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UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

BEFORE THE ATOMIC SAFETY AND LICENSING BOARD

In the Matter of	) Docket 50-344
PORTLAND GENERAL ELECTRIC COMPANY, et al	) (Control Building Proceeding)
(Trojan Nuclear Plant)	)

CERTIFICATE OF SERVICE

I hereby certify that on June 29, 1979, Licensee's letter to the Director of Nuclear Reactor Regulation dated June 29, 1979 and an attachment entitled "Request for Additional Information, Trojan Nuclear Plant, Proposed Control Building Design", have been served upon the persons listed below by depositing copies thereof in the United States mail with proper postage affixed for first class mail.

Marshall E. Miller, Esq., Chairman
Atomic Safety and Licensing Board
U. S. Nuclear Regulatory Commission
Washington, D. C. 20555

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

Dr. Hugh C. Paxton
1229 - 41st Street
Los Alamos, New Mexico 87544

Atomic Safety and Licensing Board
Panel
U. S. Nuclear Regulatory Commission
Washington, D. C. 20555

Atomic Safety and Licensing Appeal
Panel
U. S. Nuclear Regulatory Commission
Washington, D. C. 20555

Docketing and Service Section
Office of the Secretary
U. S. Nuclear Regulatory Commission
Washington, D. C. 20555

Joseph R. Gray, Esq.
Counsel for NRC Staff
U. S. Nuclear Regulatory Commission
Washington, D. C. 20555

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

Richard M. Sandvik, Esq.
Assistant Attorney General
State of Oregon
Department of Justice
500 Pacific Building
520 S. W. Yamhill
Portland, Oregon 97204

William Kinsey, Esq.
Bonneville Power Administration
P. O. Box 3621
Portland, Oregon 97208

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7908080067

CERTIFICATE OF SERVICE

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

Mr. John A. Kullberg
Route 1, Box 250Q
Sauvie Island, Oregon 97231

Mr. David B. McCoy
348 Hussey Lane
Grants Pass, Oregon 97526

Ms. C. Gail Parson
P. O. Box 2992
Kodiak, Alaska 99615

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

Columbia County Courthouse
Law Library
Circuit Court Room
St. Helens, Oregon 97051

Ronald W. Johnson

Ronald W. Johnson
Corporate Attorney
Portland General Electric Company

Dated: June 29, 1979

4sb66.27B8

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Portland General Electric Company

June 29, 1979

Trojan Nuclear Plant
Docket 50-344
License NPF-1

Director of Nuclear Reactor Regulation
ATTN: Mr. A. Schwencer, Chief
Operating Reactors Branch #1
Division of Operating Reactors
U.S. Nuclear Regulatory Commission
Washington, D. C. 20555

Dear Sir:

Enclosed are responses prepared by Bechtel Power Corporation to an additional 14 of the 50 questions submitted in your letter of May 18, 1979. In accordance with our discussion with your staff, we expect to transmit responses to the remaining questions by July 6.

Sincerely,

Ronald W. Johnson

R. W. Johnson
Corporate Attorney
Portland General Electric Company

RWJ/4sb5B16
Enclosure

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Provide clear, detailed sketches and descriptions of the connection interfaces of the additional walls to the existing structure. Additionally, describe the methods by which the effects of concrete creep and shrinkage (causing tension in the walls and/or a reduction in assumed dead weight) have been factored into the design of these additional walls. Describe and justify in detail the design and the procedures for the connections of the new walls to the existing structure.

Answer:

Detailed sketches of the connection interfaces between the new walls and the existing structure are attached. The final design may require some minor revisions to the actual sizes and spacings of the rebar and studs. The sketches show representative connection details that will be used. The connection interfaces are discussed below.

Typically, where steel beams occur at a horizontal interface, studs are used to transfer shear forces and vertical rebars are used to transfer tension forces. At horizontal interfaces where steel beams do not exist, vertical rebars make the connection. In some cases friction type A-490 bolts are also used in the connections between the new walls and the existing concrete.

The typical treatment for vertical interfaces is to expose the columns and weld studs o o them. New rebars are spliced with the exposed horizontal existing rebars by cadwelding. Drilled and grouted horizontal rebars are also used to make the connections.

The effects of the concrete creep and shrinkage have been considered in the design of the new walls. Creep is a gradual increase in strain with time when concrete is under sustained stress. The nature of the new walls for the modifications of the Complex is such that, except for their own weight, they will be under stress only during a seismic occurrence, which is of short duration. Therefore, the creep effect on these walls is not considered to be significant.

The shear strength provided by the concrete, V_c, is calculated in accordance with equations (11-33) and (11-34) of ACI 318-77, where N_u represents the tension due to shrinkage. Also in the same equations the beneficial effect of the dead weight of the new walls and the walls above them is neglected, resulting in a lower value of V_c.

The design for the connections of the new walls to the existing structure is in accordance with ACI 318-77.

The tension connection provided by the rebars is calculated by the strength design method.

The shear connections are designed by one of the following means:

i) Shear studs welded on to the structural steel members transmit shears between concrete and steel.

The design value for the studs is considered to be one-half the value given in Table 15 of the Nelson Division of TRW Inc. publication, "Design Data 10 - Embedment Properties of Headed Studs." (The design value of the shear studs is further elaborated in the answer to question No. 7).

ii) ASTM A-490 friction type bolts transfer shear from structural steel members to concrete, or from an existing wall to a new wall. They have the capacity of transmitting 52.5 kips of shear per bolt. (This value is further elaborated in the answer to question No. 6).

iii) The design of the vertical shear transfer mechanism between the existing wall and the new wall poured against it is based on the provisions of ACI 318-77.

The procedures to be followed in the construction of the connections are:

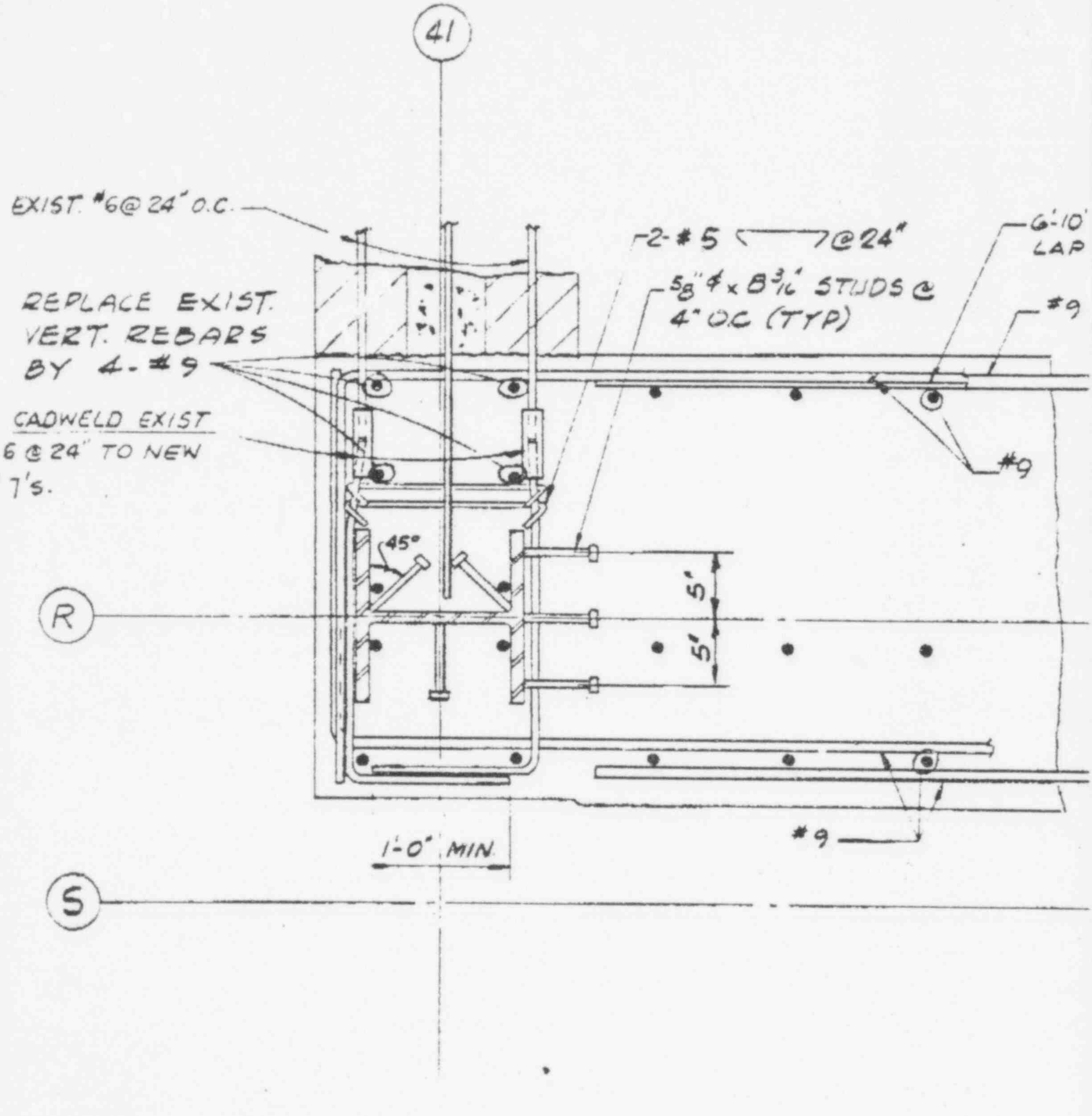
a. Surface preparation of the existing concrete will be in accordance with paragraph 11.7.9 of ACI 318-77 and paragraph 6.4.1 of ACI 349-76.

b. Surfaces of the steel to receive studs will be cleaned and studs will be welded in accordance with the stud manufacturer's recommendations.

c. Reinforcing bar splices will be in accordance with Sections 12.15 and 12.16 of ACI 318-77. The splices will also comply with the requirements of Sections 7.5 and 7.6 of ACI 349-76, except as noted in Response to Question No. 4. Mechanical connections will be made by the CADWELD method.

d. All work will be performed in accordance with the specifications listed in Paragraph 3.2.2.4 of PGE-1020.

All of the work, including the above procedures, will be performed in accordance with the applicable Codes and Standards and with conventional construction methods.

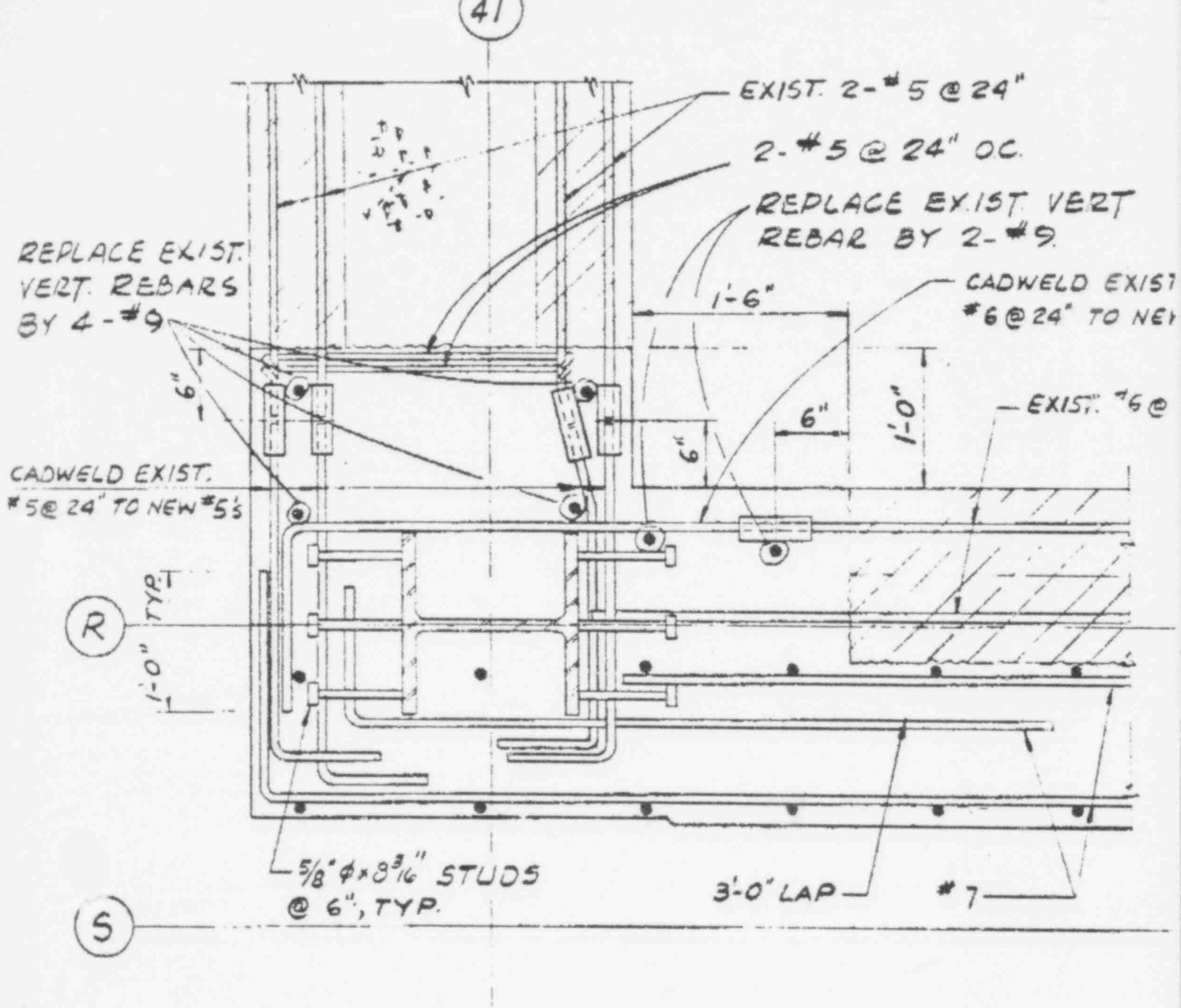


DETAIL AT COLUMN LINES (R) AND (41)

