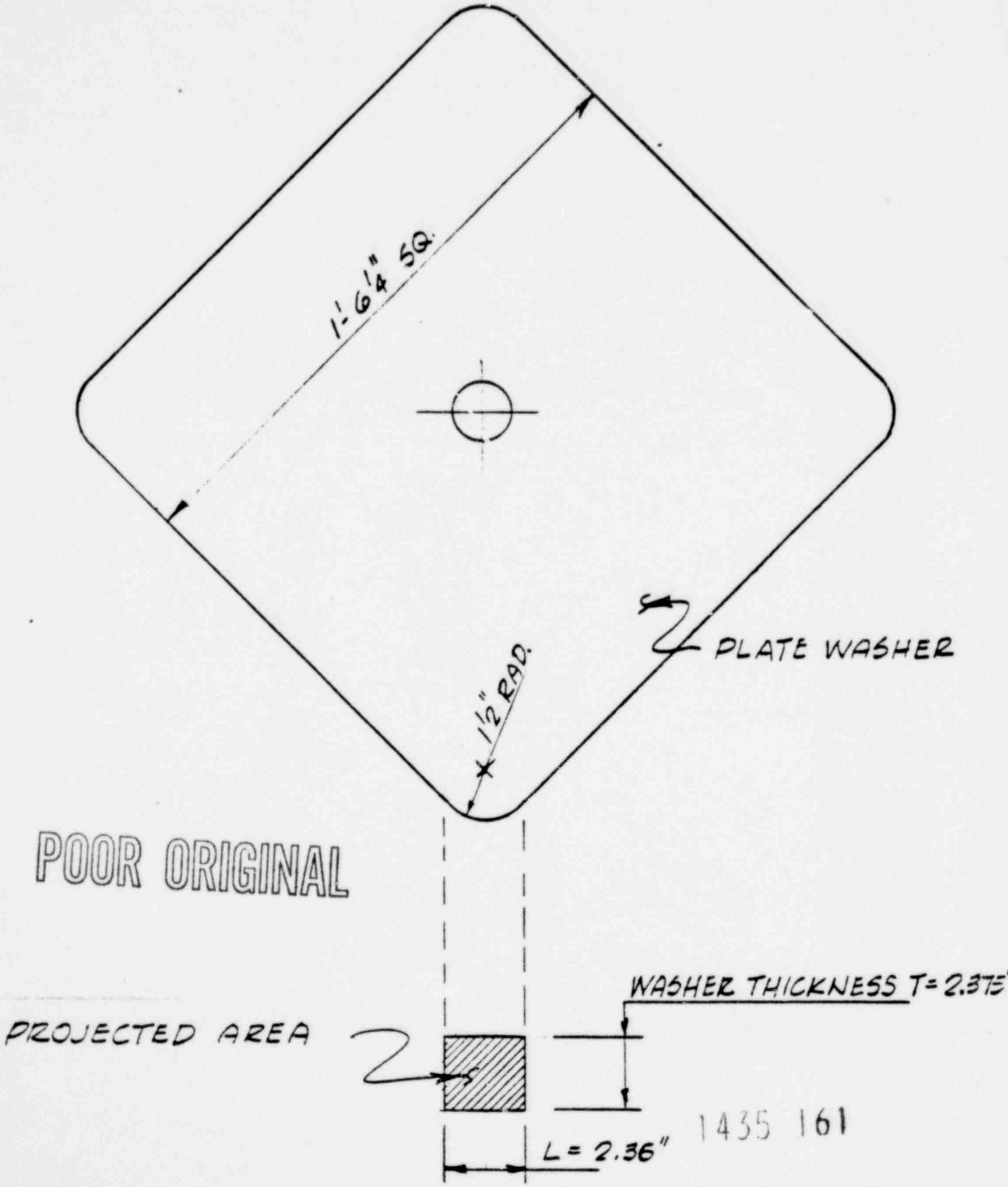

1'-6¼" SQ.
1½" RAD.
PLATE WASHER
WASHER THICKNESS T=2.375"
PROJECTED AREA
L=2.36"

FIGURE 2-1

Provide a listing of all areas containing safety-related cables or equipment in which wood framing will be used during the modification work.

Answer:

Wood will be used during the modification program for form material for placing concrete for the new walls along column lines N, N' and R, and along column line Q as follows:

a) At the new N line wall up to approximately el. 95'3".

b) At the new R line wall up to approximately el. 77', and where grouting behind the steel plate from approximately el. 77' to approximately el. 97'3".

c) At the new N' line wall up to el. 65'.

d) At the new locker room doorway at el. 45' along column line Q.

Within the above areas the following locations where wood forming will be used contain safety related cables or equipment:

1) In the Electrical Auxiliaries Room along column Line N around the equipment hatch and around the columns at the intersection of column lines R and 41.

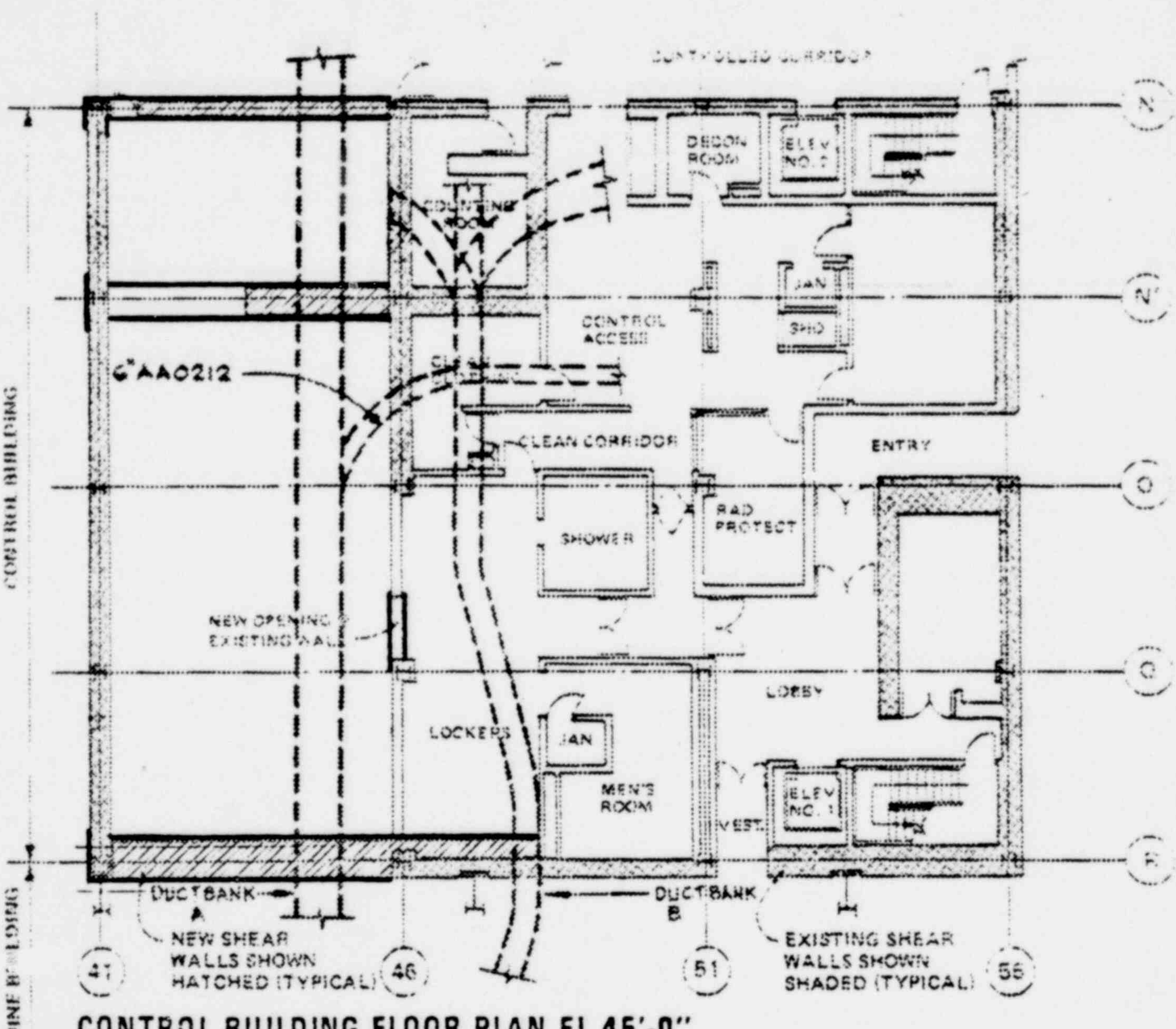
2) On the east (outside) side of the N line wall, at approximate el. 72' around the battery room exhausts.

3) On the west side of R line wall between elevations 69' and 93' around the edges of the steel plate.

4) Below grade where wood form work may be required for the grade beams supporting the new R, N' and N line walls. This form work, if needed, would be located in the vicinity of the service water piping, diesel fuel oil lines and the electrical duct bank. A minimum of 3 inches of sand will separate those items from the above form work.

Figures 3-1 through 3-4 show locations of the above described wood form work. These figures are the same as attached to the answer to NRC Question No. 7, dated July 20, 1979.

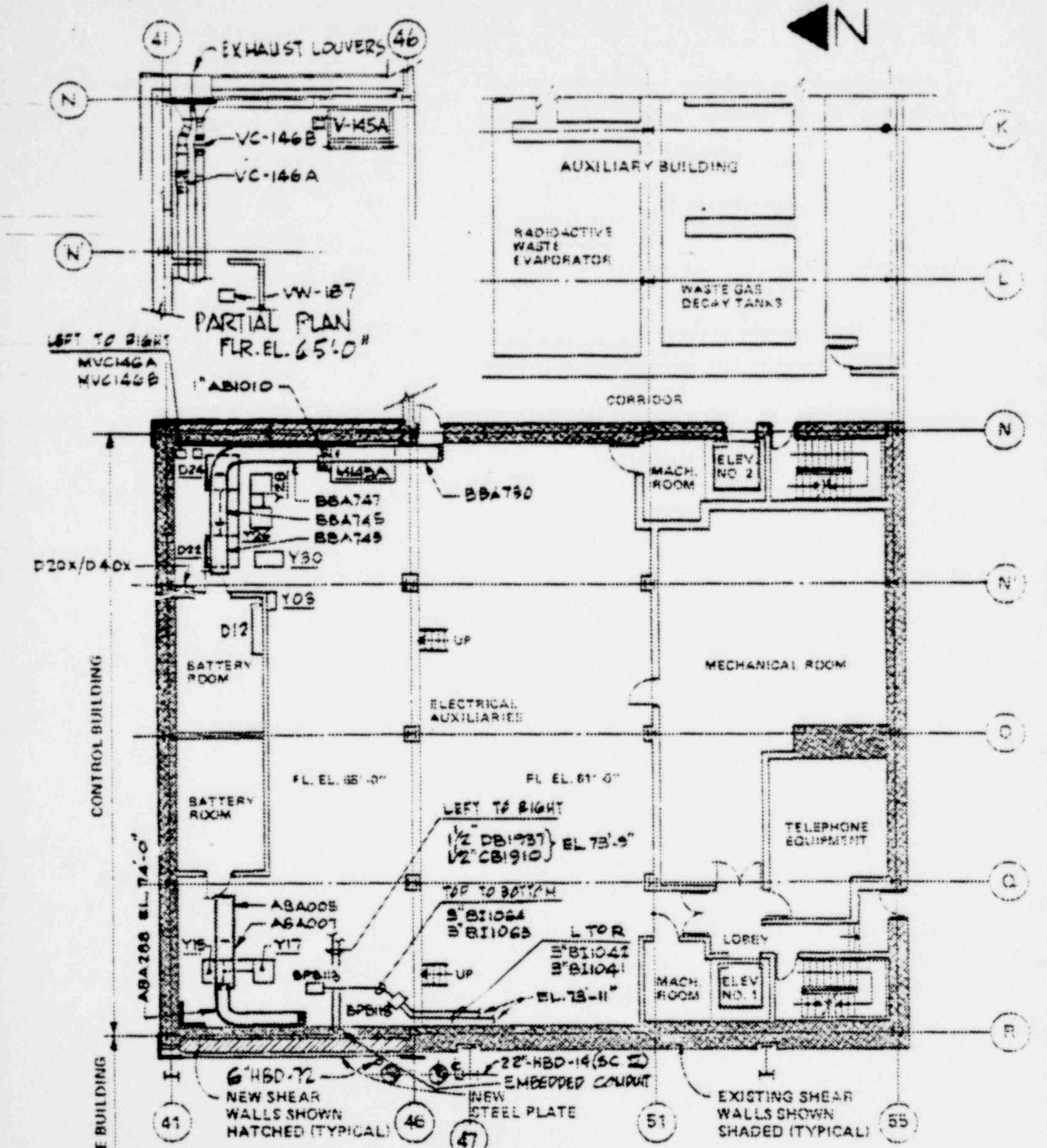
In addition, as described in response to Question 6 of this set, wood cribbing will be used as Plate 8 is being lowered into place. Figures 6-1 and 6-2 show location of the wood cribbing.