BETWEEN EL 45'-0" AND EL 65'-0"

Figure 3-1

502 181

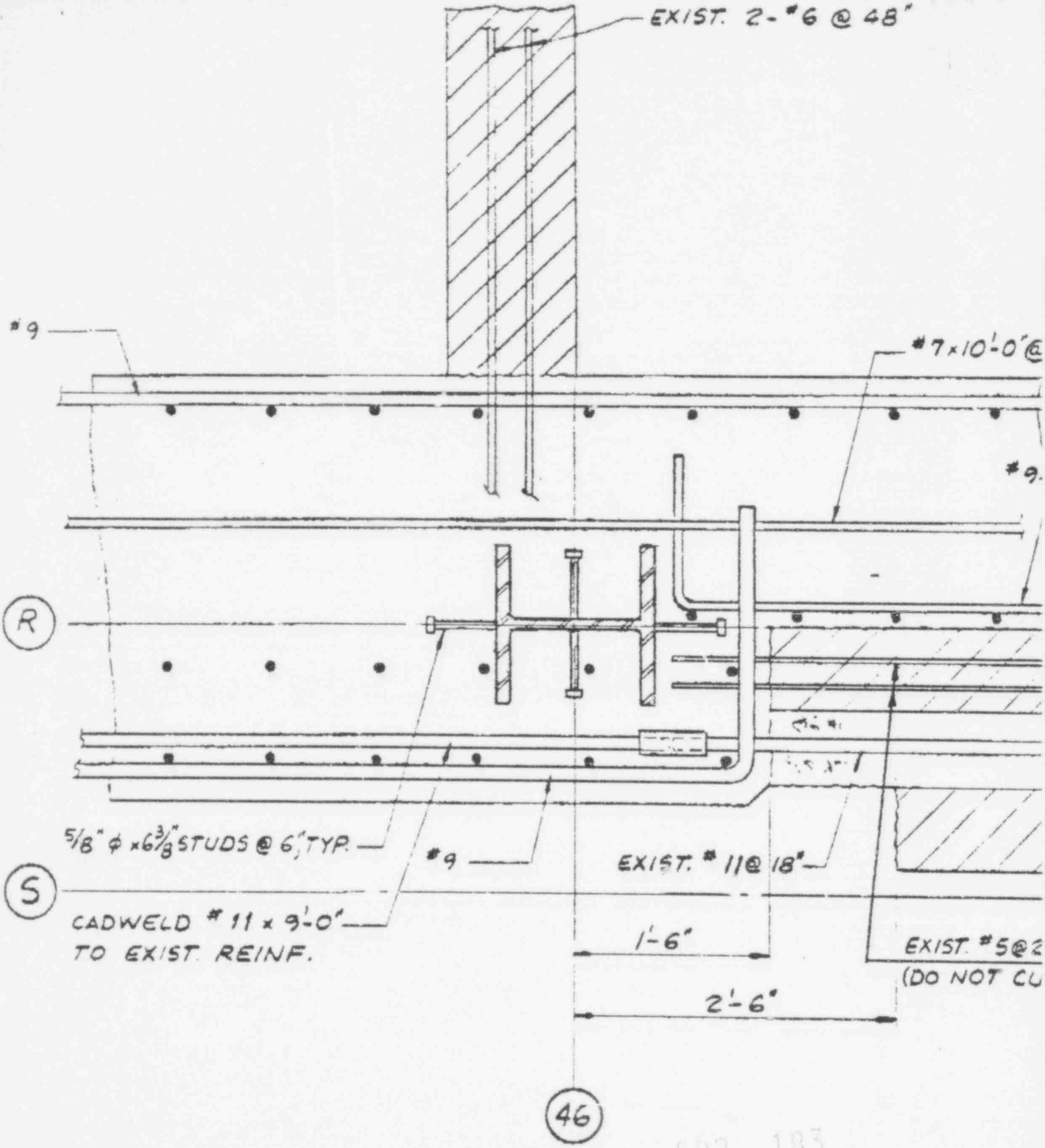


502 182

DETAIL AT COLUMN LINES (R) AND (41)

BETWEEN EL 65'-0" AND EL 77'-0"

Figure 3-2

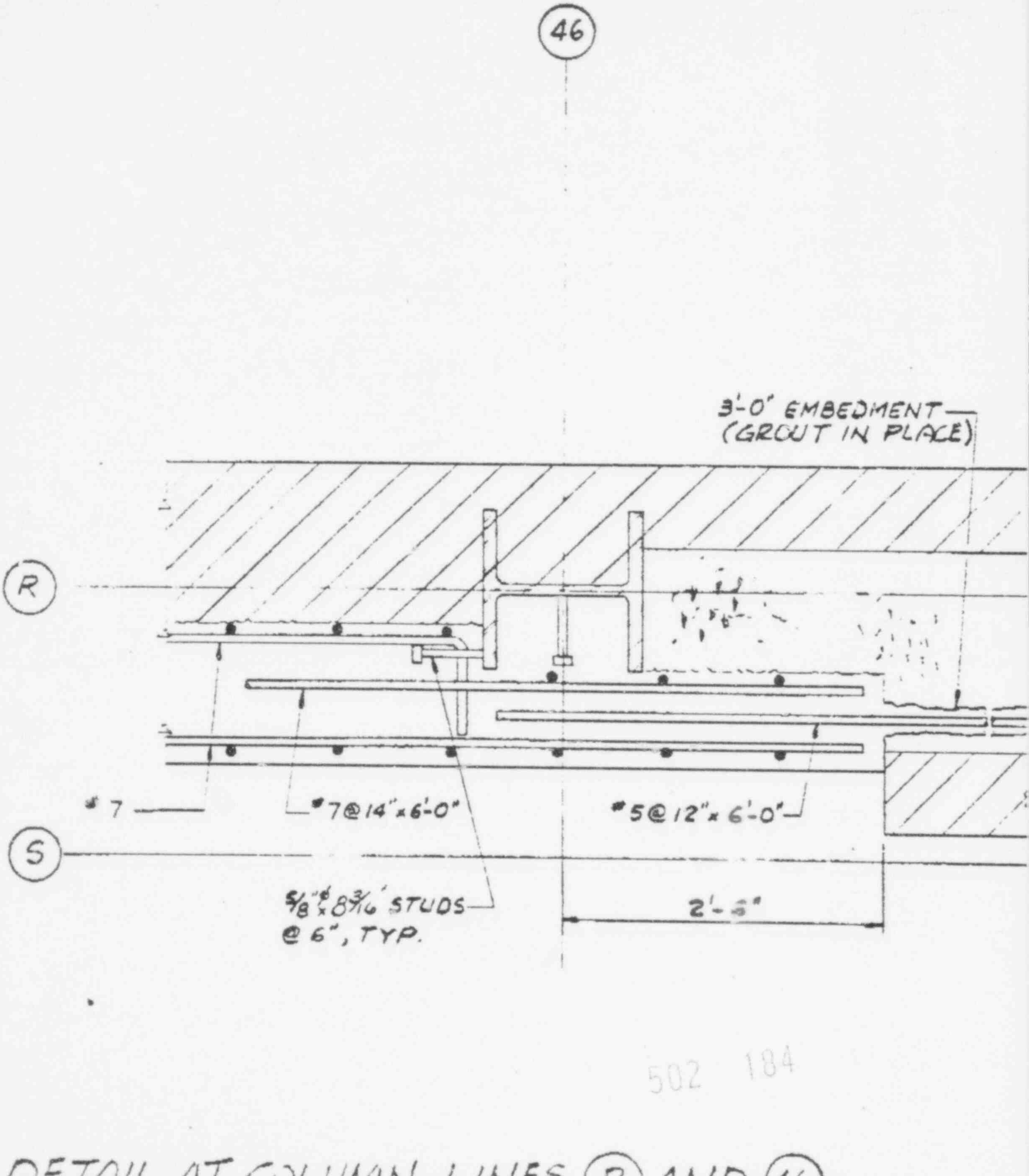


502 183

DETAIL AT COLUMN LINES (R) AND (46)

BETWEEN EL 45'-0" AND EL 65'-0"

Figure 3-3

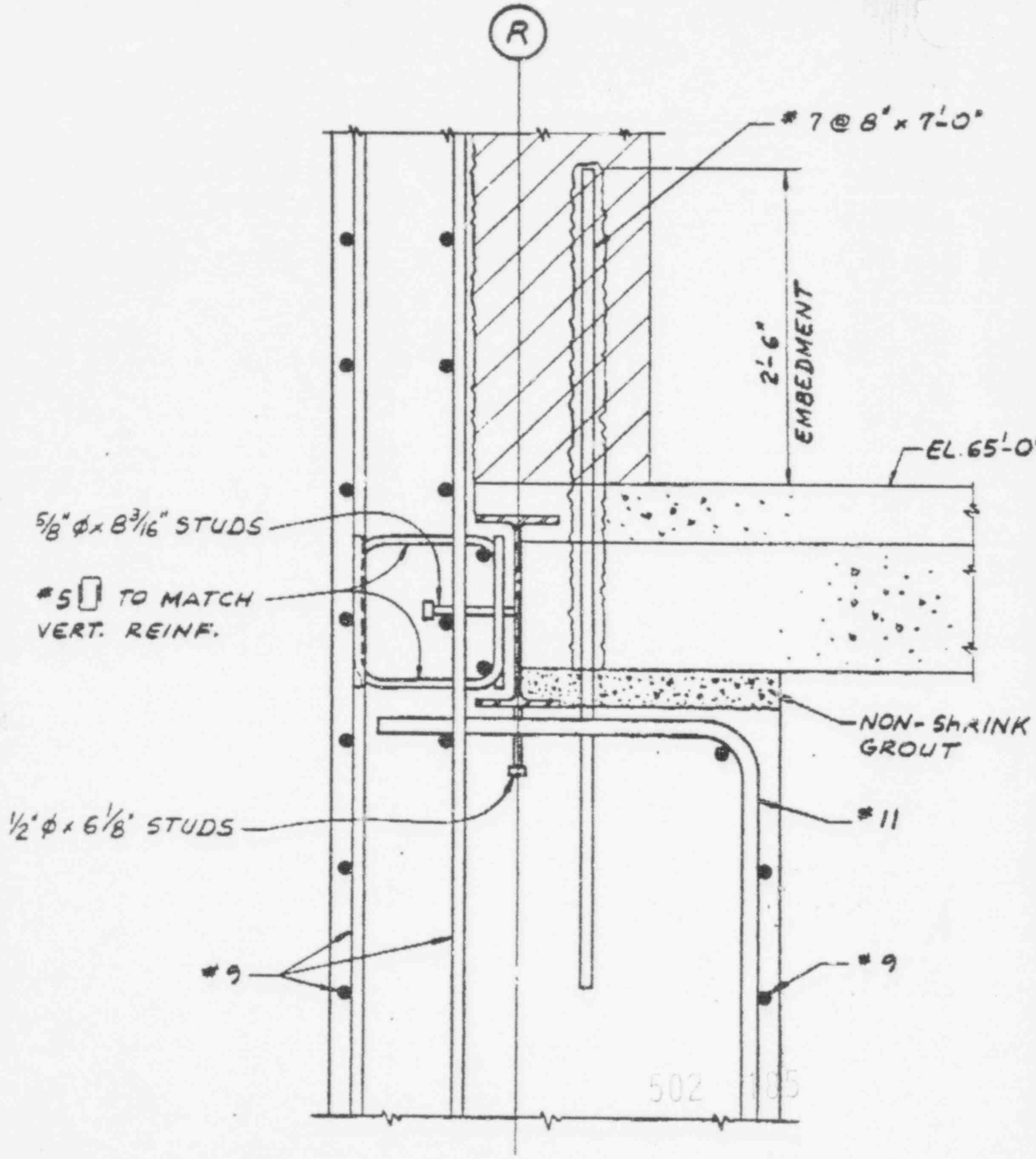


502 184

DETAIL AT COLUMN LINES (R) AND (46)

BETWEEN EL 65'-0" AND EL 77'-0"

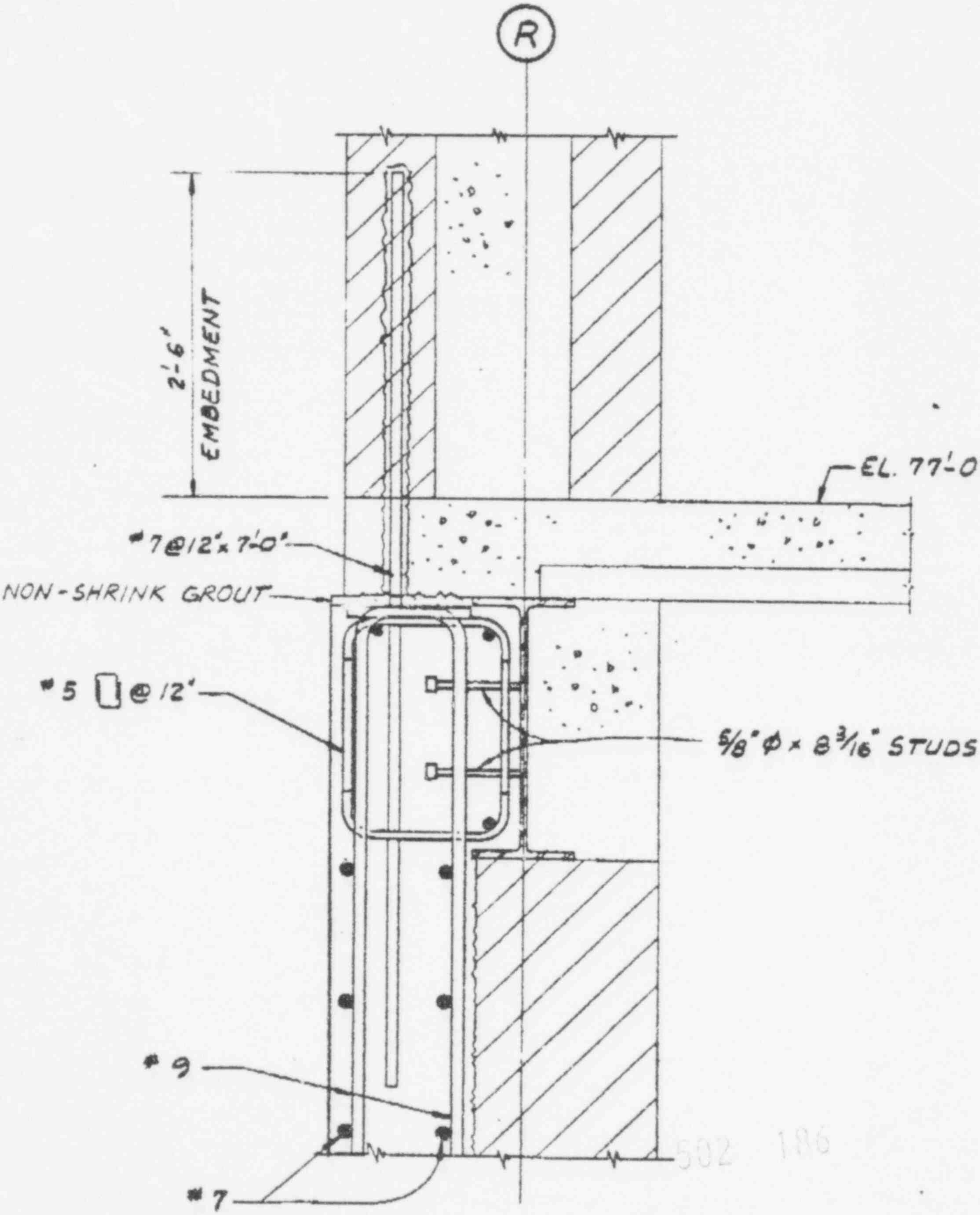
Figure 3-4



FRICTION TYPE BOLTS NOT SHOWN

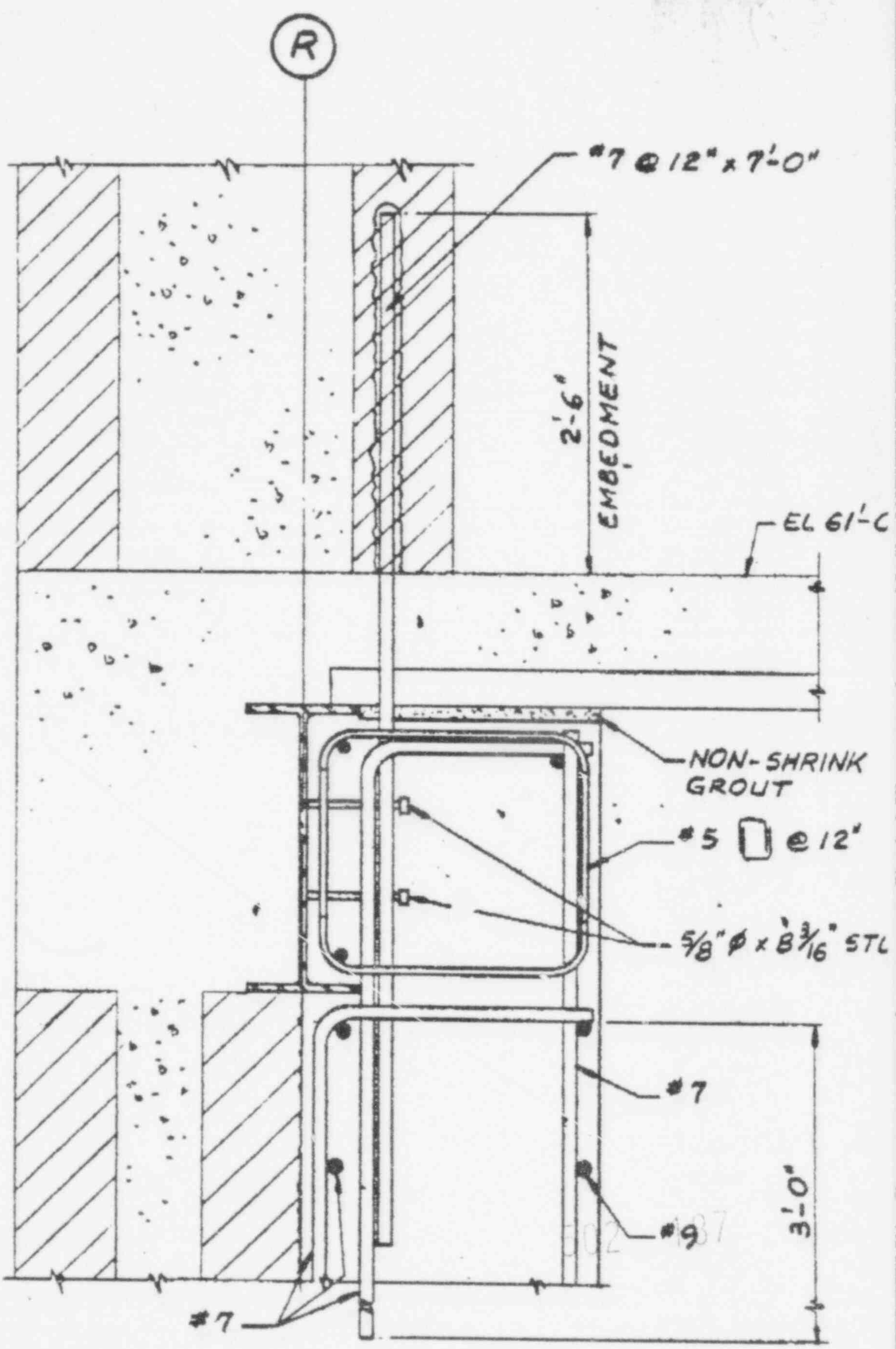
SECTION AT LINE (R) EL. 65'-0"

Figure 3-5



SECTION AT LINE (R) EL. 77'-0"

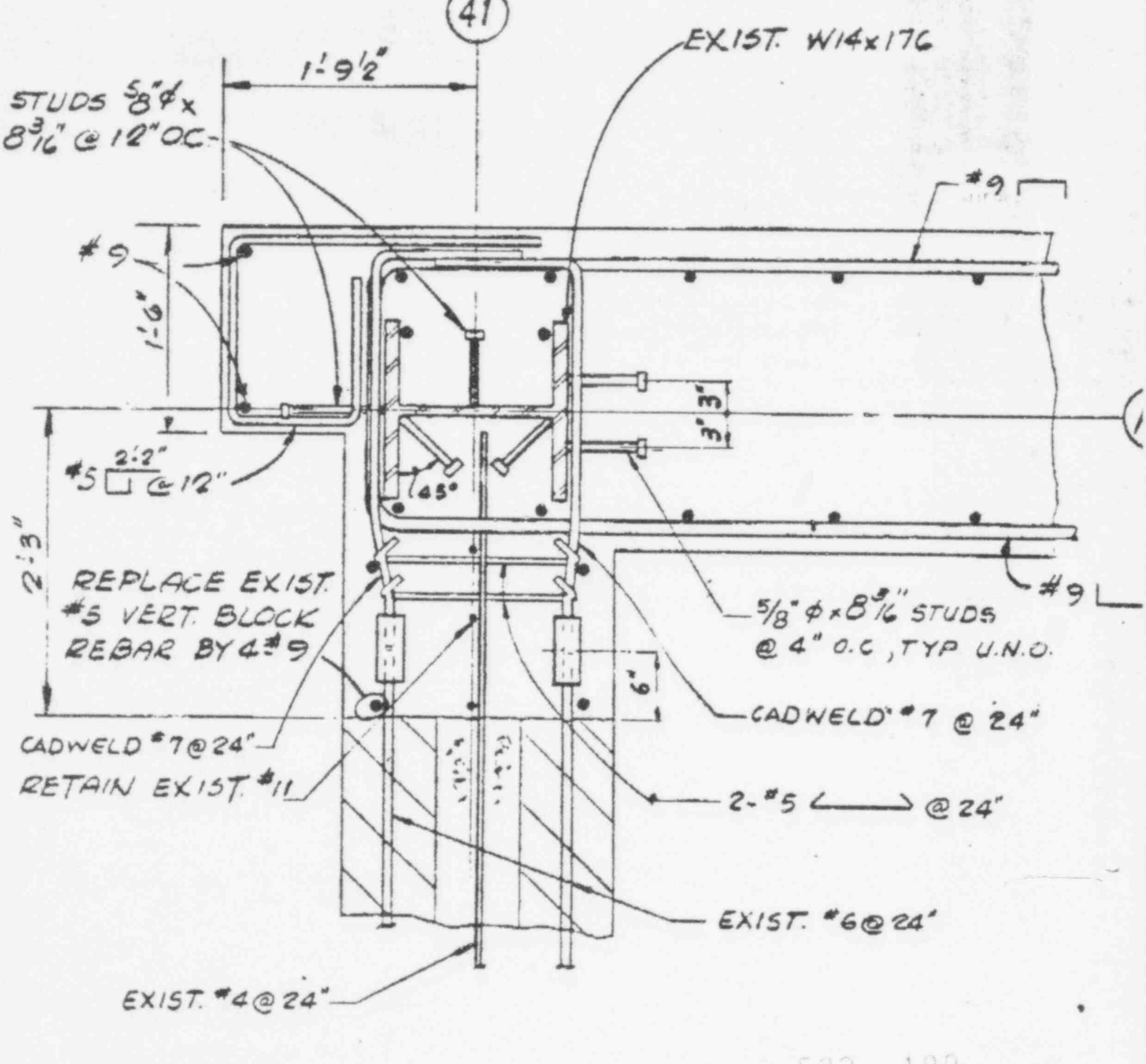
Figure 3-6



FRICTION TYPE BOLTS NOT SHOWN

SECTION AT LINE (R) EL. 61'-0"

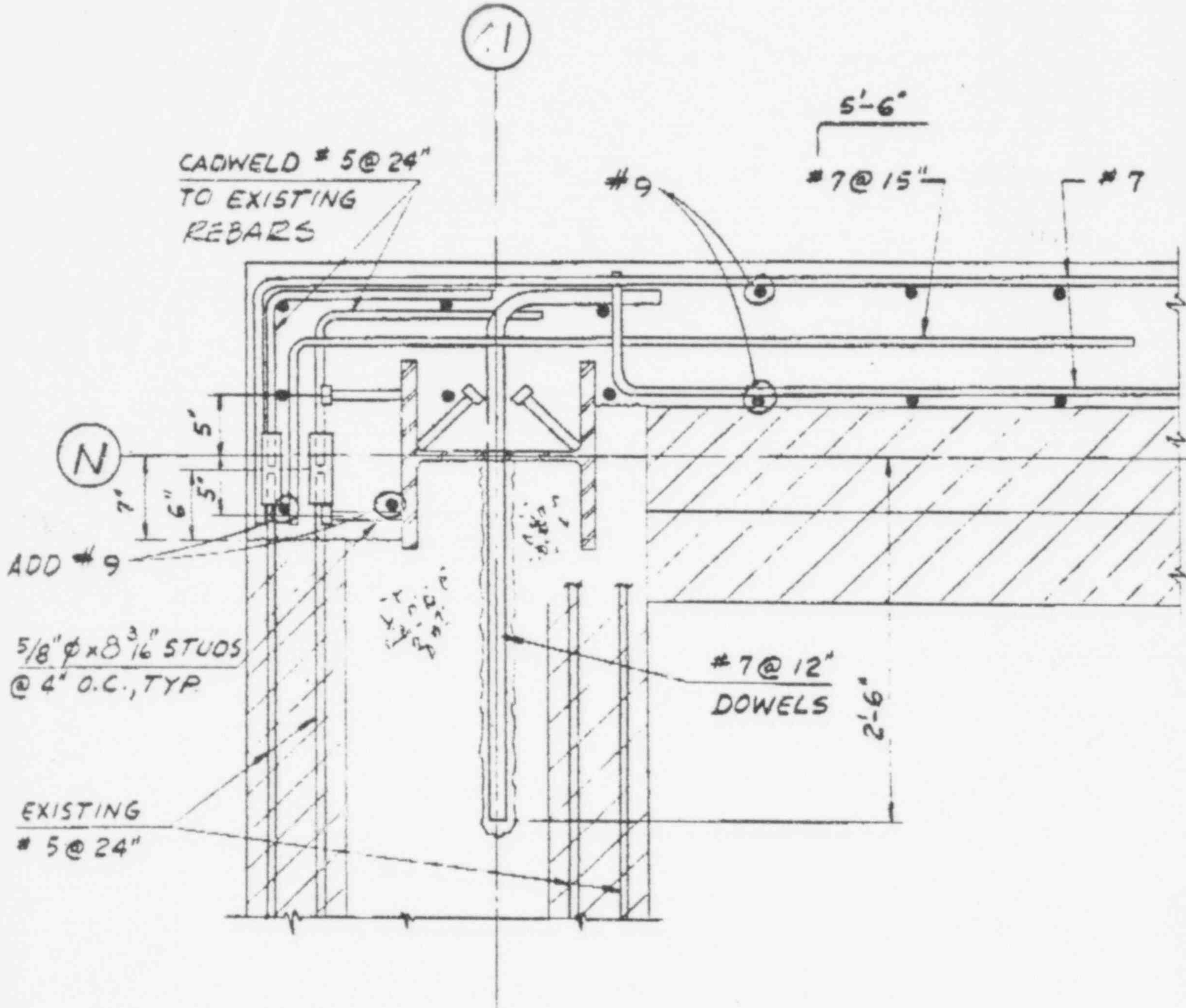
Figure 3-7



502 188

DETAIL AT COLUMN LINES N AND 41 BETWEEN EL 53'-0" AND EL 65'-0"