CONTROL BUILDING FLOOR PLAN EL 45'-0"
SHOWING EXISTING AND NEW SHEAR WALLS

POOR ORIGINAL

SKETCH NO. 3-1



CONTROL BUILDING FLOOR PLAN EL 61'-0" &65'-0"
SHOWING EXISTING AND NEW SHEAR WALLS

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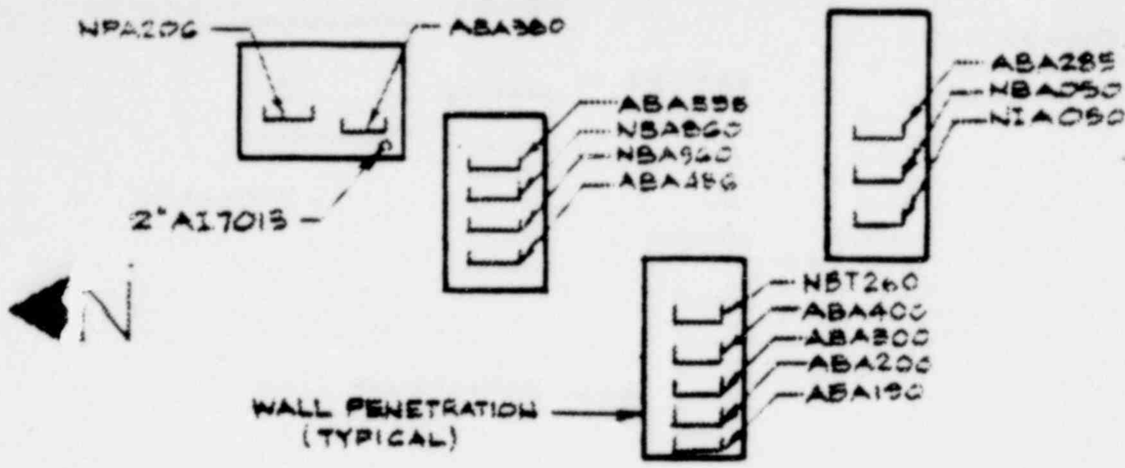
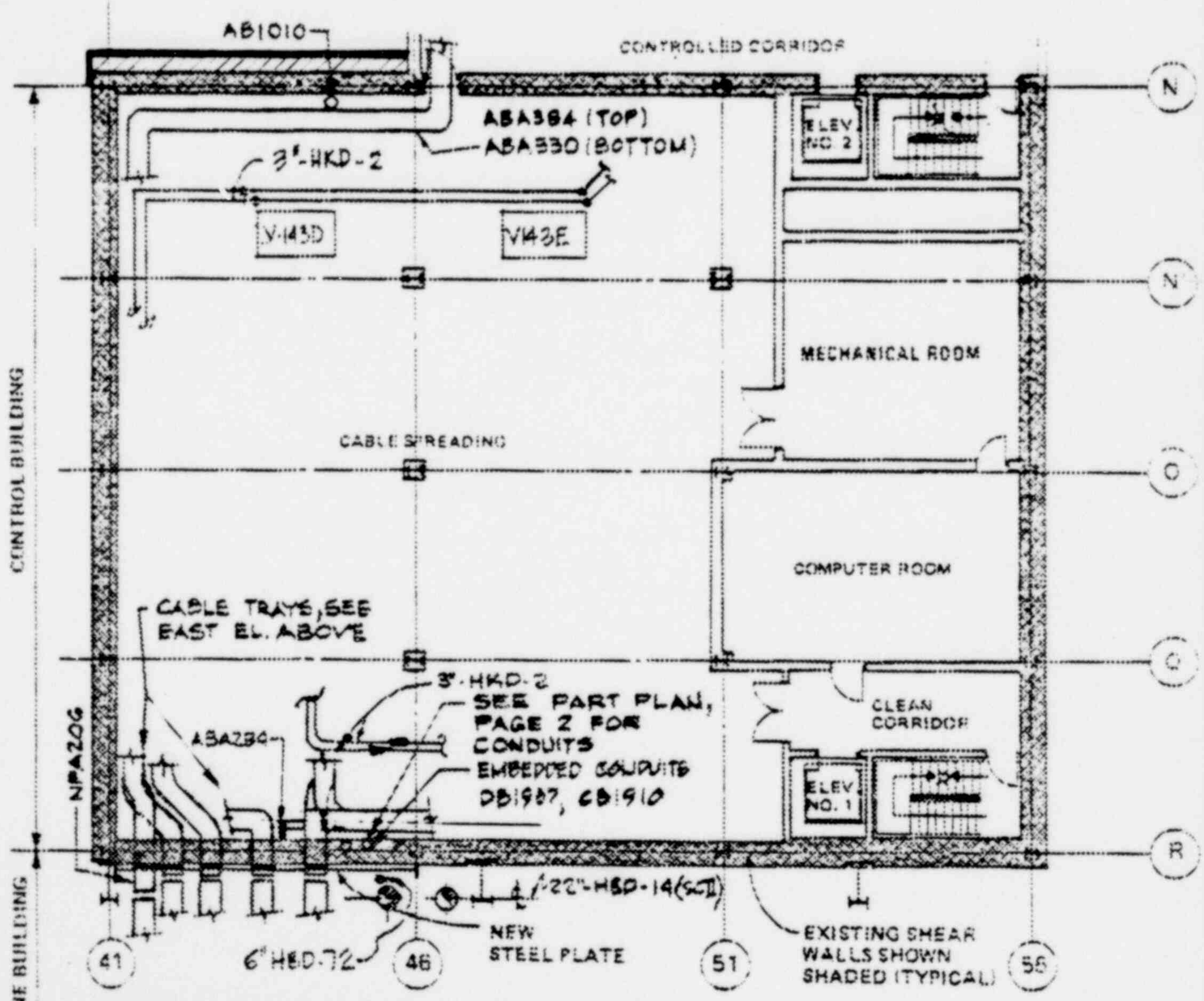
POOR ORIGINAL

SKETCH NO. 3-2



EAST ELEV. @ COL. R



CONTROL BUILDING FLOOR PLAN EL 77'-0"
SHOWING EXISTING AND NEW SHEAR WALLS

POOR ORIGINAL

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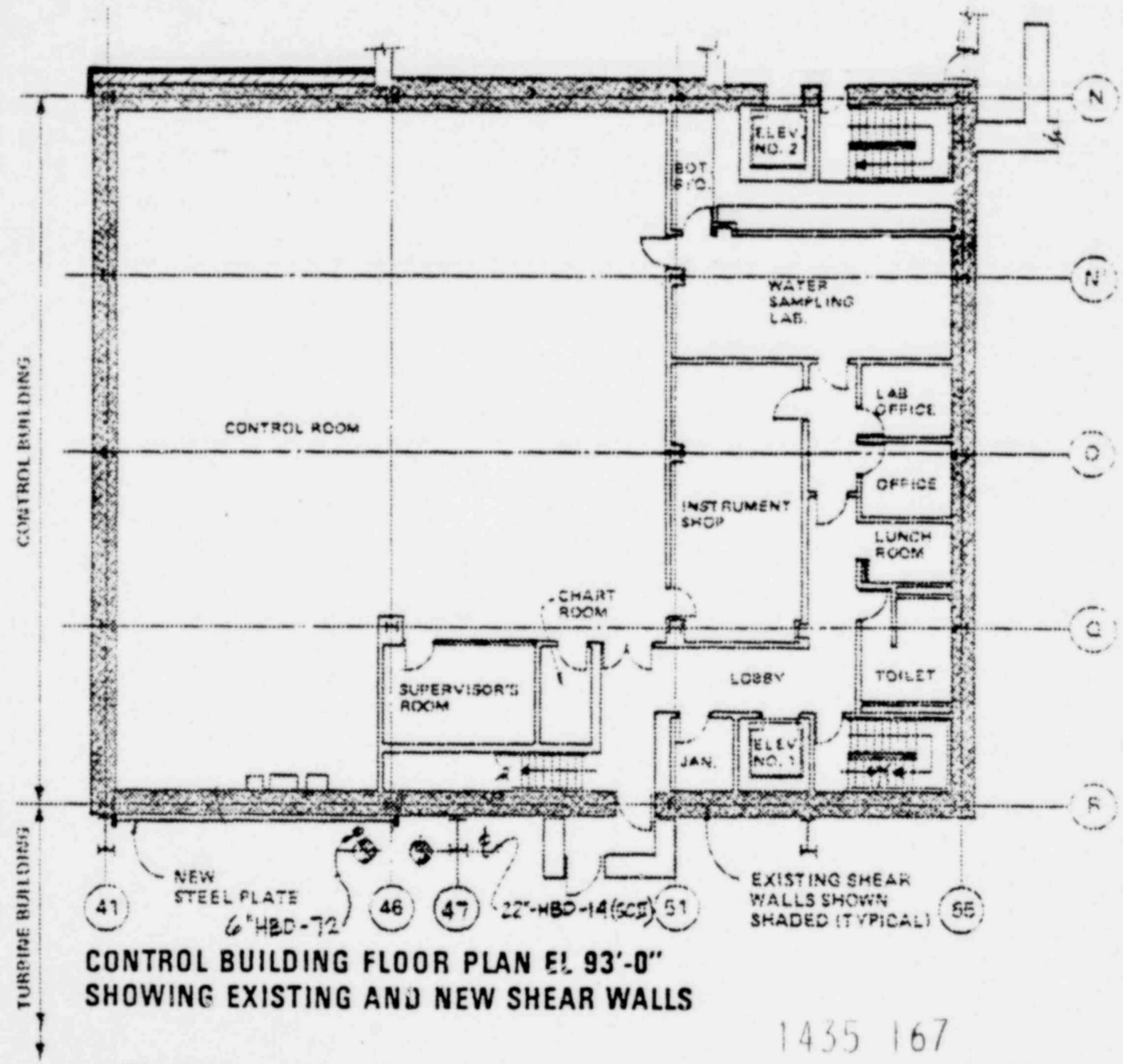
SKETCH No. 3-3



CONTROL BUILDING FLOOR PLAN EL 93'-0"
SHOWING EXISTING AND NEW SHEAR WALLS

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POOR ORIGINAL

SKETCH No 3-4

Your response regarding the use of grout for installation of rebar into the existing walls and rock does not adequately justify its acceptability in these applications. Therefore, provide the following:

a) Verification that inactive carbon, sand and cement are the only constituents of the grout and that contains no other materials.

b) Substantiation that the expansion of the grout in only the plastic stage is sufficient considering the effects of any shrinkage which may occur beyond that in the plastic stage. If there is any expansion beyond the plastic range, substantiate that it's effects are negligible with regard to splitting of the existing materials (block, concrete, etc.)

c) Test data which substantiate that the use of this grout (1) in holes of dimensions similar to those which will be used at Trojan, (2) in materials similar to those in which the rebar will be grouted (i.e., concrete grouted masonry block and rock), and (3) using the same type rebar as that to be used at Trojan that the full rebar strength will be developed in every case. In addition to the tests mentioned in the specification CRD-C588-78, the following test should be performed: 1) tensile tests on the grout in accordance with ASTM Specification C190-77, and 2) strength tests of full-scale specimens representing the proposed anchorages in accordance with the spirit of ASTM Specification E-488-76.

Answer:

(a) The attached letter (Attachment 5-1) from U. S. Grout Corporation verifies that Five Star Grout, the grout to be used for installation of rebar, consists of three components:

1) a high early strength Type 3 cement
2) a fine silica sand
3) a non-reactive chemically inert aggregate called Permanent Life Aggregate (PLA).

Permanent Life Aggregate, as specified in the attached letter, is a chemically inert form of activated carbon.*

Activated carbon is porous carbon which has affinity for water. When the activated carbon contained in the grout comes in contact with the mixing water, it absorbs water which displaces the air contained in its pores. The air thus released into the grout paste expands due to the heat of hydration. This mechanism gives the expansive characteristic to the grout during the setting process.

The percentage of the constituents as given in the response

*Licensee's response dated September 5, 1979 to NRC Structural Branch Question 7 incorrectly characterized PLA as inactive carbon.

dated September 5, 1979 to NRC Question No. 7 is by weight.

(b) Testing of the grout to ASTM C-827 has established that expansion will occur while the material is in the plastic stage. (See Attachment 5-1). Testing to CRD-C588-78 shows that Five Star Grout does not exhibit either significant expansion or shrinkage after hardening. (See Attachment 5-2).

(c) Within the Complex, rebars will be grouted only into core concrete. Connection details are being revised to obviate the need for grouting rebars into masonry.

The rebars grouted in rock for the rail stop anchorage will each be pull tested after installation to verify that they can develop the design loads.

Data on tests performed by West Penn Testing Laboratories established that under conditions very similar to those at Trojan, rebars grouted into concrete developed their full strength without failure of the grout.

The following comparison establishes that the tests referenced above sufficiently reflect the way in which rebars will be grouted at the Trojan Plant, such that the results of the tests are directly applicable:

1. Hole dimensions: 2.75 in. at test, 2.5 to 3 in. at Trojan.
2. Materials in which rebar will be grouted: 5000 psi design strength concrete in both cases.
3. Similar types of rebar: 60 ksi deformed bars #6 and #7 tested; 60 ksi deformed bars #5, #7, and #9 at Trojan.
4. Same type of grout material: Five Star in both cases.

The major difference between the tests and the Trojan condition will be the embedment length. Trojan will use embedment lengths as required by the Code. Tests were made with only 10 in. embedment length which is shorter than that required by the ACI Code.

Test data which substantiates compliance with CRD-C588-78 is attached (Attachment 5-2). Tests performed in accordance with ASTM C190-77 indicated that the tensile strength of the Five Star Grout is 722 psi (Attachment 5-1).

U.S. GROUT CORPORATION
ENGINEERING AND TECHNICAL CENTER
1154-56 EAST PUTNAM AVENUE ■ RIVERSIDE, CONNECTICUT 06878 ■ (203) 637-4305

September 19, 1979

Messrs. Ted Bushnell & Don Broehl:
PORTLAND GENERAL ELECTRIC
121 S. W. Salmon Street
Portland, OR 97204

Dear Messrs. Bushnell & Broehl:

This is to certify that Five Star Grout consists of three components: A high-early strength type 3 cement, a fine silica sand, and a non-reactive chemically inert aggregate called PLA (Permanent Life Aggregate). PLA is a chemically inert form of activated carbon.

Expansion will occur while the material is in the plastic state when tested by ASTM C-827 and will exhibit no shrinkage or expansion after hardening. Five Star Grout conforms to the specified criteria in CRD-C-588 and may exhibit a minute amount of expansion by this test. Five Star Grout has a tensile strength of 722 psi when tested by ASTM C-190-77.

All additional data on pull-out test and volume change are being forwarded under separate cover.

Very truly yours,

John Reilly
Asst. Mgr. Industrial Division

JR:jg

Enclosure:

cc: Mr. Everett L. Thompson
14806 Bothell Way N. E., Apt. 326
Seattle, WA 98155
(206) 363-8829

ATTACHMENT S-1

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CPR

CONSTRUCTION PRODUCTS RESEARCH, INC.
The Babcock Building, Old Greenwich, Connecticut 06870 · Phone (203) 637-2002 · Cable CPR

CERTIFICATION

Date: May 11, 1978

Product: Five Star Grout

Water Added for Test: 23% by weight

Lot Number: C780322 04

Volume Change, ASTM C-827 Max, % +1.9%

	3 Day	7 Day	14 Day	28 Day
Expansion, CRD-C-588-76	+.03%	+.03%	+.03%	+.03%

Compressive Strength ASTM C-109

1 Day	9200 9300 9500	2330 psi
7 Day	21600 20600 21200	5280 psi

Time of Set ASTM C-191

Final 3 hours 20 minutes

This is to certify that the above tests were performed on a sample of material taken from the above lot and that the above results were obtained.

D. F. Fiala
Vice President

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ATTACHMENT 5-2

West Penn Testing Laboratories, Inc.
An Independent Inspection Bureau and Testing Laboratory
482 West Eighth Avenue West Homestead, Pennsylvania 15120
P.O. Box 324 - Area Code 412 462-3717

File No. WP-2002
March 14, 1978
Page 1

Attachment 5-3
Page 1

Report of

REINFORCING BAR SHEAR BOND TESTING

PROJECT: [redacted]
OWNER: Pennsylvania Power & Light
CONTRACTOR: Research-Cottrell
DATE OF INSPECTION: March 10, [redacted]

Scope

To determine if the shear bond strength of grout used to anchor reinforcing bars could withstand loading as great or greater than the tensile strength of the steel.

Description of Reinforcing Anchoring

The reinforcing to be tested were grade [redacted]s. The bars were anchored into pre-drilled holes of varying diameters. Two different products were used to achieve the bond. One agent was [redacted] produced by U. S. Grout Corporation. The other agent was Sika Hi-Mod produced by Sika Chemical.

Test Set-up

All single bars were tested using a calibrated 20 ton Holl-o-ram Centerhole Jack (RCH 202 014) connected to a hydraulic pump through a Duragauge 10,000 lb. Test Gauge used to measure line pressure. Two 8 inch channels with their webs back to back one inch apart were welded together to form a yoke. The yoke was placed over the bar, bearing on steel shims set at a distance of 10 inches on either side of the bar. The test jack was placed over the bar and set on the yoke. A cadweld was placed on the bar over the jack to provide a means of applying the load to the bar.

The double bar set up was tested using a calibrated 50 ton Enerpac Jack (RC 506 AH5) connected to a hydraulic pump through the Duragauge Test Gauge used to measure line pressure. A reinforced W8 x 24 beam was centered perpendicular to the centerline of the bars. The beam had bearing on steel shims placed 10 inches from the centerline. The test jack was centered on the beam. The yoke previously described was placed over the bars and centered on the jack. Cadwelds were again placed on each bar to facilitate load transfer.

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West Penn Testing Laboratories, Inc.

An Independent Inspection Bureau and Testing Laboratory

482 West Eighth Avenue West Homestead, Pennsylvania 15120

P.O. Box 324 • Area Code 412 462-3717

File No. WP-2002
March 14, 1978
Page 2

REINFORCING BAR SHEAR BOND TESTING
Susquehanna Steam Electric Station
Pennsylvania Power & Light
Research-Cottrell
March 10, 1978

Attachment 5-3
page 2

Test Procedure

In all tests a surcharge of 1,000 lbs. was applied to the completed test apparatus for the purpose of seating all components. The load was released and all bearing distances were rechecked. The test load was applied at a constant rate until a load of 125% of the bar design was obtained, or until failure. In applicable cases the maximum load was held for 5 minutes then gradually released to zero load.

TEST RESULTS:

[redacted]

Test No.	Bar Size	Hole Size	Comment
1	[redacted]	[redacted]	No failure at full load of 45,060 lbs.
2	#7	2.75"x10"	No failure at full load
3	#7	2.75"x10"	No failure at full load
4	#6	2.75"x10"	Double bar set up No failure at full load of 61,120 lbs. Load increased to 69,000 lbs. causing cracking in concrete
5	#6	2.75"x10"	No failure at full load of 30.560
6	#6	2.75"x10"	No failure at full load
7	#6	2.75"x10"	No failure at full load

Provide the results of your analyses showing that plates 1 through 6 are sufficient to sustain without detrimental effects on plates 1-6, the structure, equipment, piping, or cable trays, the impact of plate 8 should a drop of plate 8 occur. Include (a) a detailed description of all assumptions used in the analyses, and (b) detailed justification for all of the assumptions used in the analyses, all of the loads and all of the acceptance criteria relied upon. Include an identical discussion for plate 7.

Answer:

To preclude any possibility of detrimental effects on Plates 1-7, the structure, equipment, piping or cable trays should a drop of Plate 8 occur, the maximum drop height of Plate 8 will be limited to 4 inches by placing timber cribbing on top of Plates 5, 6, and 7 as shown on the attached Figures 6-1 and 6-2. The timber cribbing will consist of two piles of 4" x 4" x 4' long pieces stacked on top of each other. As the plate is being lowered, 4" thick segments will be removed one at a time from each pile, thus limiting the drop height of Plate 8 on wood to approximately 4". The last piece removed from each pile will be 1" thick, thus further reducing the drop height of Plate 8 on the plates below to 1".

The timber cribbing will be made using Douglas Fir or similar wood. It will be supported on the bottom by brackets attached to the lower plates. The cribbing will be braced laterally by guide plates designed to prevent bulging and subsequent collapse of the cribbing. The guide plates will be supported by the Turbine Building floor at el. 93', the girder, and the lower plates. Temporary lateral bracing will be added to the girder to resist the lateral forces induced by the cribbing and guide plates should Plate 8 drop.

The maximum vertical force induced by a drop of Plate 8 on the timber would be limited by the crushing strength of the timber normal to the grain. Therefore, the force on the lower plates would equal to

$$F = P_{cr}A(D.I.F.)$$

where

P_{cr} = crushing strength of timber, taken as 800 psi

A = contact area

D.I.F. = Dynamic Increase Factor, taken as 2.0

$$F = \frac{800 \text{ lbs}}{\text{in}^2} \times 2 \times 48 \text{ in.} \times 3 \text{ in.} \times 2.0 = 460.8^K$$

This force would be resisted by the 84 bolts holding the lower plates in place. Twenty-one (21) of the bolts are bearing on block walls and sixty-three (63) are bearing on concrete. The allowable shear on bolts in masonry and concrete was established based on Tables No. 24-G and 26-G of the 1976 UBC and extrapolating to 1-3/4" diameter. The following allowable shear loads per bolt were used:

Concrete: 7.7^K/bolt (with special inspection)

Masonry: 3.8^K/bolt

Therefore, the total capacity of all the bolts equals:

21 bolts x 3.8^k/bolt = 79.8^K

63 bolts x 7.7^K/bolt = $\underline{485.1^K}$

Total capacity = 564.9^K

Since the total capacity exceeds the applied load, the bolts will hold the lower plates in place.