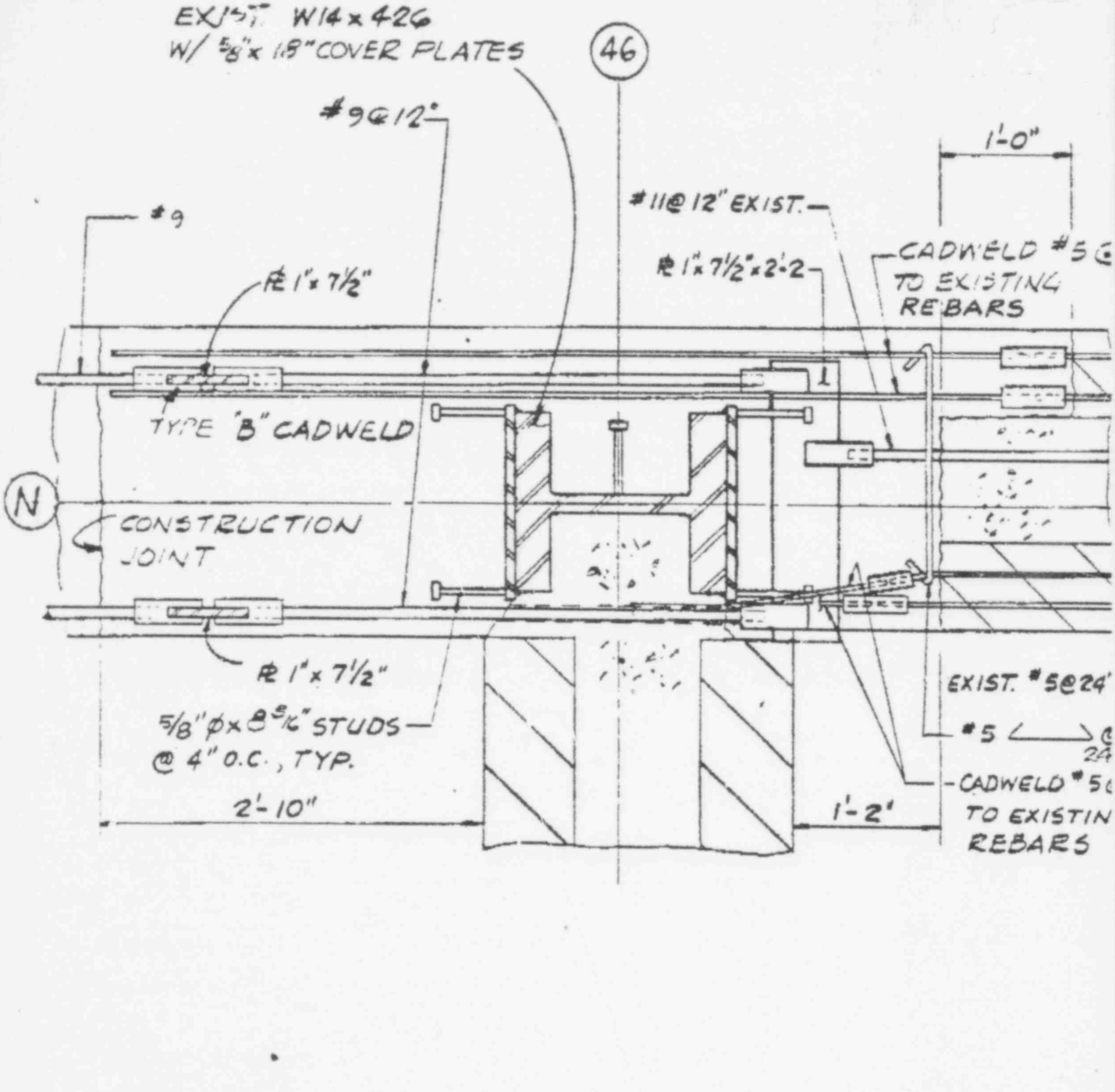
Figure 3-8



DETAIL AT COLUMN LINES (N) AND (41)
BETWEEN EL 65'-0" AND EL 77'-0"

Figure 3-9

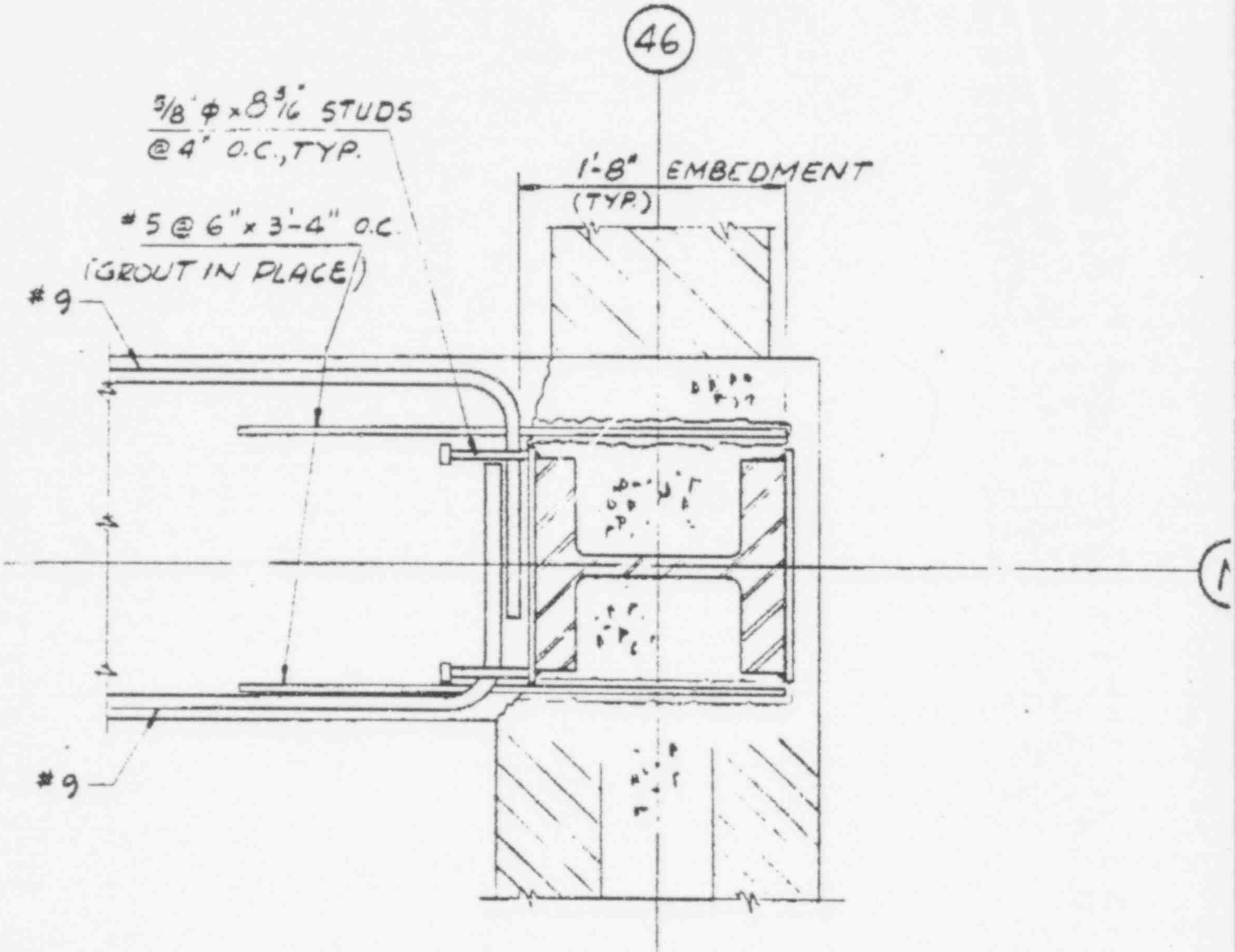
502 189



502 190

DETAIL AT COLUMN LINES (N) AND (46)
BETWEEN EL 45'-0" AND EL 53'-0"

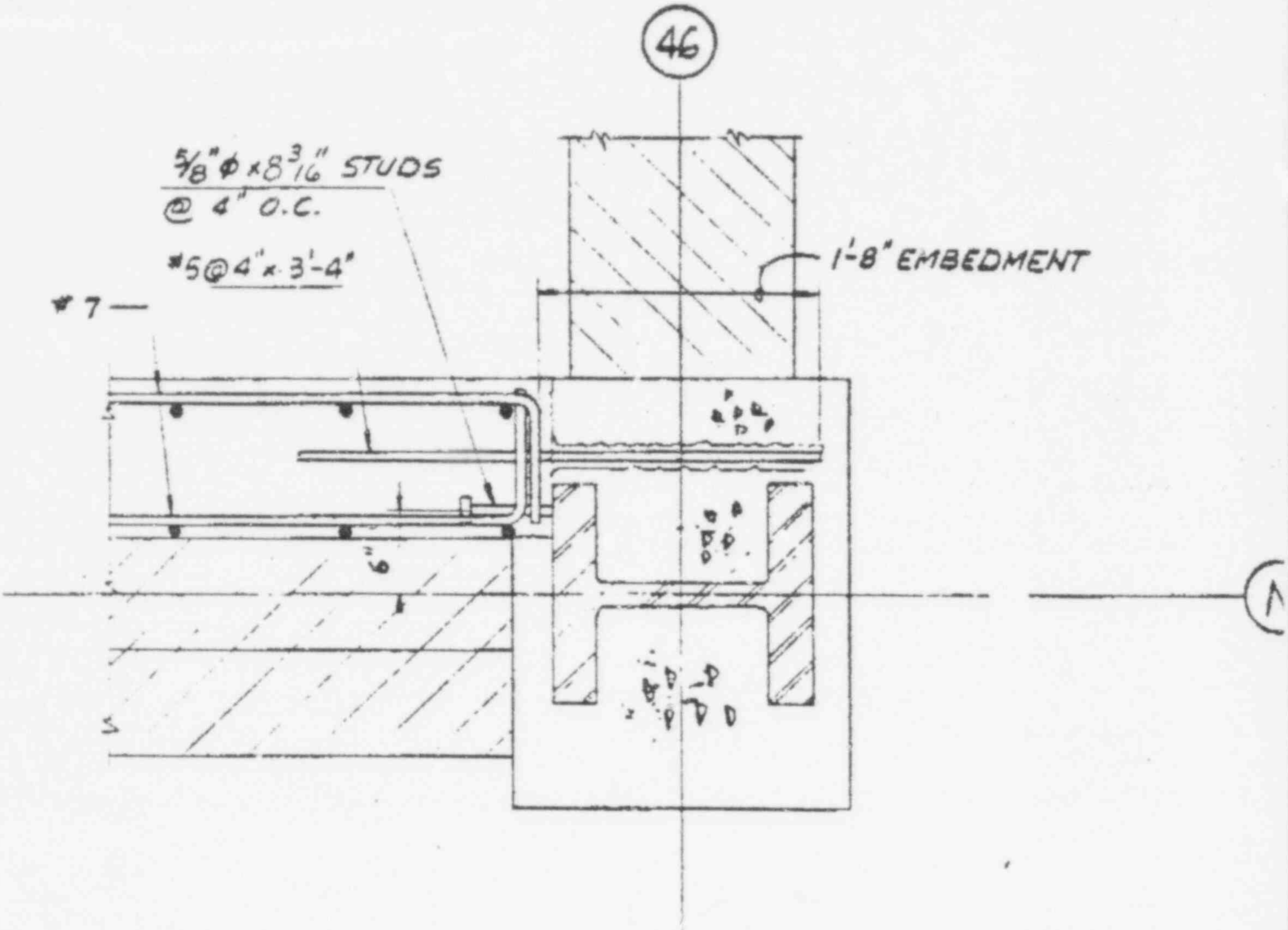
Figure 3-10



502 191

DETAIL AT COLUMN LINES (N) AND (46)
BETWEEN EL 61'-0" AND EL 65'-0"