Steps will be also taken to preclude any possibility of detrimental effects on Plates 1-4, the structure, equipment, piping or cable trays should a drop of Plate 7 occur. A corrugated aluminum HEXCEL pad, stabilized and precrushed, will be placed on Plate 4 to absorb the energy of the drop. The HEXCEL pad will be 4" wide, 24" long, and 17" thick. It will be attached to the top of Plate 4 as shown in Figures 6-3 and 6-4. A "shoe" under Plate 7 will spread the load. The Z bars shown in Figure 4-1 in Licensee's response dated September 5, 1979 to Systems Branch Question No. 9 will guide the plate.

The analysis to show the adequacy of this system is as follows:

Weight of Plate 7, W = 3 kips

Maximum drop height, H = 14.75 ft.

Maximum kinetic energy, KE = 3 x 14.75 = 44.25 ft-kips

or KE = 44.25 x 1000 x 12 = 531,000 in-lbs

The corrugated aluminum HEXCEL pad will have a 750 psi crush strength. For added conservatism, it is assumed that half of the honeycomb core thickness is available for crushing (the manufacturer suggests that up to 7/10 of the thickness is available for crushing). The energy absorbed equals the kinetic energy:

t_c = honeycomb core thickness
S = depth of crushed core
A = Area of core

$KE = f_{cr} \times A \times S$
$f_{cr} = 750$ psi
$A = 24 \times 3\text{-}1/2 = 84 \text{ in}^2$
$S = .5\ t_c$

$$531{,}000 = 750 \times 96 \times .5\ t_c$$

$$t_c = \frac{531{,}000}{750 \times 84 \times .5} = 16.9 \text{ in.}$$

17 in. thickness will be used.

The vertical force induced in the lower plates would be

$$F = 1.3 \times f_{cr} \times A$$

where 1.3 is a dynamic factor suggested by the manufacturer.

$$F = 1.3 \times 750 \times 84 = 81.9^K$$

This force would be resisted by the 71 bolts holding the lower plates in place.

The total capacity of the bolts equals:

$$8 \text{ bolts} \times 3.8^K/\text{bolt} = 30.4^K$$

$$63 \text{ bolts} \times 7.7^K/\text{bolt} = \underline{485.1^K} \text{ (concrete)}$$

$$\text{Total capacity} = 515.5^K$$

Since the total capacity exceeds the applied load, the bolts will hold the lower plates in place.

Reference 1: "Wood Handbook" No. 72, by the U.S. Dept. of Agriculture, 1955, Table 12, page 75.

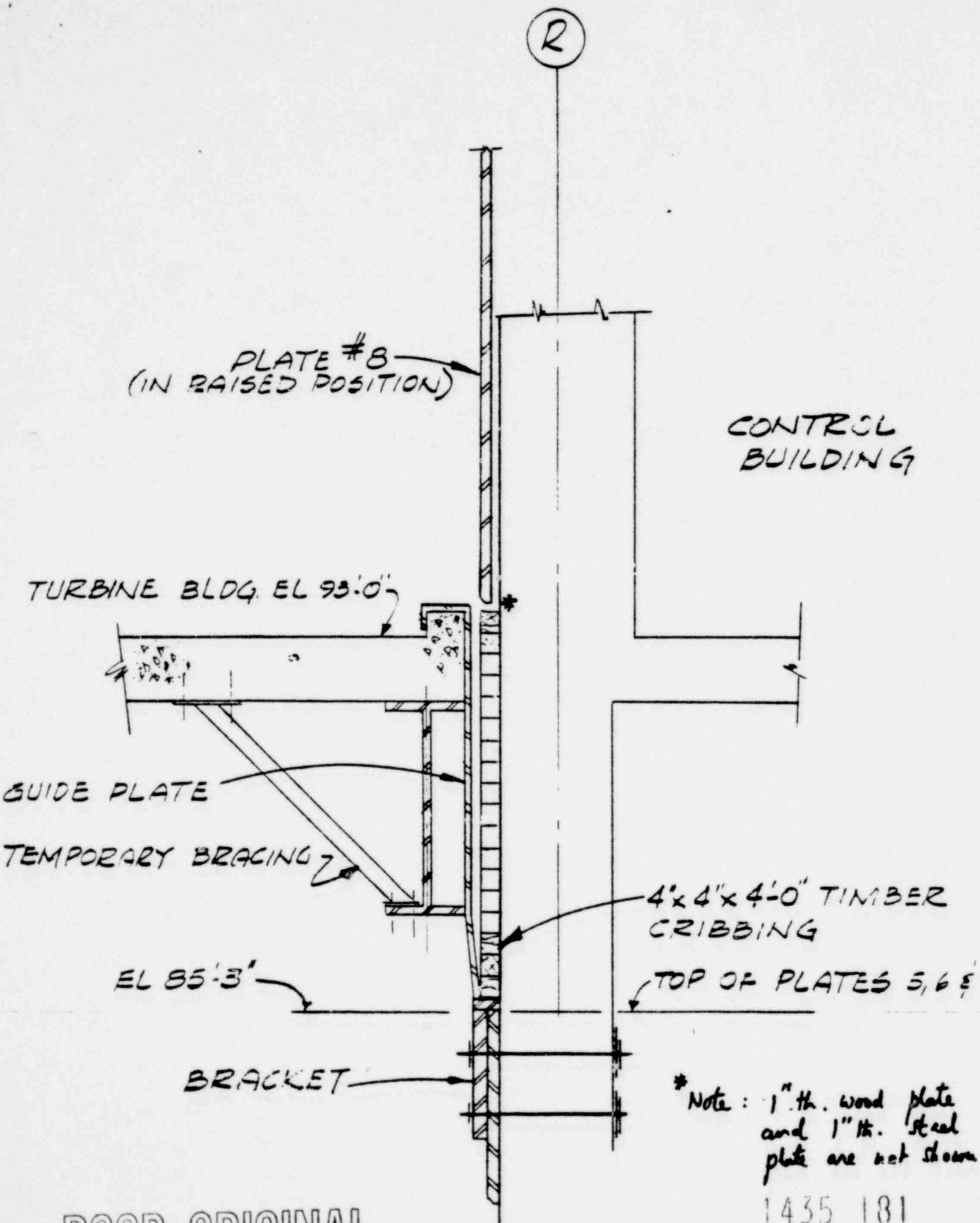

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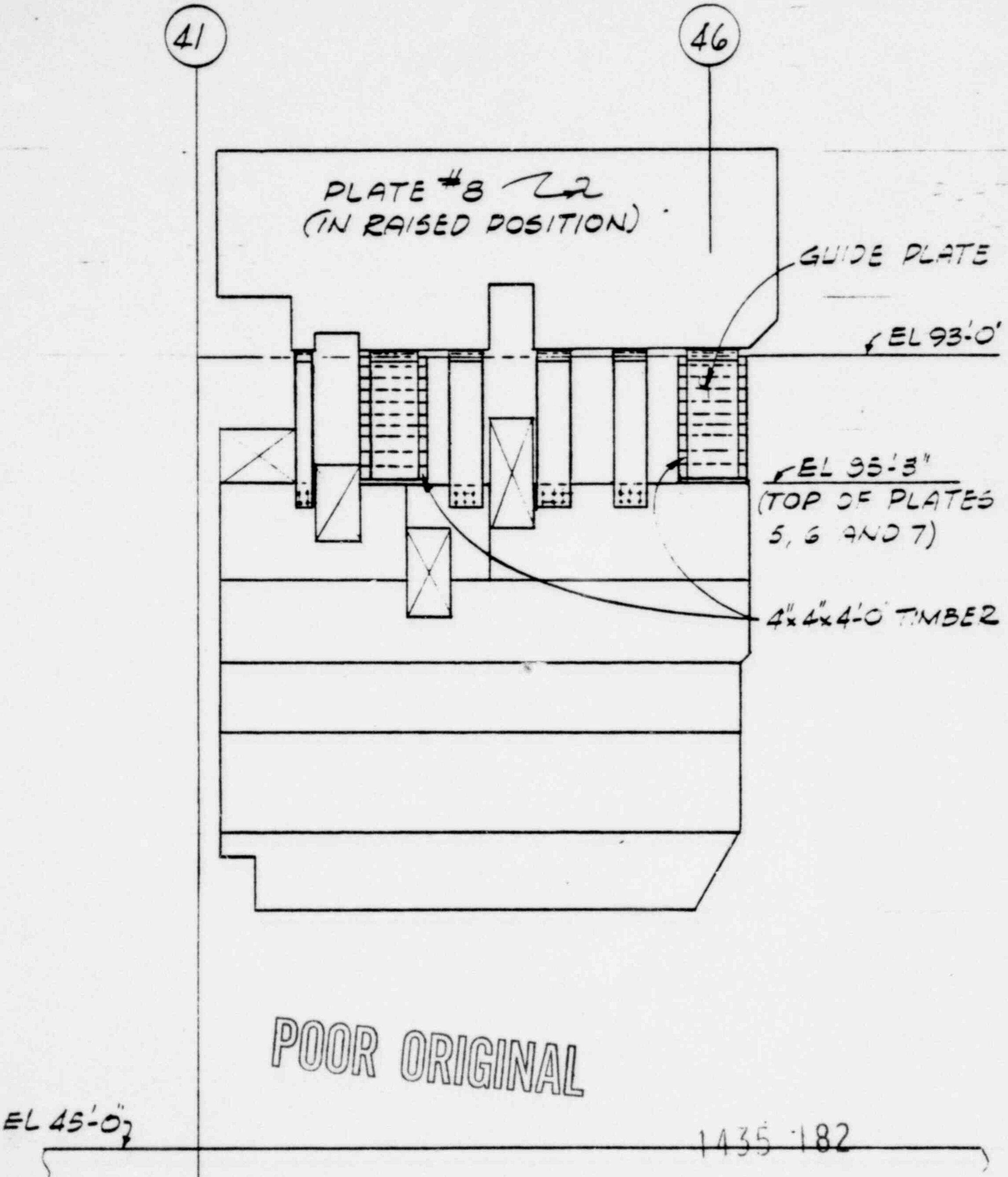
FIGURE 6-1



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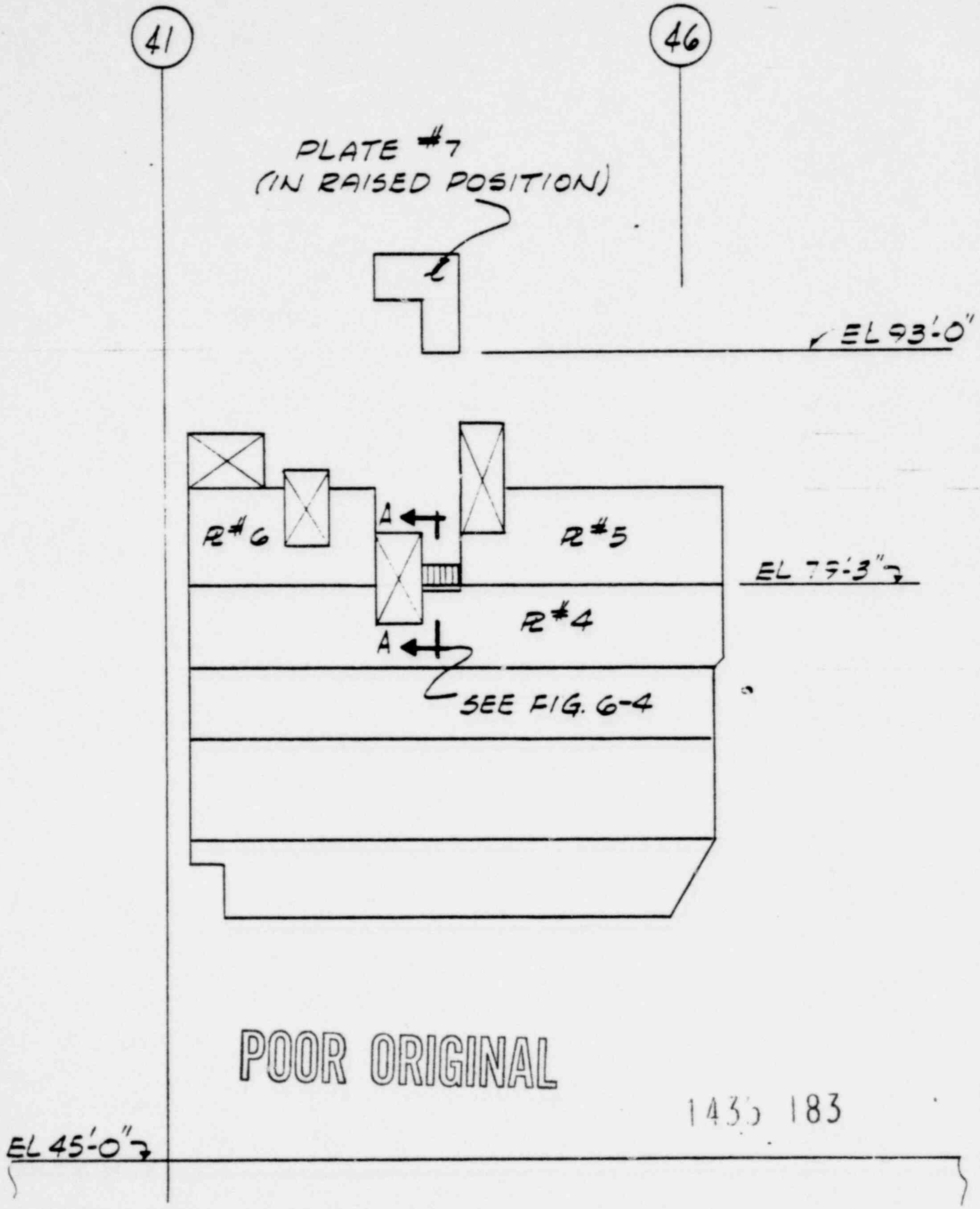
ELEVATION OF R WALL LOOKING EAST

FIGURE 6-2



POOR ORIGINAL

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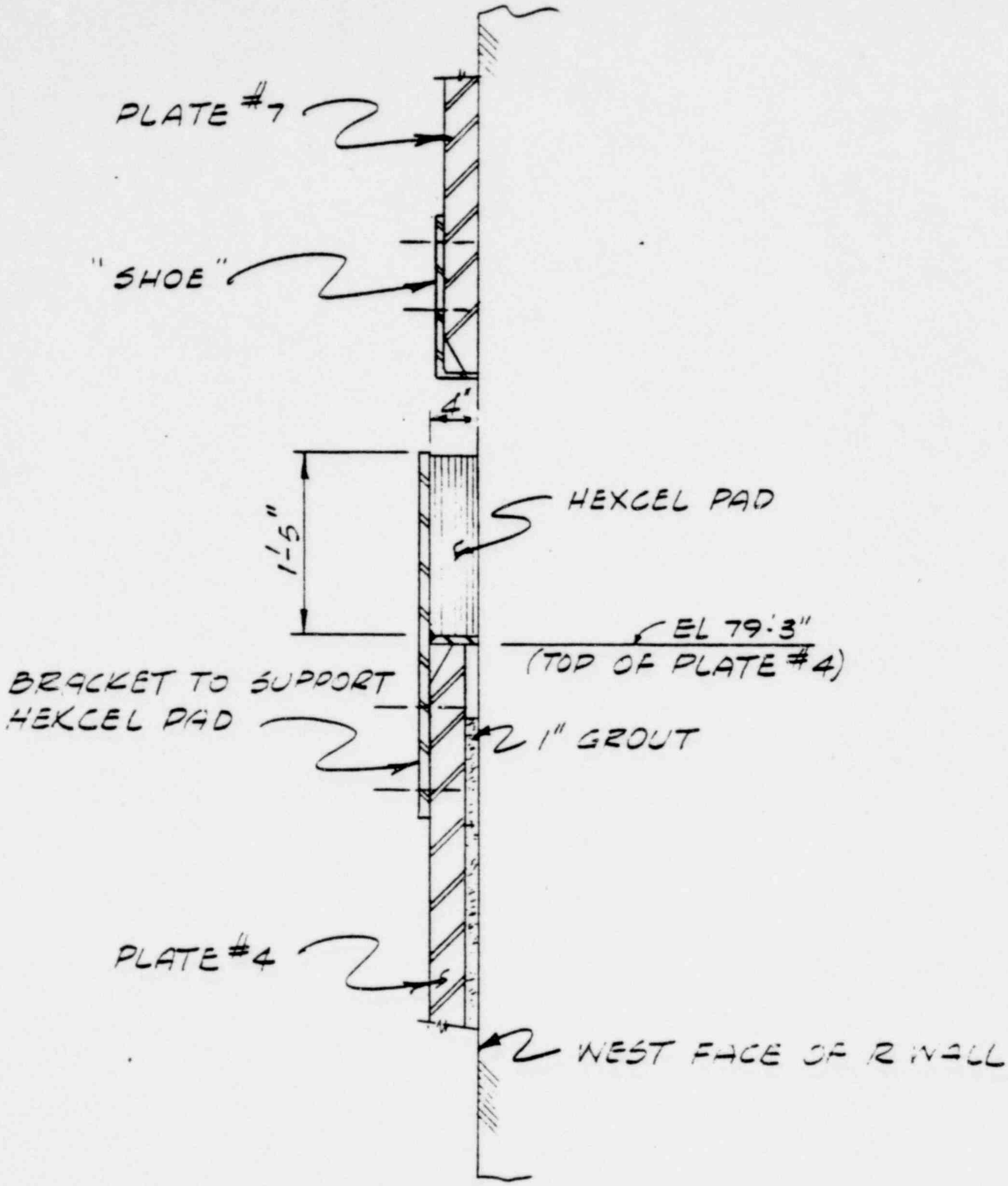
ELEVATION OF R WALL LOOKING EAST

FIGURE 6-3



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SECTION A-A

FOR LOCATION SEE FIG. 6-3

POOR ORIGINAL

FIGURE 6-4

Propose an inservice inspection program for the bolts to be used to provide for shear transfer between the new and existing structural elements. Provide and justify the bases on which it can be concluded that the proposed inspection program will provide assurance that the relied-upon bolt tensions will be maintained in <u>all</u> bolts throughout the life of the plant.

Answer:

An inservice inspection program for bolt tension will be conducted on new bolts included in the Control Building modification for which bolt tension is relied upon to develop the frictional force for shear transfer between new and existing structural elements.

Although potential pretension losses in the bolts have been conservatively considered in the design (design based on an assumed loss of 25% of final construction pretension), the following inservice inspection program to verify bolt tension with time will be implemented:

Control Building Modification Connection Bolts

The structural adequacy of the bolts used to reinforce the Control Building shall be demonstrated at the end of one, three and five years after initial tensioning and at five year intervals thereafter. Structural adequacy shall be demonstrated by:

a. Demonstrating that each bolt in a random and representative sample of not less than 25% of the total number of bolts has a tension of equal to or greater than 80% of the initial bolt tension. If the tension in any bolt is below 80% of the initial bolt tension, the tension in two adjacent bolts shall be measured. If either of these bolts is found to have less than 80% of the initial bolt tension, then all bolts shall be tested. All bolts found to have less than 80% of the initial bolt tension shall be retensioned to the original installation tension value.

b. Demonstrating the acceptability of the entire test sample by showing that $\bar{x} - 2\sigma \geq 0.8\ \bar{x}_o$, where $\bar{x}$ is the mean sample tension, σ is the standard deviation and $\bar{x}_o$ is the mean initial bolt tension. If this criterion is not met, then all bolts shall be tested to the criteria in (a) above.

c. Determining that there is no evidence of degradation or abnormal conditions by visual inspection of the condition of all bolts in the sample, their end anchorages and concrete or masonry in the vicinity of the anchorage.

d. If the bolts inspected during the first three inspections meet the acceptance criteria of (a), (b) and (c), then the sample for the subsequent inspections may be reduced to not less than 10% of the total number of bolts.

This proposed inservice inspection program will provide an appropriate evaluation of 1) the tension in the bolts at the time of the test, 2) the relationship of possible bolt pretension losses with time, and 3) the conditions of the concrete or masonry at the bolt anchorages.

A random and representative sampling of 25% of all bolts will provide a suitable sample size from which a meaningful standard deviation can be determined, particularly since all bolts are of identical configuration (straight through-wall, loaded in direct tension only with constant design preload values, all of the same material and diameter, and all of similar length). Also, the service environment for the bolts is essentially the same throughout.

The acceptance criterion for an individual bolt test tension of equal to or greater than 80% of the initial pretension value furnishes a margin against the 75% of initial pretension value that was used, in addition to the factor of safety of 2 provided in the bolt tension-shear transfer relationship, as a basis for the original design pretension. The acceptance criterion for the entire sample requires that the sample mean minus twice the sample standard deviation ($\overline{X} - 2\sigma$) be equal to or greater than 80% of the mean value of the initial bolt pretension ($\overline{X}_o$). This provides reasonable assurance that, as a minimum, 97.5% of all the bolts will have pretension values not less than 80% of the initial pretension value, still with a factor of safety of at least 2.

The condition of exposed portions of the test sample bolts, end anchorages, and concrete or masonry surfaces adjacent to the end anchorages will be visually inspected during each test (the portion of the bolt within the wall is subjected to essentially the same environment as conventional reinforcing steel, and corrosion is not a concern).

The time dependent behavior of the bolts is expected to be an exponential function of time where most losses that will occur should occur relatively soon after the initial installation. Therefore, with the condition that the first three tests demonstrate that bolt pretension losses are essentially stabilized, reduction in the size of the test sample is justified.

We believe that the proposed inservice inspection program will provide assurance that the bolt tension, in all bolts, which is relied upon to develop the frictional force for shear transfer between new and existing structural elements will be maintained throughout the life of the Plant.

Verify that the computer program WECAN was used only for linear elastic analyses. Additionally, verify that the computer program verifications for the CYLNOZ, SPHNOZ and DESREV meet the requirements of Standard Review Plan Section 3.9.1.II.

Answer:

In the reevaluation of equipment with response spectra based on the modified Complex, the computer program WECAN was used only for linear elastic analysis. The equipment so analyzed was auxiliary mechnical equipment such as tanks, heat exchangers, and demineralizers.

The computer programs CYLNOZ and SPHNOZ were used only to calculate local stresses caused by external loadings in cylindrical and spherical shell elements of auxiliary mechanical equipment. CYLNOZ and SPHNOZ were developed by the Franklin Institute, Philadelphia, Pa. and are based on the curves presented in Welding Research Council Bulletin 107. The CYLNOZ and SPHNOZ programs have been verified by Westinghouse. Verification was accomplished by comparing the stresses calculated by the programs to stresses determined directly from the curves presented in Bulletin 107. Good correlation was obtained between the numbers calculated by the programs and those obtained from the curves. This method of computer program verification is consistent with the acceptance criteria for verification in Standard Review Plan Section 3.9.1.II.2.c.