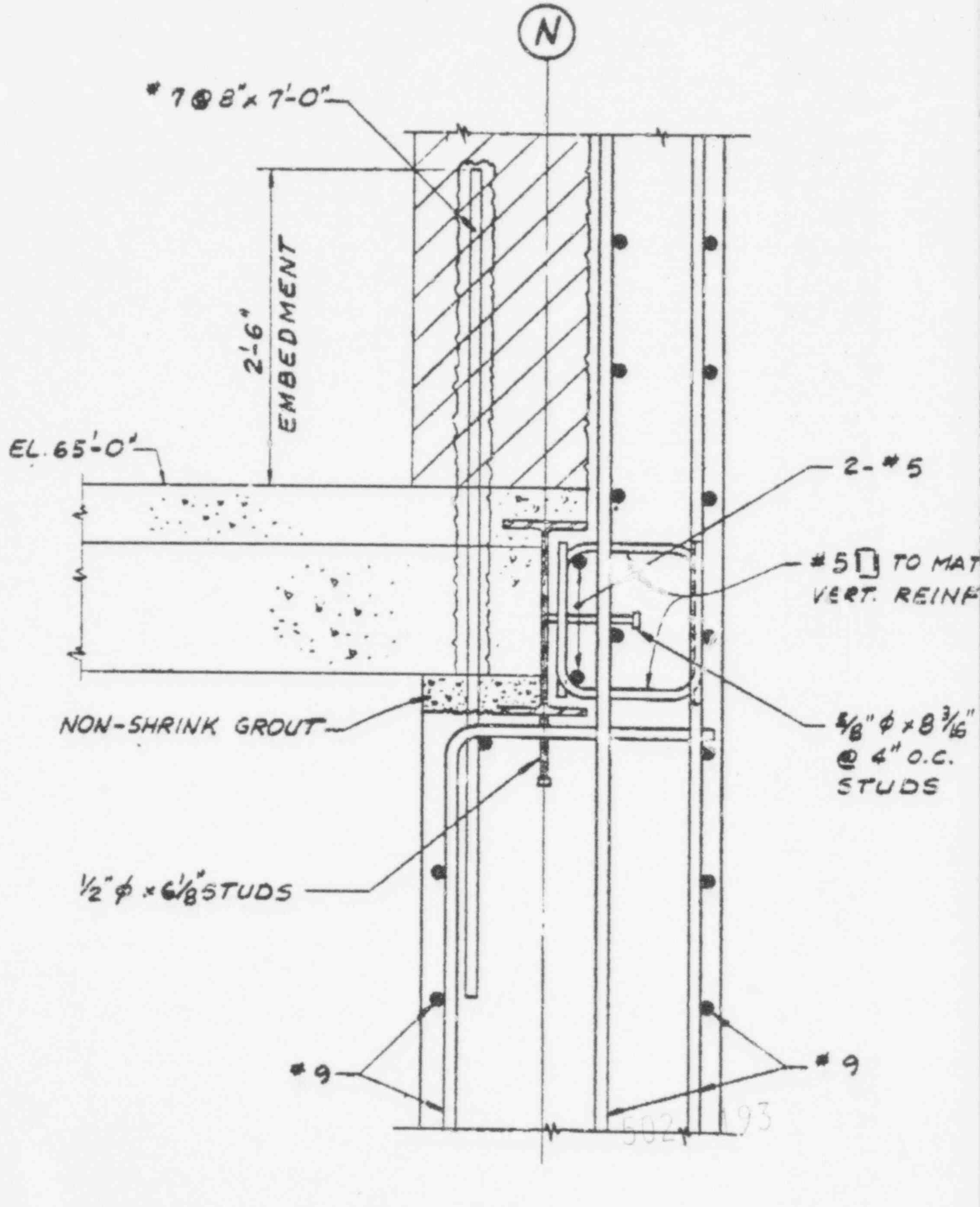
Figure 3-11



502 192

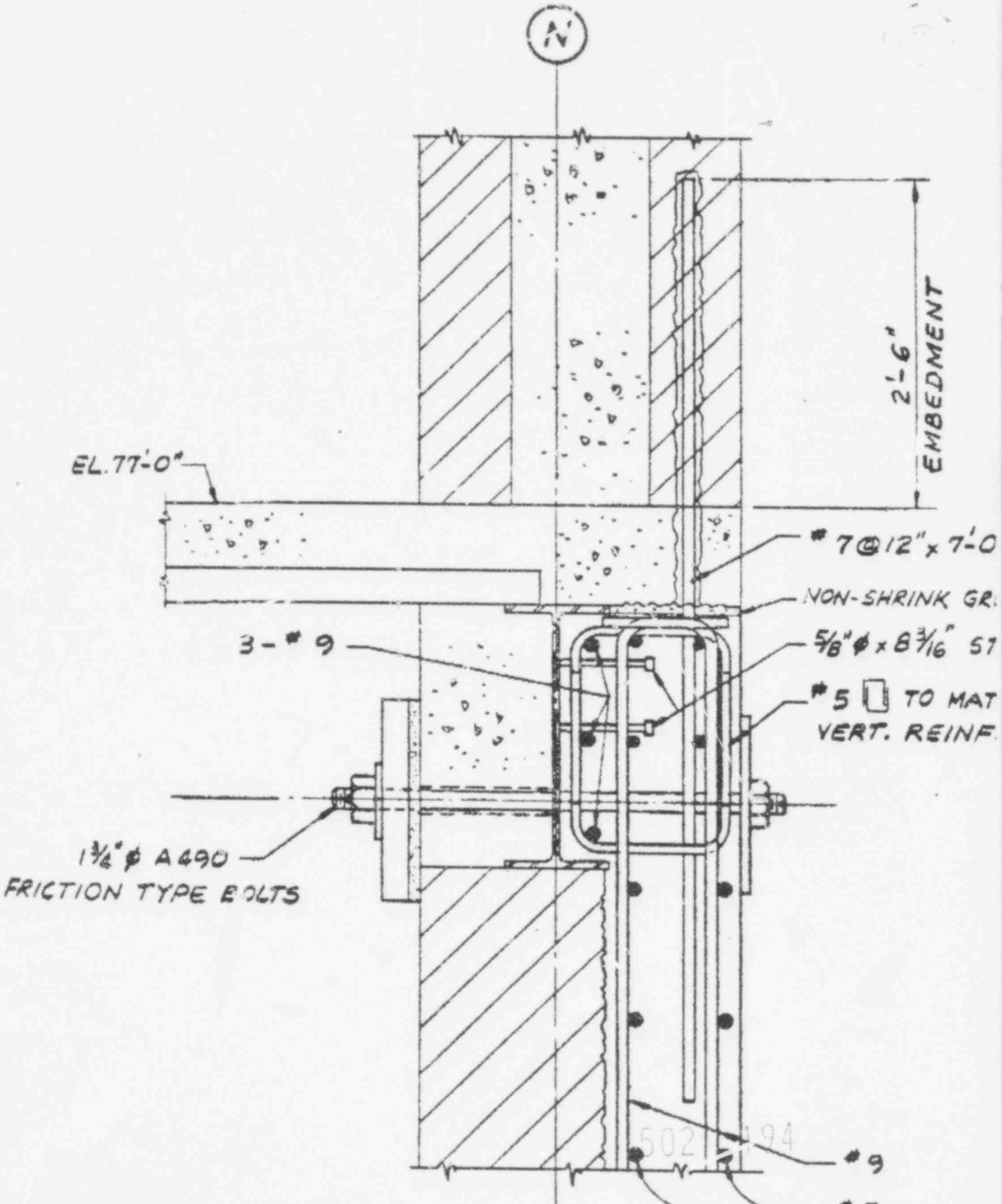
DETAIL AT COLUMN LINES N AND 46
BETWEEN EL 65'-0" AND EL 77'-0"

Figure 3-12



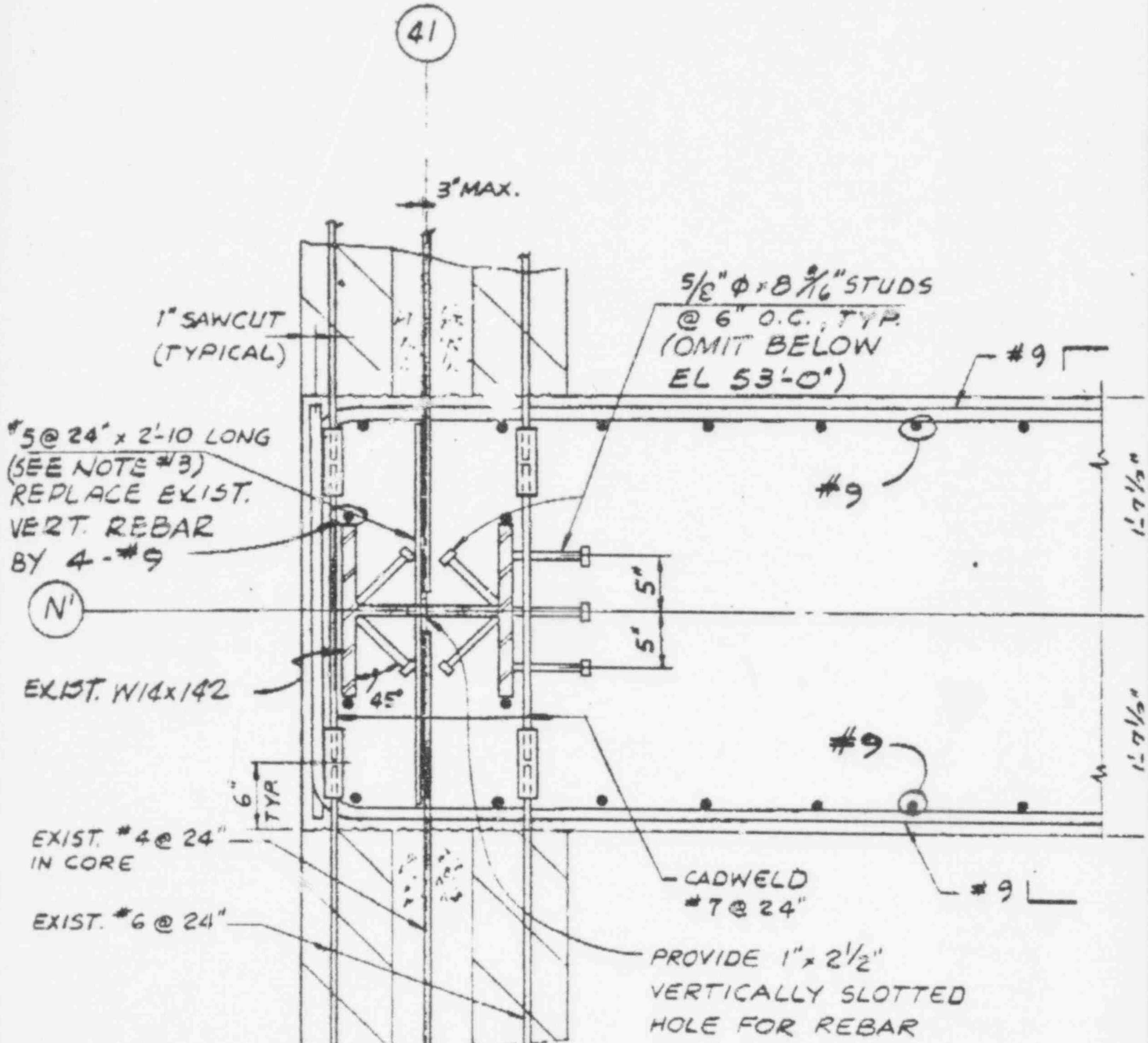
SECTION AT LINE N EL.65'-0"

Figure 3-13



SECTION AT LINE N EL. 77'-0"

Figure 3-14

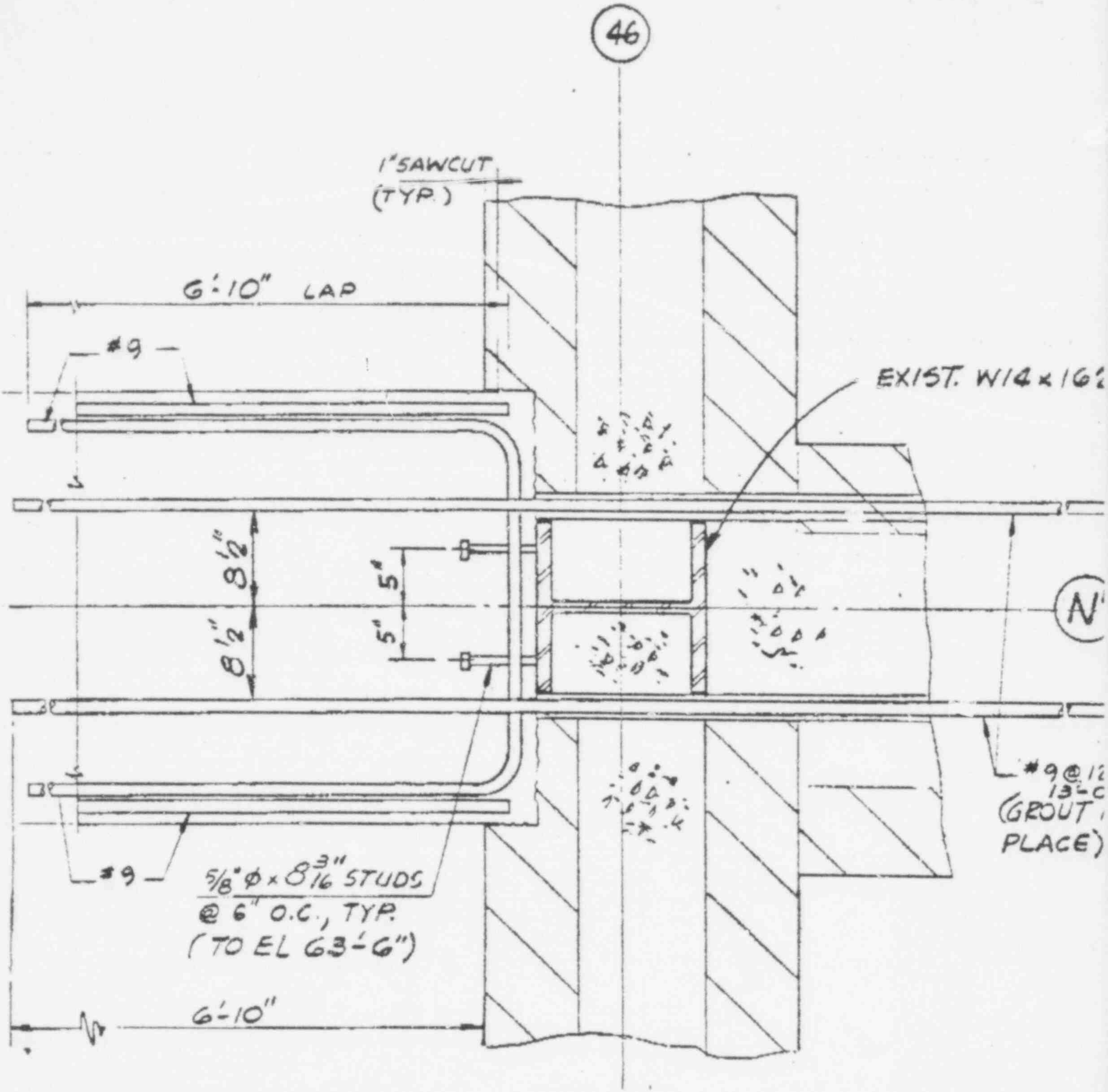


502 195

DETAIL AT COLUMN LINES N' AND 41

BETWEEN EL 45'-0" AND EL 65'-0"