The DESREV computer program, which was used only in the reevaluation of the CVCS holdup tank recirculation pump, performs static analyses of Gould's end-suction, foot-mounted pump assemblies (which consist of pump, motor, coupling and baseplate). In addition to nozzle and seismic loads, loads created by the pump operation are considered in the analysis of the functional capability and structural integrity of the pump, bedplate, shaft and hold-down bolts. These loads are also considered in the analysis of the pressure retaining portions of the pump.

The DESREV program solutions to a series of test problems are substantially identical to hand calculations, and program verification has been performed in accordance with the criteria of Standard Review Plan Section 3.9.1.II.2.c.

Verify that the Nelson studs are being placed in accordance with all criteria required by "Embedment Properties of Headed Studs" by the Nelson Division of TRW. Additionally, substantiate the conservatism of the shear/tension interaction relationship assumed for the reinforcement and the studs in your September 5, 1979 response to question 3.

Answer:

The placement of Nelson studs will be in accordance with all criteria specified in "Embedment Properties of Headed Studs" by the Nelson Division of TRW.

The spacing of the studs to develop their full tension and full shear capacities is influenced by the stud embedment, the distance between the anchors in a group, and the distance from an anchor to a free edge. Table 6 of the referenced publication provides the minimum spacing of studs for full tension capacity development. Table 4 provides tension capacity corresponding to the embedment. Tables 16 and 23 provide the minimum distances for full shear capacity development.

Although the studs in the Complex modification are designed for pure shear only, the placement and spacing of the studs will comply with the requirements for the development of full shear and full tension according to the above tables.

As shown below the shear/tension interaction assumed for the reinforcement and the studs in the Licensee's response dated September 5, 1979 to Structural Branch Question No. 3 is conservative.

As a representative example, a #7 reinforcing bar and a 5/8 diameter x 8 3/16 stud will be considered.

Considering a load factor of 1.4 for the reinforcing bar and a factor of safety of 2 for the stud (Licensee's response dated June 22, 1979 to NRC Question No. 7), the maximum allowable force on each element will be:

Tension on the #7 bar:

$$T = \frac{\phi\ f_y\ A_s}{1.4} = \frac{.9 \times 60 \times 0.6}{1.4} = 23.1k$$

Shear on the 5/8 stud:

$$V = \frac{S_{uc}}{2} = \phi[1.106\ A_s\ f'^{\,0.3}_c E_c^{\,0.44}]\frac{1}{2} =$$

$$0.85[1.106 \times 0.307 \times (3.5)^{0.3} \times (3410)^{0.44}]\frac{1}{2} = \frac{15}{2} = 7.5k$$

where

ϕ = capacity reduction factor

f_y = yield strength of reinforcing steel

A_s = area of reinforcing steel or stud material

S_{uc} = concrete shear capacity of stud

f'_c = compressive strength of concrete

E_c = modulus of elasticity of concrete

In terms of ultimate strength:

Ultimate tension force on #7 bar:

$T_u = 1.4 \times 23.1 = 32.4k$

Ultimate shear force on 5/8 stud:

$V_u = 1.4 \times 7.5 = 10.5k$

Assuming that the distribution of these forces between the reinforcement and the stud is proportional to their cross-sectional areas, the forces on each element are:

Area of stud = 0.307 in^2

Area of bar = 0.6 in^2

P = tension force

V = shear force

$$P\ stud = \frac{0.307}{0.307 + 0.60} 32.4 = 11k$$

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$$P\ bar = 32.4 - 11 = 21.4k$$

$$V\ stud = \frac{0.307}{0.307 + 0.60} 10.5 = 3.55k$$

$$V\ bar = 10.5 - 3.55 = 6.95k$$

The interaction of tension and shear in the reinforcing bar is considered in the following manner:

$$\text{Minimum } A_s = \frac{P\ bar}{\phi\ f_y} + \frac{V\ bar}{\phi\ f_y\ \mu}$$

where

ϕ = capacity reduction factor

f_y = yield strength of reinforcing steel

μ = coefficient of friction

$$\text{Minimum } A_s = \frac{21.4}{.9 \times 60} + \frac{6.95}{.85 \times 60 \times 1.4} = .40 \text{ in}^2 < 0.60 \text{ in}^2$$

Since the area of reinforcement provided (.6 in.^{2}) is more than the minimum area required (.40 in.^{2}), the capacity of the reinforcement will not be exceeded.

The interaction of tension and shear in the stud is considered as follows: (see Section 6 of the referenced TRW publication)

$$\left(\frac{P\ stud}{P_u'}\right)^{5/3} + \left(\frac{S\ stud}{S_u'}\right)^{5/3} < 1$$

P'_u = ultimate tension capacity of stud (from Table 4) = 16.56k
S'_u = ultimate shear capacity of stud = S_{uc} = 15k

$$\left(\frac{11}{16.56}\right)^{5/3} + \left(\frac{3.55}{15}\right)^{5/3} = .6 < 1$$

Therefore, the capacity of the stud under combined tension and shear will not be exceeded.

In your July 10, 1979 response to question 13, an unrestrained strain of 100 x 10 exp -6 in/in (and a restrained strain of 70 x 10 exp -6 in/in) is assumed for the in-situ walls. In your September 5, 1979 response to question 11, an unrestrained shrinkage strain of 280 x 10 exp -6 in/in is assumed for the new walls. In your September 5, 1979 response to question 22, shrinkage strains are calculated to be 174 x 10 exp -6 in/in for the new walls and assumed to be 200 x 10 exp -6 in/in for the existing walls, the latter being based upon the assumption that new concrete placed against the existing wall causes the existing to swell (as would be the case for the block when the core concrete was placed). These values are extremely inconsistent. Justify this inconsistency in detail, and provide calculations indicating how each was established (in addition to those already provided) along with justifications for all assumptions (including those for calculations already provided), including details of the associated concrete mixes.

Answer:

The differences in the values for shrinkage strain cited in answer to the various questions arise primarily because the values were determined in response to questions relating to differing circumstances, which called for differing approaches with differing degrees of conservatism. For example, NRC Question 13, dated July 10, 1979 addressed the issue of the effect of creep and shrinkage on the dead load distribution on the existing Complex walls. NRC Question 11, dated

September 5, 1979 related to the reduction of allowable shear stress, V_c, in the new reinforced concrete wall as a result of tension developing at the interface between the new and existing walls owing to shrinkage in the new walls. Question 22, dated September 5, 1979, on the other hand, dealt with the evaluation of bolt losses because of shrinkage in the new concrete walls and also possible shrinkage in the existing walls due to the evaporation of the absorbed moisture in the existing walls.

1. Existing Walls

The Licensee's response dated July 10, 1979, to NRC Question No. 13, described the effects of creep and shrinkage phenomenon in the existing walls of the Complex on the distribution of wall dead load to the embedded structural steel columns. In that response an unrestrained shrinkage strain of approximately 100×10^{-6} in/in was taken for the composite walls based on published shrinkage test results as referenced therein. Also, a restrained shrinkage strain of 70×10^{-6} in./in. was assumed for the walls.

A detailed evaluation of the shrinkage strain, specific to the walls of the Complex, is given below for a typical 30-inch thick wall. The analysis is based on the outline as given in ACI paper No. SP 27-13 (Reference 2-1) which is the basis of the recommendation as reported in ACI paper No. SP 27-3 by the ACI Committee 209 (Reference 2-2). The correction factors to the ultimate shrinkage strain are based on the values of the associated parameters of the concrete mix given in Table 2-1.

The unrestrained shrinkage strain at any time t is given by

$$\varepsilon_{sh} = \varepsilon_{shu} S_t S_h S_{th} S_s S_f S_e S_c$$

where ε_{shu} is the ultimate shrinkage strain as obtained from tests on laboratory sample.

Ultimate shrinkage strain (ε_{shu})

Tests carried out on the laboratory samples for concrete mixes used in the construction of the Complex walls gave the following unrestrained shrinkage strain:

$$42 \text{ days shrinkage} = 540 \times 10^{-6} \text{ in/in}$$

The time of shrinkage coefficient, S_t, gives the fraction of strain in time t days of the ultimate shrinkage strain.

From Ref. 2-2,

$$S_t = \frac{t}{35 + t} \quad \text{for moist-cured concrete}$$

$$S_{42} = \frac{42}{35 + 42} = 0.545$$

$$\varepsilon_{shu} = \frac{540 \times 10^{-6}}{0.545} = 990 \times 10^{-6} \text{ in/in}$$

Time of shrinkage coefficient, S_t

This factor is defined above. The total dead load at a particular elevation of a wall was built up in an incremental fashion as the portions of the wall above were constructed. Since the time that elapsed in erecting a wall from ground elevation up to the roof of the Control Building was about four to six months, consideration was made of the shrinkage of a portion of a wall prior to its being loaded by the wall weight above it. This time lag effect was conservatively taken as 21 days because the time period that elapsed between erection of a wall below and the dead load coming from the wall above is more than 21 days.

Therefore,

$$S_t = S_{40 \text{ years}} - S_{21 \text{ days}}$$
$$= 1.0 - \frac{21}{35 + 21}$$
$$= 0.62$$

Relative humidity coefficient, S_h

The average annual humidity furnished by the Portland, Oregon Weather Bureau is 73%. However, in consideration of the fact that both the faces of the walls are not exposed to outside atmosphere, an average humidity of 60% was assumed.

S_h = 1.40 - 0.010H, where H = 60
= 0.80

Minimum thickness of member coefficient, S_{th}

S_{th} = 1.17 - 0.029T, where T = 30 inches (flow path for moisture evaporation consistent with composite wall thickness)
= 0.30

Slump of concrete, S_s

S_s = 0.89 + 0.041S, where S = 3 1/2 inches slump
= 1.03

Fines coefficient, S_f

S_f = 0.30 + 0.0140F, where F = 40 (percentage of fine aggregate by weight)
= 0.86

Air Content coefficient, S_e

S_e = 0.95 + 0.0080A, where A = 3.8 (Air Content in percentage)
= 0.98

NOTE: The values for concrete slump, percentage of fines, air content and cement content are based on data obtained from original concrete design mix of the Complex walls.

Cement content coefficient, S_c

$$S_c = 0.75 + 0.034B, \text{ where } B = 6.60 + 0.95 \text{ (number of 94 lb. sacks of cement and pozzolan per cu. yd. of concrete)}$$
$$= 1.00$$

$$\therefore \ \varepsilon_{sh} = 990 \times 0.62 \times 0.80 \times 0.30 \times 1.03 \times 0.86 \times 0.98 \times 1.00$$
$$= 128 \times 10^{-6} \text{ in/in}$$

This is not substantially different than the value derived from the published literature, which was used in the response to NRC Question No. 13, and thus has no significant impact on the response provided to that question.

The grouted masonry block walls, along with their continuous reinforcing steel, will inhibit the unrestrained free shrinkage of the core concrete. The following analysis of the existing Complex walls illustrates the restraining effect and also determines the value of restrained shrinkage in the wall. In determining the restraining effect, the wall at el. 45' is assumed to be vertically held and the entire height of the wall is considered to tend to shrink down. A 12-inch length of wall is taken for analysis. Thickness of the wall is 30 inches. See figure 2-1 for the analytic model.

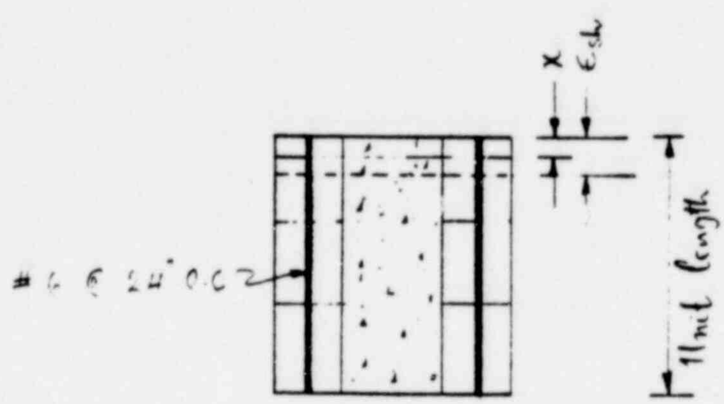

Figure 2-1

A_c = Area of concrete core, inches^2

A_b = Area of cell filled block, inches^2

A_s = Area of reinforcing steel, inches^2

E_c = Modulus of elasticity of concrete

= 4.074×10^6 psi (based on f'_c = 5000 psi and w = 145 pcf)

E_b = Modulus of elasticity of cell filled block (Average of block and cell fill, area of block and cell fill being approximately equal)

= $[22(100^3 \times 2000)^{0.5} + 4.074 \times 10^6]1/2$

= 2.53×10^6 psi

E_s = Modulus of elasticity of steel
= 29×10^6 psi

ε_{shu} = Unrestrained shrinkage strain
= 128×10^{-6} in/in

χ = Restrained shrinkage strain

C_t = Creep coefficient
= 0.88

Assuming creep coefficient of cell filled masonry to be the same as that of concrete,

Effective modulus of elasticity of concrete = $\dfrac{E_c}{1 + C_t}$

Effective modulus of elasticity of block = $\dfrac{E_b}{1 + C_t}$

$$f_s = \chi E_s \quad ; \quad f_b = \frac{\chi E_b}{1 + C_t} \quad ; \quad f_c = (\varepsilon_{shu} - \chi)\frac{E_c}{1 + C_t}$$

From force equilibrium

$$f_s A_s + f_b A_b = f_c E_c$$

or $$\chi\left[A_s E_s + \frac{A_b E_b}{1 + C_t}\right] = (\varepsilon_{shu} - \chi)\frac{A_c E_c}{1 + C_t}$$

for $A_s = 0.44$ in^2/ft ; $A_b = 2 \times 8 \times 12 = 192$ in^2 ;
$A_c = (30 - 16) \times 12 = 168$ in^2

$$\text{or, } {}_{X}\left[(0.44 \times 29) + \frac{192 \times 2.53}{1.88} + \frac{168 \times 4.074}{1.88} \right] \times 10^{6} =$$

$$\frac{128 \times 10^{-6} \times 4.074 \times 10^{6} \times 168}{1.88}$$

$$\text{or, } {}_{X} = 73 \times 10^{-6} \text{ in/in}$$

This value is only 4.3% higher than the restrained shrinkage specified in response to NRC Question No. 13 and, therefore, would not alter the magnitude of dead load distribution due to the effect of shrinkage as given in that response.

Licensee's response to NRC Question No. 22 assumed a conservative restrained shrinkage value of 200 x 10^{-6} for the existing walls for the limited purpose of calculating bolt tension losses. Before erecting the new wall adjacent to the existing wall with the 3 inch thick steel plate as the outside form, the surface of the existing block face will be sprayed with water. This will moisten the block and possibly some of the cell fill concrete and would cause some amount of swelling. The bolt loss from shrinkage for this swelled portion of the existing wall would occur only if the entrapped moisture finds a path to diffuse to the outside environment. This diffusion process would be inhibited by the steel plate on one side and the core concrete (where existing) and the outside core filled masonry wythe. Furthermore, any loss in bolt stress due to this effect would be detected during the surveillance and the bolt stress would be monitored to ensure that it did not fall below the design stress level. Considering the above, and also noting that an unrestrained shrinkage for the entire 30 inch thick existing wall is only 128 x 10^{-6} in/in, a shrinkage strain of 200 x 10^{-6} in/in for the swelled portion of the in-situ wall for the purpose of calculating bolt losses is an appropriately conservative figure.

2. New Walls

For the new wall elements, an analysis similar to the one described above was performed to provide a basis for Licensee's response to NRC Question No. 22, dated September 5, 1979. However, the thickness effect, as given by the term S_{th}, was conservatively taken as 0.84, which is applicable for a 9 inch thick wall only. Consequently, if the thickness coefficient is appropriately modified to correspond to the actual wall thickness, the resulting strain will be substantially reduced from the 174 x 10^{-6} in/in shrinkage strain shown in that response. Also, the strain of 174 x 10^{-6} in/in was conservatively established as the remaining shrinkage in the new walls after 28 days from the time of pouring. This was the minimum time envisaged for tightening the bolts. That analysis differed from Licensee's response to NRC Question No. 11, dated September 5, 1979 which described the evaluation of tension forces in the new walls which result from interaction between the newly cast concrete and the existing wall. Recognizing that the new walls would be kept moist for the first seven days, during which period shrinkage of the wall would not take place, only the shrinkage occurring after that period would have to be considered. Hence, the factor, S_t, which was taken as 0.62 in deriving the value of 174 x 10^{-6} in/in w. taken as 1.0 and the total shrinkage strain was calculated as $(174 \times 10^{-6})/0.62 = 280 \times 10^{-6}$ in/in. It should be noted here that in deriving this strain the thicknes. effect was also very conservatively taken as that for 9 inc. walls, and consideration of the actual wall thickness would substantially reduce this value.

The concrete design mix used in the construction of the in-situ composite walls of the Complex is given in Table 2-1. The information provided in this table was compiled from the data given for 3/4 in. aggregate and concrete mix D1 as they appear in Table 3.8.17 of the Trojan FSAR. The mix design for the new concrete walls will be made using aggregates which have less shrinkage characteristic.

References:

1. Branson, D. E., and Christiason, M. L., "Time Dependent Concrete Properties Related to Design Strength and Elastic Properties, Creep, and Shrinkage", ACI Publication No. SP27-13.

2. "Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structure", Reported by ACI Committee 209, ACI Publication No. SP27-3.

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TABLE 2-1

Concrete Mix Design Used in the In-Situ Walls of the Complex

	psi	lb	cu ft	sacks	%	W/C	oz	lb/ft^3
Strength	5000							
Cement		620	3.15	6.60				
Pozzolan		85	0.55	0.90				
Sand		1137	7.06		40			
3/4 in. Aggregate		1760	10.60		60			
Water		310	4.96			0.44		
WRA							11.3	
AEA			0.68				4.5	
Total		3912						144.9

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NRC Questions (9/28/79)

DRAFT

Q. 1 10/16/79 1:00 PM

Verify that the installed Hexcel energy absorbing material will be

(a) "stabilized" in order to ensure the edge material is stabilized and therefore will absorb the anticipated amount of energy should it be crushed by a falling plate.

(b) "precrushed" in order to eliminate the peak load shown in Figure V-2 of Hexcel catalog #TSB-120.

Answer:

(a) The Hexcel energy absorbing material will be "stabilized" by bonding a plate on the top and bottom of the material.

(b) The Hexcel energy absorbing material will be "precrushed" in order to eliminate the peak load shown in Figure V-2 of Hexcel catalog #TSB-120.

NRC Questions (9/28/79)

DRAFT

Q. 2 10/16/79 8:00 AM

Previous responses have indicated, in response to the control of dust, grit and debris, that the work area may be isolated. In this regard, the staff believes a small portable enclosure should be employed on the east and west inside walls of the Control Room and the electrical auxiliaries room when drilling holes in the walls. This box shall be capable of containing and collecting any dust, dirt, debris and water that may enter the room as the drill penetrates the wall.

Verify that such a small enclosure and collection means will be provided in order to preclude the release of this material inside the rooms.

Answer:

A small enclosure will be used on the inside of the walls as outlined in the above question. It will be constructed so as to collect and contain any dust, dirt, debris and water incidental to the drilling. It will also be constructed so that a workman can hold the enclosure against the wall with his hands and at the same time be able to see the wall to determine when and where the drill bit is penetrating. Additional measures to control dust, grit and debris are described in response to Question 7 of this set.

NRC Questions (9/28/79) DRAFT

Q. 3 10/16/79 11:00 AM

Confirm that the required control room differential pressure requirements (Technical Specification 4.7.6.1.d.3) can be continuously maintained with open drilled holes in the control room wall. Provide the basis for your conclusion. Also, confirm that these requirements can be met during installation of Plate 8.

Answer:

The referenced Technical Specification requires periodic verification that the Control Room emergency ventilation system, CB-1, is capable of maintaining a positive pressure in the Control Room relative to the outside atmosphere during certain specified events.

Each hole drilled into the Control Room will be temporarily plugged before the next hole is drilled. Therefore, there will be no more than one 3" hole open into the Control Room at any one time due to the modification program. Such a hole would not reduce the capability to maintain a positive pressure. During installation of Plate 8, as each bolt is placed through the hole in the Control Room wall an "O" ring will be placed in the annulus between the bolt and the concrete on the Control Room side of the drilled hole. This "O" ring will be removed immediately prior to grouting the bolt hole, thus preserving the capability to achieve the Control Room pressure differential during the process of installing Plate 8.

The Trojan response of September 5, 1979 to Systems Branch question 10 is confusing in that it speaks of areas external to Category 1 equipment. The staff believes that a fire watch patrol should be established to perform hourly inspections for areas where a fire could affect safety related cables or equipment in which non-fire retardant wood will be used for concrete forms or other purposes.

The person while assigned as a fire watch patrol should have no other duties. This fire watch patrol should be instituted when the non-fire retardant wood is taken into any of these areas and continue until it is removed. The fire watch patrol would not be necessary during the times when a continuous fire watch has been established in an area for other reasons. Identify each of the areas where such a fire watch patrol would be necessary to monitor for fires in areas where a fire could affect safety-related cables or equipment.

Answer:

The intent of Licensee's response dated September 5, 1979, to Systems Branch Question 10(i) was to indicate that, during the modification program described in PGE-1020, Licensee will establish a fire watch patrol when non-fire-retardant wood is utilized in areas where a fire could affect safety-related cables or equipment. The fire watch patrol will perform hourly inspections from the time the nontreated wood is brought into any such area until it is removed, and will not be assigned other duties. The areas where such a fire watch patrol might be necessary are listed as Areas 1, 2, 3, and 4 in Licensee's response dated ________________________ to NRC Question 3 of September 14, 1979.

NRC Questions (9/28/79) DRAFT

Q. 5 10/16/79

In reference to the construction noise levels in the control room, response 18 to the staff's July 20, 1979 questions, you indicated that "Should it be determined by the plant operator in the Control Room that excssive noise is being created, lighter weight tools or other means of concrete removal will be employed". The staff believes it is essential that if either the NRC IE resident inspector or the plant operator should determine that excessive construction noise is being created, lighter weight tools or other means of concrete removal will be employed.

Verify that the above additional control on control room noise is acceptable and will be complied with.

Answer:

In the event that either the NRC IE Resident Inspector or the Plant operator determines that excessive construction noise is being created, lighter weight tools or other means of concrete removal will be employed.

Q. 6 10/16/79 8:00 AM

Presently it is proposed to utiliz> a positive feed control drill on the east and west control building outer walls. Further a person will be stationed on the inside for the purpose of detecting when the wall has been penetrated and notifying the driller via radic communications or by sound or battery powered telephones. Describe and discuss any other additional measures that can and will be implemented to further provide assurance the drill will not be allowed to penetrate to such an extent as to damage equipment within, e.g., positive stops or a paint strip on the core drill to alert the driller that wall penetration is imminent.