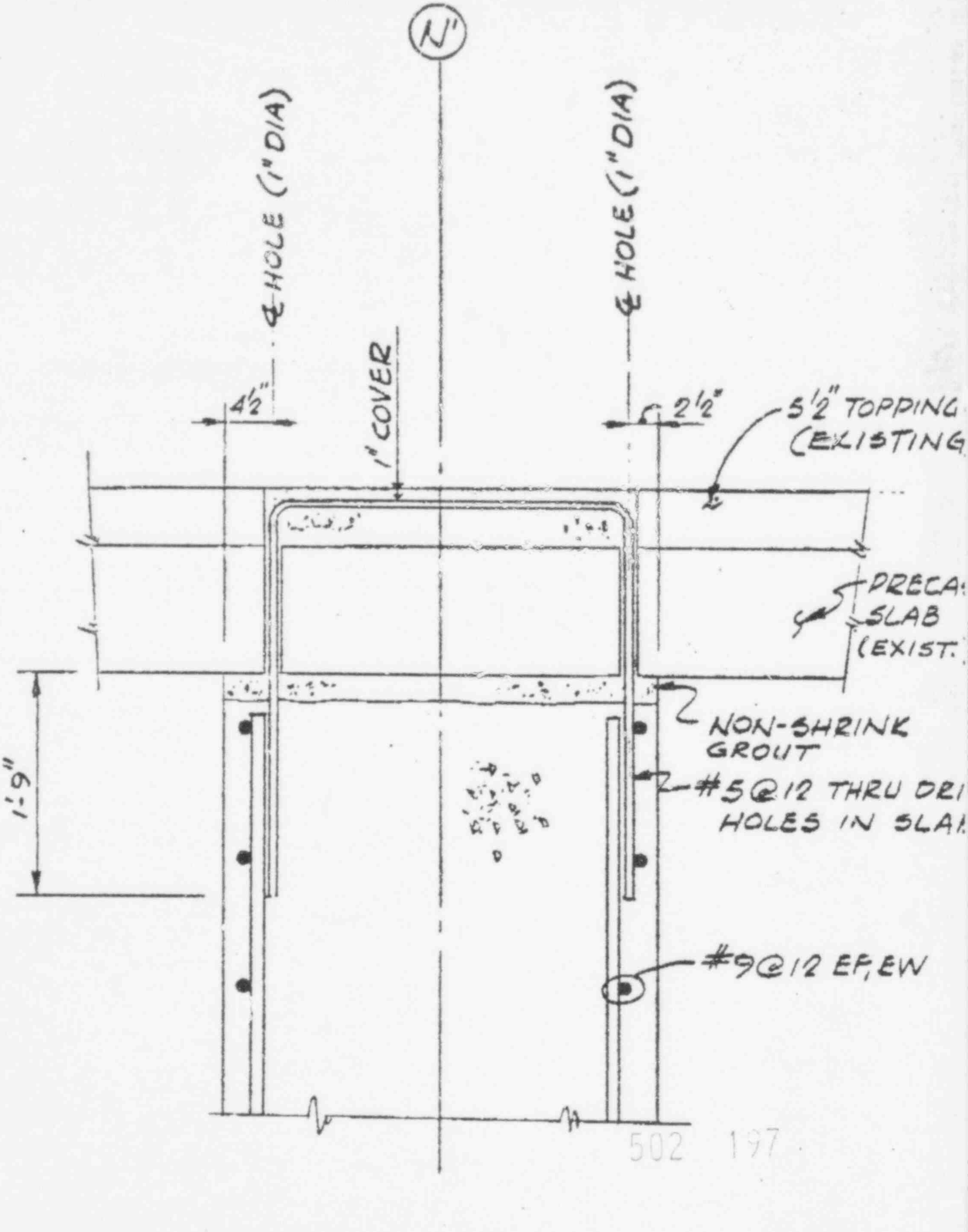
Figure 3-15



502 196

DETAIL AT COLUMN LINES N' AND 46
BETWEEN EL 45'-0" AND EL 65'-0"

Figure 3-16



502 197

Figure 3-17

Provide the shear capacities of the column connections vs. the required shear resistance under the combined loadings to support your claim in Section 3.4.2.2 that the derived flexural capacities of the Trojan walls are conservative in that the building walls will not slide.

Answer:

Section 8 of the report "Trojan Control Building Supplemental Structural Evaluation", September 19, 1978, describes the mechanism by which the shear forces carried by the walls of the Complex are transferred to the rock foundation. Resistance to sliding is provided by friction at the grade beams to rock interface as well as friction between the steel columns and the concrete spread footings. Table 8-1 of the above-referenced report lists the sliding resistances and the base shear forces and the factor of safety against sliding for each major wall of the unmodified structure. The modifications will not significantly change the total base shear forces. Furthermore, the new walls will provide additional sliding resistance and hence factors of safety will be further increased.

Calculations are also made to obtain the sliding resistance as provided by the steel columns and the shear friction developed by the continuous vertical reinforcing steel crossing the wall-slab interface at each floor level together with the dead load. These two individual resistance mechanisms are calculated as follows:

1. Column Resistance

The shear resistance provided by steel columns is given by:

$V_1 = A_c f_v$

where:

A_c = cross-sectional area of steel columns (inches^2); typically one end column and 50% area of other end column is neglected.

f_v = yield shear stress (ksi)

The value of f_v is taken as $(1/3)^{0.5} f_y$ where for f_y= 36 ksi,

f_v = 20.78 ksi.

The shear resistance of a steel column is checked against the bearing in core concrete. Concrete bearing stress is assumed to vary linearly from 0 to $0.85 f_c'$ over a height equal to twice the depth of the column. The lesser of the column resistance and concrete bearing governs.

2. Shear Friction by Vertical Reinforcing Steel

Shear friction at the wall-slab interface as provided by the mechanism of shear friction is taken as:

$V_2 = \mu (A_s f_y + N)$

where:

μ = apparent coefficient of friction = 1.4

(See response to Question No. 16)

A_s = area of vertical reinforcing steel (inches^2)

f_y = yield stress of rebar = 40 ksi

N = direct dead load on wall reduced for the effect of vertical earthquake (kips)

The ultimate shear resistance against sliding at wall-slab interface is

$$V = V_1 + V_2$$

For the unfactored OBE condition the sliding resistance is obtained by multiplying V by the capacity reduction factor of 0.85 and dividing by the load factor of 1.4.

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Analysis of Test Result

The above criteria for sliding resistance is applied to the data obtained from testing the specimen L2, described in Appendix A of PGE-1020.

Test parameters:

A_s = 2 x 4 - #4 bars = 1.60 in^2

σ_n = 31.4 psi ; N = 31.4 x 17.25 x 80 = 43.33 kips

A_c = 2 nos. W 10 x 25 columns

= 2 x 7.36 = 14.72 in^2

f_y = 51.8 ksi (See table A3-3 of Appendix A, PGE-1020)

V_1 = 14.72 x 20.78

= 305.9 kips

V_2 = 1.4(1.60 x 51.8 + 43.33)

= 176.7 kips

$$V = V_1 + V_2$$

$$= 482.6 \text{ kips}$$

$$\text{Shear resistance} = \frac{482.6 \times 1000}{17.25 \times 80}$$

$$= 350 \text{ psi}$$

The specimen did not fail in sliding. The failure shear stress was 367 psi which shows that the analytically obtained results provide for a realistic assessment of the resistance against sliding.

Table 11-1 shows the calculated resistances and also the OBE shear forces at various floor levels in the west wall along column line R of the Control building. The factor of safety against sliding is also presented. Similar results are obtained for other walls of the Complex.

The results of the analysis, therefore, confirm that an adequate amount of sliding resistance exists both at the foundation level and also at all wall-slab interfaces so that the walls will develop their flexural capacities as described in Section 3.4.2.2 of PGE-1020.

SLIDING RESISTANCE AND SHEAR FORCES IN KIPS
WALL ALONG COLUMN LINE R

Elevation	Resistance against sliding-ultimate			OBE Resistance	Shear Forces *	Factor of Safety
	Shear Friction	Steel Columns	Total = V	$V_1 = \frac{0.85V}{1.4}$	OBE = 0.15g; β= 2%	
45'	6480	2730	9210	5590	2140	2.61
61'	4050	2730	6780	4120	2650	1.55
77'	3840	2080	5920	3590	2480	1.45
93'	4390	2080	6470	3930	2260	1.74

*Results of latest STARDYNE run - to be included in the forthcoming revision of PGE-1020

NOTE: The table above does not include elevation 117' since sliding does not occur at that level; the shear transfer mechanism between the slab and the wall, however, has been investigated and found to be adequate.

TABLE 11-1

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Additionally, for all walls discuss the causes of (e.g. shrinkage) and the effects of the observed separation between the bottom of the steel beams and the concrete along the west wall of the Control Building and limitations on the rotational restraint of the in-situ wall on the appropriateness of using the double curvature specimen test results.

Answer:

A detailed survey of the shear walls of the Control Building has been made to determine the locations and extent of any separation between the steel beams and the composite walls. The only place where such a separation was observed is on the west wall along column line R between column lines 41 and 46 and below the bottom flange of the steel beam supporting the floor slab at elevation 77 ft. The wall at this location is composed of two wythes of grouted reinforced masonry blocks without any core concrete. At other locations in the west wall and in all the other major walls of the Control Building at least a portion of the masonry, and in some instances the core concrete, continues beyond the floor steel beams. But in this particular location of the west wall the outside wythe stops just below the steel beam flange. Concrete was poured from the inside face of the wall to fill in the space between the top of the inside wythe of masonry and the bottom of the floor slab. It was expected that the concrete would flow to the outside face of the wall thus filling in the interspace between the bottom of the beam and top of the outwythe. This did not happen throughout the length of the panel and some amount of gap, especially in the center portion of the panel, remained open. Physical examination shows that that the gap extends to a depth of about 8 inches, which is the thickness of the outside wythe. There is no visual crack or gap on the inside face of the wall at the beam location. It is therefore concluded that the unique geometry and construction adopted to build this portion of the wall is the reason for the separation between the bottom flange of the beam and top of the outside wythe. Shrinkage is thus not the cause of the separation, otherwise, not only the inside face of this particular portion of the west wall but the other walls would have exhibited separations of similar nature.

Limitations on the rotational restraint of the in-situ wall on the appropriateness of using the double curvature specimen test results are discussed in Question 43.

Q. 11. (c)

Significant separation of the concrete away from the beams or tension induced in the walls where there is no separation could impact the consideration of the "box effect" or confinement as suggested by PGE-1020 thereby reducing the shear capacity assumed for the wall. Quantify the extent of and effects of this unbonded condition for all walls.

Answer:

The separation between the beam flanges and the wall panels is addressed in response to Question No. 11 (b). The "box effect," as it exists in the Complex, enables the side walls to act as webs while the cross walls participate in providing the flange action when the Complex is subjected to an overturning moment due to lateral load. The "box effect" is realized when the mechanism exists to transfer the vertical shear forces from the side to the cross walls at their common interfaces. This capability of shear transfer has been analyzed and found to be adequate as explained in response to Question No. 16.

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Q. 11. (d)

Also, in addition to considering the concrete strength of 5000 psi, discuss the effects of the interfaces with 3000 psi design strength concrete.

Answer:

The existing shear wall core concrete and the concrete block wall cell fill grout are 5,000 psi design mixes. In many areas, the concrete slabs are a 3,000 psi design mix. The mechanism of shear transfer at the floor levels is described in response to Question No.11 (a). The portion of the shear force which is transferred at the wall - slab interface is obtained from the shear-friction of the fully embedded vertical reinforcing steel and the direct dead load stress. Because the "coefficient of friction" along the joint is independent of the concrete strength, so also is the shearing strength, provided the shearing stresses do not exceed some limiting value. This limiting shearing stress, as suggested by Mattock, Johal and Chow in "Shear Transfer in Reinforced Concrete with Moment or Tension Acting Across the Shear Plane," PCI Journal/July-August 1975, can be taken as $0.2f_c'$, which for 3,000 psi slab concrete is 600 psi. The ultimate shearing stress used in the analysis of the Complex shear walls is well below this limit.

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Discuss in detail why the dead load acting for the SSE is greater than that acting for the OBE, thereby resulting in greater shear capacities for the SSE than considered for the OBE.

Answer:

Due to the construction sec of the Complex, the dead load carried by the steel frame under static conditions consists of its own weight and the reinforced concrete floor slabs. The walls carry their own weight. During an earthquake event, however, when the structure undergoes any lateral deformation, the axially stiff steel column will tend to deform less than the adjacent wall panel in the vertical direction. The displacement compatibility between the encased steel frame and the concrete walls causes redistribution of axial loads in these elements. The tension side of the wall will pick up additional vertical load thus unloading the precompressed column.

The actual vibratory motion during the SSE will be more severe than the OBE. The lateral deformation for the SSE is greater than for the OBE. Therefore the precompressed columns will be unloaded more during the SSE than the

OBE, thus resulting in a greater increase in the SSE dead load on the wall panel. For the purpose of determining the capacities of the existing walls of the modified Complex, however, the increase in the dead load has been conservatively neglected for the OBE. As indicated in PGE-1020, the OBE controls the design of the modified Complex and only the direct dead load is used for capacity determination.

Q. 19. (a) Page 1 of 2 pages

Provide the basis for your claim that, in lieu of the test program results, there are no UBC requirements addressing the type of walls in the Trojan Complex since Sec. 2417 of UBC-1963 specifies that for combinations of units, materials, or mortars, the maximum stress shall not exceed that permitted for the weakest of these.

Answer:

The major shear walls of the Complex are constructed of high strength concrete core, both reinforced and unreinforced, (nonmasonry units) sandwiched between two wythes of reinforced grouted concrete blocks (masonry units). The Uniform Building Code, Chapter 24, is devoted solely to masonry construction that employs the units, materials and mortars specified in Section 2403. Section 2403 does not include the concrete core which is covered in a different chapter of the UBC. Section 2417(a) places allowable limits on design and construction that uses a combination of the masonry units, materials and mortars specified in Section 2403. However, when the combination includes a non-masonry element such as the concrete core in the Complex walls then the allowable stresses of Section 2417 (applicable solely to masonry) no longer apply. Thus the the major shear walls of the Complex are not addressed by Section 2417.

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Since the FSAR did not specifically address the composite shear wall construction of the type used in the Complex, it is understandable that ambiguities could have arisen as to the intent of the reference in the FSAR as to the UBC. Section 3.8.1.5. indicates that "concrete block walls" in Category I structures are designed to the UBC requirements for masonry; and such requirements were in fact observed for those walls constructed solely of masonry. However, there was no intent to apply those requirements to the composite masonry-concrete wall construction of the type used in the Complex; and, as discussed above, those requirements would not be applicable to such construction.

Provide the basis for your statement that the UBC did not envision the use of a model such as STARDYNE, therefore, higher allowables are appropriate. UBC Section 2417 merely states that forces be determined from the principles of continuity and relative rigidity, which is what STARDYNE does.

Answer:

The Trojan FSAR went beyond the minimal requirements of the UBC by calling for response spectrum analysis to determine the dynamic loads for Category I structures. As discussed in Section 3.6.3 of PGE-1020, the original evaluation of the Complex was done by performing a response spectrum analysis on beam-stick mathematical model which, was an adequate approximate representation of the physical structure. In the reevaluation study, the detailed three dimensional finite element modeling of the Complex was a more accurate representation of the structural system, and therefore the STARDYNE response spectrum analysis on this model more accurately determined the dynamic response of the Complex.

The Uniform Building Code, along with several other codes, while maintaining that the forces in the structural elements be determined from the principles of continuity and relative rigidity, does not specifically call for applying techniques as sophisticated as an extensive finite element analysis. A simpler static analysis based on relative rigidities of the participating structural elements is adequate to satisfy the code requirement. However the STARDYNE analysis, which is a rigorous finite element analysis, takes into consideration not only the rigidities of the structural elements for combinations of their deformation modes, but also provides a tool for evaluating structural discontinuities and their effect on the system behavior. This kind of analysis, therefore, provides far better knowledge and consequently a higher level of confidence by eliminating analytical uncertainties that may be present in a relatively simpler analysis.

PGE-1020 did not state that higher allowables are appropriate in light of STARDYNE. Section 3.6.3 indicated that, on the basis of the fact that such an improved analysis was performed and the better understanding it provided of the Complex, the need for design margin was reduced. However, in PGE-1020 credit is not taken for this additional conservatism.

Q. 25.

In Section 4.2.3, reference is made to the tensioning of bolts after concrete has attained "adequate strength." Define "adequate strength" and describe how it will be determined.

Answer:

Adequate strength is defined in this context as the design strength of the new concrete. Determination of when the concrete has attained this strength will be made with cylinder tests conducted in accordance with ACI standards. The bolts attaching the plate to the wall will not be fully tensioned until development of the design strength has been demonstrated. Prior to this time, the bolts will be made snug to remove any play between the plate and the wall.

Q. 26.

Verify that the static and dynamic effects of the rigging and the steel plate on the Turbine Building above elevation 93 feet have been considered.

Answer:

The structural elements that will be affected by rigging the steel plate on the Turbine Building above elevation 93' are the floor steel beams at elevation 93' and the crane girder at elevation 130'-11" to which the chain hoists will be attached for handling the plate.

An analysis of these elements has shown that the static and dynamic loads that will be imposed on them during the handling of the steel plate result in stresses below AISC code allowed values. The eccentric loading on the crane girder has been considered in the investigation.

502 215

Q. 27.

What strength concrete was used to model the new walls in the STARDYNE Analysis of the modified complex? In Section 3.2.5 a concrete strength of f'_c = 5000 psi at 90 days is specified for the new walls. Will the qualification of the modified complex be affected while this strength is being developed after concrete placement considering both in plane and out of plane wall loadings? Provide the basis for your response.

Answer:

The capacity of the walls is based on a design strength of 3500 psi. The STARDYNE analysis used a stiffness based on a concrete strength of 5000 psi which is the expected long-term capacity of the new concrete. The structural qualification of the modified Complex, as affected by the capacity of the new walls as their strength increases from zero to the full design values, is discussed in the response to Question No. 31.

502 216

Provide your evaluations of the effects of the proximity or configuration of hole patterns, including the effects of any cracking which is present in the walls.

Answer:

The capacity of the walls is controlled by flexure which is dependent upon the vertical reinforcing bars. Since none of the reinforcing steel will be cut, the flexural capacity of the panels will not be reduced. Except for the separation discussed in the response to Question No. 11(b), only hairline cracks are present in the walls where holes will be drilled. For the same reason discussed above such hairline cracks will not affect the panel capacity.

In resisting either the horizontal or vertical shear forces along a line of bolt holes, there are three important factors to be considered. First, the 3" diameter holes are spaced a minimum of 8 diameters which results in a small amount of material being removed. If a bar is encountered while drilling, the hole will be abandoned and fully grouted before the replacement hole is drilled. Since the reduction in shear area owing to any such abandoned holes would be insignificant, the replacement hole may be drilled even if the grout in the abandoned hole has not yet developed its designed strength. Second, the resistance being relied upon is produced by the reinforcing steel and the encased column or the beam-column connection, none of which is sensitive to the small amount of concrete and block being removed. Third, the reduction in area due to the bolt holes is less than 4% in the horizontal shear plane, less than 6% in the vertical plane and less than 5% in any diagonal plane. These reductions in shear areas have been considered in evaluating the shear capacities of existing walls. Along these planes, the row of bolt holes does not traverse the entire structure and any tendency for a crack to develop along the bolt holes will be resisted by portions of the wall without holes.

After the new structural elements are bolted into place, they will bridge across the holes.

Q. 32.

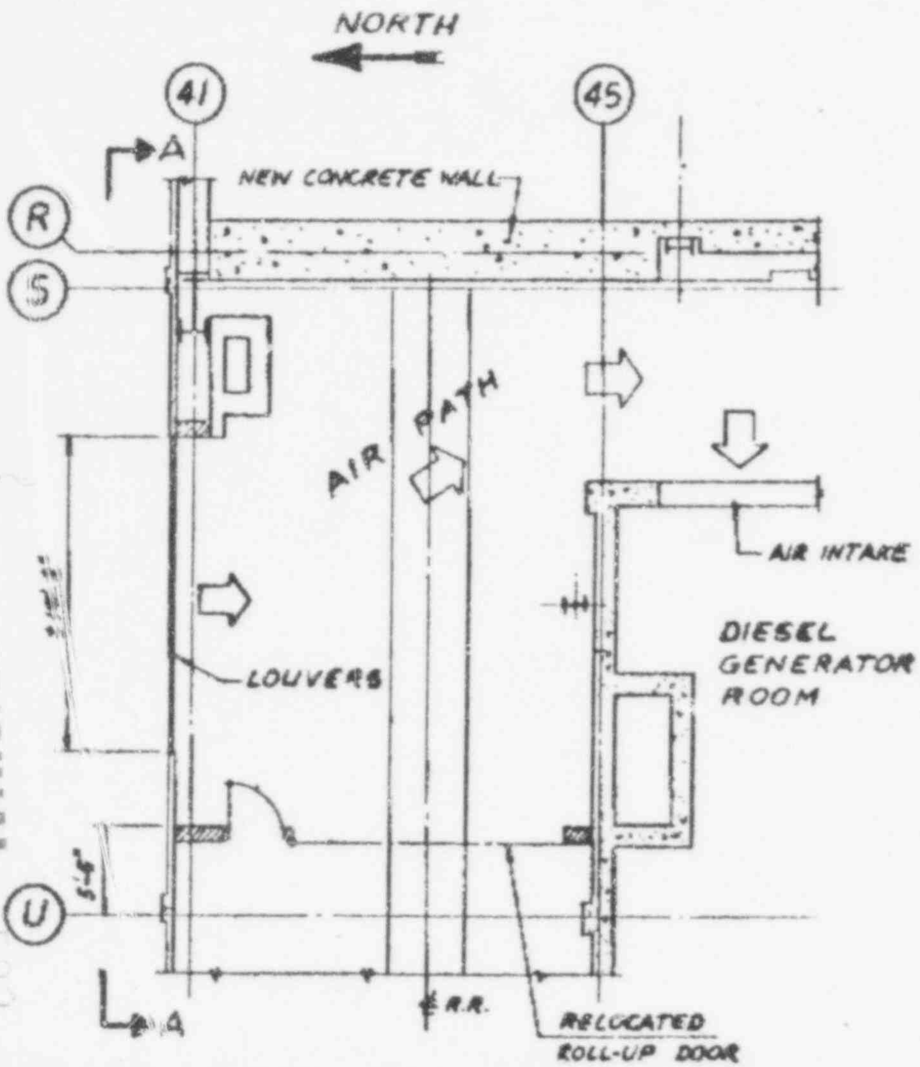
Summarize the loads and load combinations and corresponding acceptance criteria for which the diesel generator air intake will be designed. Include a discussion of how the effects of the Turbine Building, a non-Category I structure has been considered.

Answer:

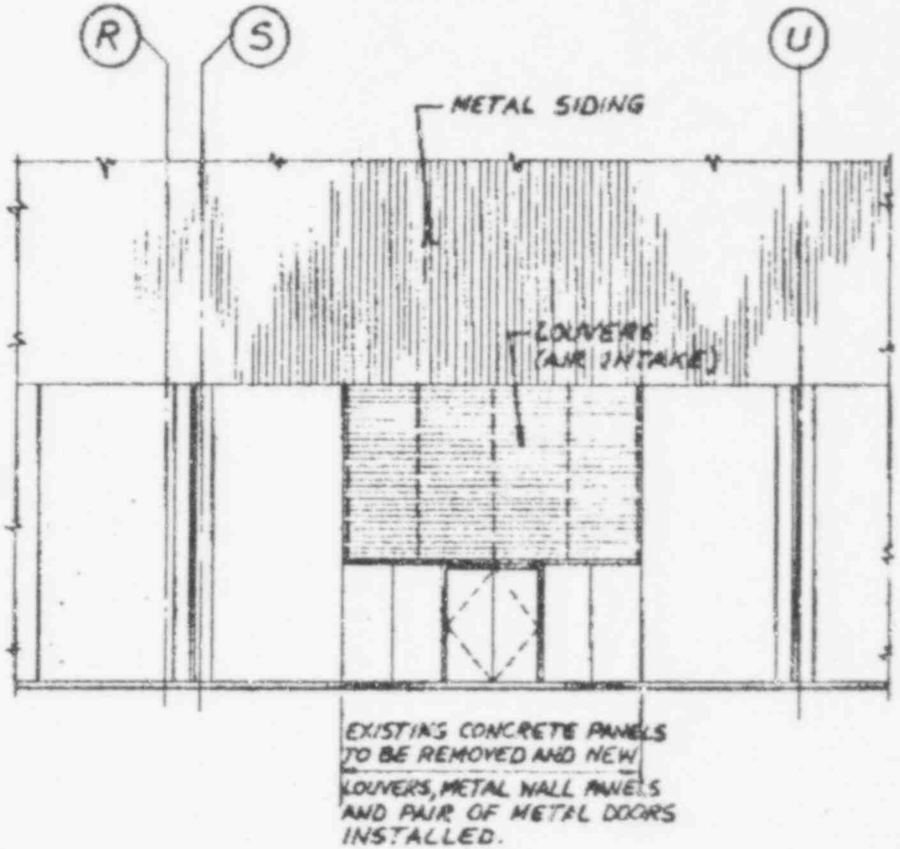
The new diesel generator air intake through the north wall of the Turbine Building consists of a louvered opening in the wall. The purpose of the louver is to keep wind, rain, and debris from entering the Turbine Building. The louvers need not be designed for abnormal loads since their collapse would not preclude air supply to the air intake located on the East side of the Diesel Generator Room. In any event, the air intake on the north wall of the Turbine Building has been sized to allow blockage of 50% of the area. The attached figure shows the location of the louver and the air supply path to the Diesel Generator Room.