Answer:

Conventional practice for such drilling operations includes the use of marking on the core drill so that the drill operator knows where his drill bit is located in relation to his planned penetration depth. Such a marking procedure will be used for all concrete or masonry core drilling required for the modification work. The type of marking used will be one that the drill operator can easily see while operating the drill. Either a tape or painted stripe is the method which we would plan to use.

1435 ?14

NRC Questions (10/2/79)

DRAFT

Q. 5 10/16/79 10:00 AM

Your July 6 response to question 16 indicates that the vertical shear forces at corners R-55 and N-55 are 2357 kips and 1260 kips, respectively. Section 3.5 of PGE-1020, Revision 2 indicates that these same forces are 1686 kips and 1593 kips, respectively. Provide the correct shear forces.

Answer:

The shear force values which appear in Licensee's July 6, 1979 response to NRC Question 16 were taken from PGE-1020, Revision 1. The values in PGE-1020 Rev 1 were based on the results of an analysis of a STARDYNE model of the Complex with the modifications described in PGE-1020, Rev 0. The shear force values provided in the July 20, 1979 Revision 2 to PGE-1020 are based on the results of an analysis of the current STARDYNE model which incorporates the changes in the modification described in Licensee's letter dated June 22, 1979.

The correct shear forces for the modified Complex at corners R-55 and N-55 are 1686 kips and 1593 kips, respectively, as provided in PGE-1020, Rev 2.

NRC Questions (10/2/79)

Q. 9 10/10/79 9:00 AM

Your June 29 response to question 3 and PGE-1020, Revision 2 indicates that the appropriate factor of safety for the Nelson studs is 2. Your June 22 response to question 22 indicates that a factor of 3 was used in the design of the studs and, therefore, may be more appropriate. Clarify this apparent inconsistency.

Answer:

In PGE-1020 Section 3.2.4.3 and in Licensee's response dated June 29, 1979 to NRC Question No. 3, it is stated that the allowable design values for Nelson studs are one-half of the values given in Table 15 of the Nelson Division of TRW, Inc. publication, "Design Data 10 - Embedment Properties of Headed Studs". A justification for the allowable design values is presented in Licensee's response dated June 22, 1979 to NRC Question No. 7

Licensee's response dated June 22, 1979 to NRC Question No. 22 indicates that the maximum calculated forces on the studs are one-third of the values given in Table 15 of "Design Data 10 - Embedment Properties of Headed Studs". Since the calculated forces are less than the allowables, the design of the studs is adequate.

NRC Questions (10/2/79)

DRAFT

Q. 16 10/16/79

Your July 10 response to question 13 indicates that the maximum vertical amplification factor is 16 percent while your September 5 response to question 15 indicates that it is 13 percent. Therefore, provide the correct maximum vertical amplification factor.

Answer:

Licensee's response dated July 10, 1979, to NRC Question No. 13 stated that the maximum vertical amplification factor is 16 percent. Licensee's response dated September 5, 1979, to NRC Question No. 15 states that "the dead load was reduced 13% to account for vertical motion". Thus, the 13% is the reduction in dead load, and is not a value for vertical amplification.

NRC Questions (10/2/79)

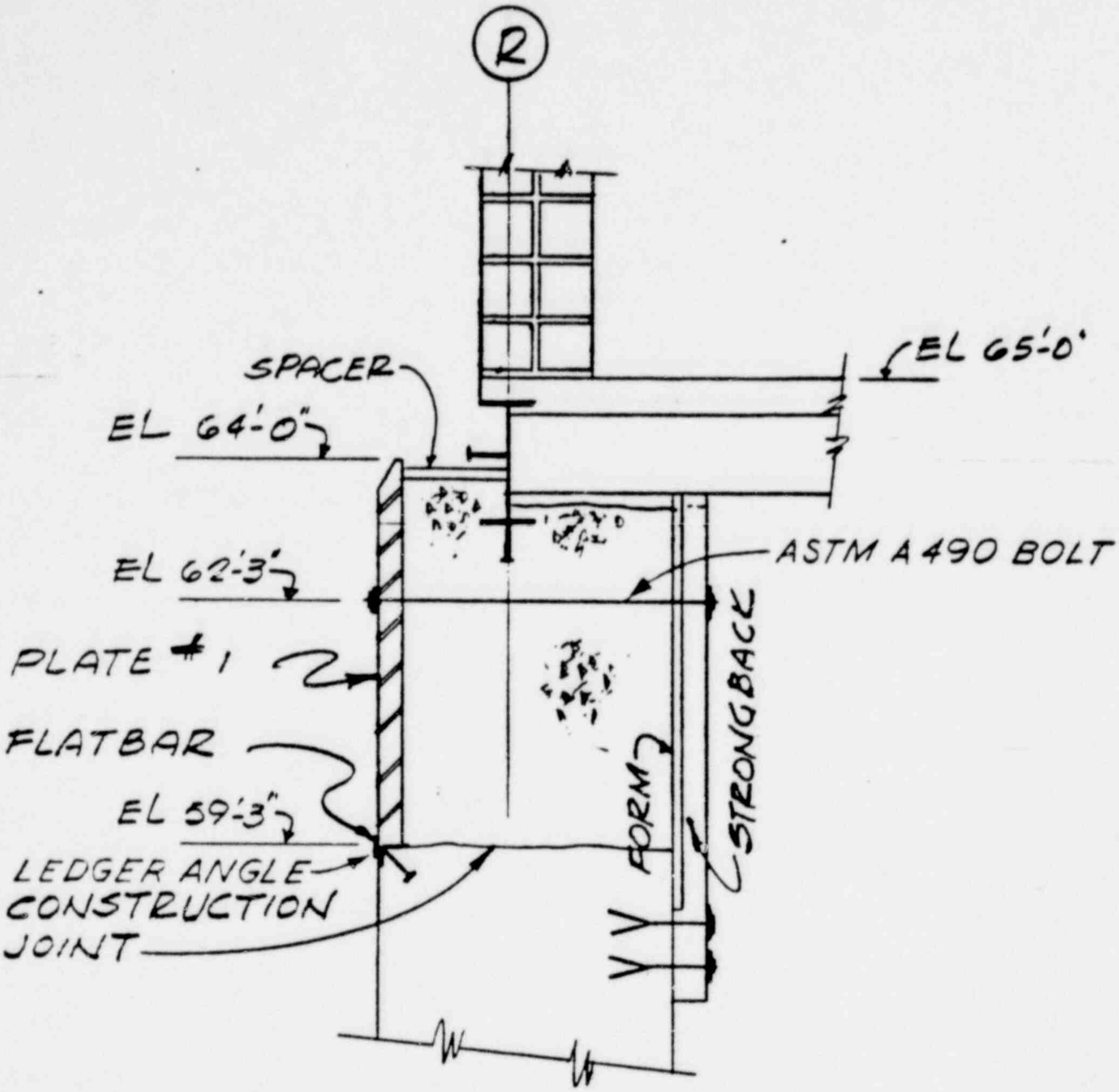
DRAFT

Q. 18 10/16/79 3:00 PM

In your September 5 responses to questions, the response to question 17 indicates that for the combination of dead, live and SSE loadings, the maximum allowable stress in bending and tension is limited to 0.9 fy and the maximum allowable shear stress is limited to 0.5 fy. Verify that this limitation was imposed for the evaluations of steel elements discussed in the responses to questions 18 and 25.

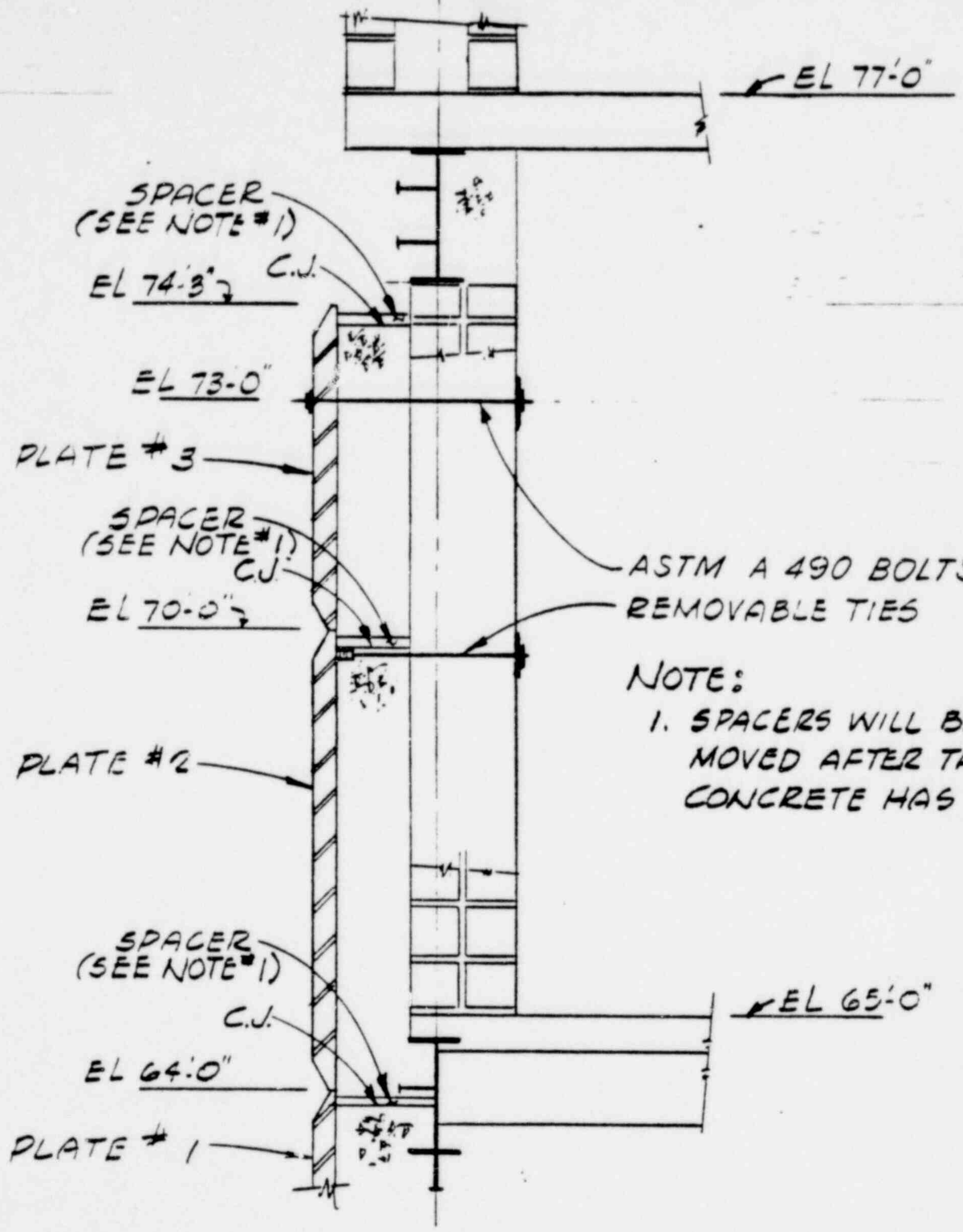
Answer:

In Licensee's responses dated September 5, 1979 to Structural Branch Questions Nos. 18 and 25, the maximum allowable stress in bending and tension of the steel elements was limited to 0.9 f_y and the maximum allowable shear stress was limited to 0.5 f_y for the load combinations referred to.



R
EL 65'-0"
SPACER
EL 64'-0"
EL 62'-3"
ASTM A 490 BOLT
PLATE #1
STRONGBACK
FLATBAR
FORM
EL 59'-3"
LEDGER ANGLE
CONSTRUCTION
JOINT

FIGURE 8-1



NOTE:

1. SPACERS WILL BE REMOVED AFTER THE CONCRETE HAS SET.

FIGURE 8-2

1435 220