The fact that the Turbine Building is not a Category I structure has no effect on supplying air to the Diesel Generator Room because it has been designed to resist a Safe Shutdown Earthquake (FSAR Sec. 3.8.1.1.6). The siding and the connections have been analyzed and it has been determined that the siding will not become detached during a Safe Shutdown Earthquake.

502 219



PARTIAL PLAN SHOWING
AIR SUPPLY PATH TO DIESEL GENERATOR ROOM



VIEW A-A

(ATTACHMENT TO RESPONSE No 32)

Q. 33.

Provide the basis for your determination that removal of the face masonry block and a portion of the concrete core at columns lines 41 and 46 on column line N' will not significantly affect the shear capacity of these walls.

Answer:

The sketches provided in response to Question No. 3 show the portions of the existing shear walls which will be removed during modification work and filled in later to make necessary connections to the new concrete walls. All of these existing walls, including those at column lines 41 and 46 and on column line N' have been evaluated to assess their shear capacities during such modification work. The shear capacities have been calculated both on the basis of the criteria established in the report "Trojan Control Building Supplemental Structural Evaluation September 19, 1978" and also Section 3.4.2.2 of PGE-1020. It has been found that the walls with the portions removed will be capable to withstand an SSE level greater than 0.25 g, and also an OBE greater than 0.08g. This demonstrates that the shear capacities of the walls will not be significantly affected during performance of the modifications.

502 221

Q. 34.

Provide the capacity assumed for the dowels used to perform the wall modifications and the basis for this assumed capacity.

Answer:

The capacity of the dowels used to perform the wall modifications is calculated in accordance with the requirements of ACI-318-77 and it is basically a function of the capacity reduction factor, yield strength of reinforcing steel, and the amount of steel present, modified as required for the type of application (e.g., tension or shear). The design of the connections between the new walls and the existing walls and slabs is discussed further in response to Question No. 3.

502 222

Define "repres[illegible]tative" as used in defining the struts used in specimens E1, F2 and H2. Include a discussion of the similarity between the way in which the struts were anchored into the bulkheads, thus encasing the wall vs. the way the walls are encased in the frame formed by the columns and beams in the actual structure. Expand this to include a similar discussion for specimens L1 and L2. Also, discuss the similarities between the horizontal steel anchorage at the edges of the test specimens vs. that of the actual walls interrupted by openings, and those which intersect cross walls (e.g. the wall intersection at the intersection of column lines R and 55.)

Answer:

The specimens E1, F2 and H2 had two steel struts each, located externally on either side as shown in Figure A3-1 of PGE-1020. The struts were attached to the top and bottom beams through hinged connections with 3/4" A-325 bolts. The steel struts were used to simulate the axial resistance behavior of the steel columns in the Complex walls where the columns would be assumed to act as external members without any vertical shear transfer at the column-wall interface. The struts also provided a deformation-controlled rotational resistance (as opposed to a force-controlled rotational resistance) at the ends of the specimens. The external steel struts increased the shear capacity of wall specimens by inducing additional dead load.

502 223

The specimens L1 and L2 had the two steel columns fully embedded in the core concrete as shown in Figure A3-2 of PGE-1020. These columns were anchored to the top and bottom beams by an embedment length of about 3'-0". These steel columns in the test specimens simulated the embedded steel columns which are continuous through adjacent floors of the Complex.

The area of the steel column in the test specimen was dimensionally reduced to simulate an average column size in the Complex. The ratio of the column to the wall cross section was approximately the same between the test specimens and a typical wall panel.

It should be noted that the specimens E1, F2 and H2 with steel struts and the specimens L1 and L2 with embedded columns were tested to investigate the behaviour of Complex walls with embedded steel frames. As explained, two extreme conditions for bond were simulated since the exact conditions of Complex walls are difficult to create in test specimens.

As shown in Figure A3-2 of PGE-1020, in order to simulate interrupted reinforcement in the actual walls, the horizontal reinforcing bars of the test specimens were not anchored at their ends. Only the horizontal reinforcing bars in the masonry blocks of L1 and L2 specimens had U ties simulating the continuity of block reinforcement in the actual walls. Also, it can be seen from the test results that the

502 224

horizontal reinforcement is not an important parameter for the shear capacity of specimens unless the specimens failed in the classical shear mode. The horizontal reinforcement helped to control the width of major diagonal cracks in specimens which had a shear mode of failure.

Discuss in detail the error band associated with each of the test results (e.g., stiffnesses, strengths, degradation, etc.). Explain and justify how these were factored into your evaluation of the complex.

Answer:

Tables A1-1 and A1-2 of PGE-1020 list the test program and the specimen description respectively. As can be seen, the test parameters were not duplicated, therefore restricting a direct assessment of error associated with the test results. However, the following discussion provides the basis to estimate conservatively the probable error associated with the test results and the procedure to account for such error in the evaluation of the Complex.

The probable error from the quality of materials such as mortar, masonry blocks, grout, concrete and reinforcing steel and from the fabrication of test specimens was minimized by implementing a good quality control program. The effect of such an error is negligible because the behavior and capacity of specimens were predominantly governed by the most reliable parameter, namely, the vertical steel reinforcement. Therefore, the probable error from the quality of materials and from the fabrication of test specimens can be reasonably estimated to be $\pm$ 1%.

The specimens were loaded using calibrated hydraulic rams and pressure gauges. The performance of hydraulic pumps was continually monitored to assure the steady application of intended load on the specimens. The deformation of specimens under load was measured using dial gauges, linear variable differential transducers (LVDT) and X-Y recorders. The dial gauges and LVDTs can indicate deformations up to an accuracy of 0.0001" and 0.0005", respectively.There were duplicate pressure gauges, dial gauges and LVDTs to measure and monitor the important quantities such as ram pressure (load) and lateral deformation. There were at least two technicians to read and record gauge readings. Also, at least two test engineers were engaged to check the test set-up and measurements. Thus, adequate precautions were taken to minimize the probable error from test set-up and measurement. A ± 3% would be a conservative estimate for such an error.

The accuracy and consistency in the test results are demonstrated in Figure 41-1 by the small scatter among the ultimate strength of all the 23 specimens. This type of experimental scatter is common among the results of concrete test specimens. Such a scatter can be attributed to various sources as discussed above and other probable sources such as the construction joint at the beam-specimen interface. The consideration of error band associated with the test results is not applicable to the capacity evaluation because the test results were not used directly, as explained in Section 3.4 of PGE-1020.

502 227

As explained in Appendix B of PGE-1020, the test results were used to evaluate the stiffness of the Complex walls as a function of axial stress, shear stress and vertical reinforcement ratio. During this evaluation, the uncertainties associated with the test results are considered conservatively in response spectra broadening, as explained in response to Question No. 47.

502 228

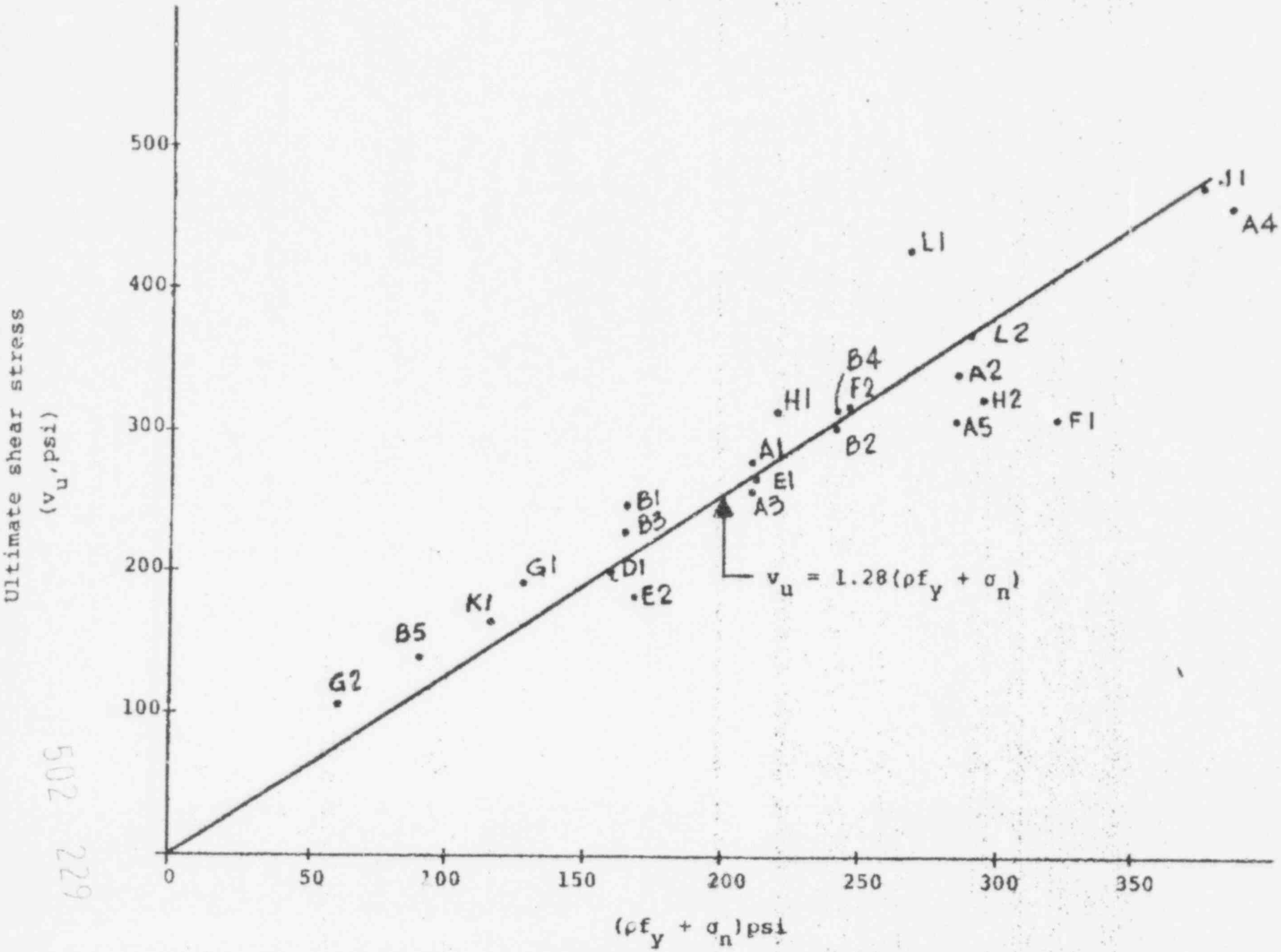


FIGURE 41-1 ULTIMATE STRENGTH OF SHEAR WALL SPECIMENS

Compare the slopes of the sides of the peaks in floor response spectra for the complex frequency shift vs. stress (therefore, ground acceleration) level as derived from the test data results to verify that the floor response spectra are conservative for all earthquake levels for both the OBE and the SSE spectra. Justify any non-conservative deviations.

Answer:

In considering the adequacy of the OBE response spectra, a review of stiffness reduction factors was made to determine the largest earthquake that could occur before significant reduction in frequency resulted. It was estimated that for an earthquake of approximately 0.10g the stiffness reduction would be approximately 2% resulting in a 1% shift in frequency. The response spectra associated with the 0.10g earthquake was estimated by assuming the same general shape as for the 0.15g OBE except the frequencies associated with the peaks are increased and the ordinates are reduced by the ratio of 0.10/0.15 = 0.67. The estimate of the increase in frequency was made by considering several of the STARDYNE analyses involved in the overall stiffness iteration process. This resulted in an increase of 5 to 7 percent. This is the shift of the first mode in the N-S direction, and the shift associated with the other modes is less. Since the peaks of the response spectra have sloping sides, the width of the peaks increases with decreasing acceleration. As shown in Figure 49-1, the width of the peaks of the response spectra for an 0.15g OBE varies between 15 and 20 percent at 0.67 of the peak ordinate. This will allow the frequency shift of 5 to 7 percent, plus approximately 10 percent curve broadening, with the resulting spectra for an 0.10g earthquake still being within the 0.15g OBE response spectra. As the earthquake level increases from 0.10g to 0.15g, the stiffness reduction factors decrease gradually which will result in a gradual transition to the 0.15g response spectra.

In the event of an earthquake greater than the OBE, there is expected to be a gradual transition from the 0.15g OBE response spectra to the 0.25g SSE response spectra. Designing safety-related components, equipment and piping to the OBE and SSE criteria provides a high level of confidence of being able to withstand an earthquake between 0.15g and 0.25g.

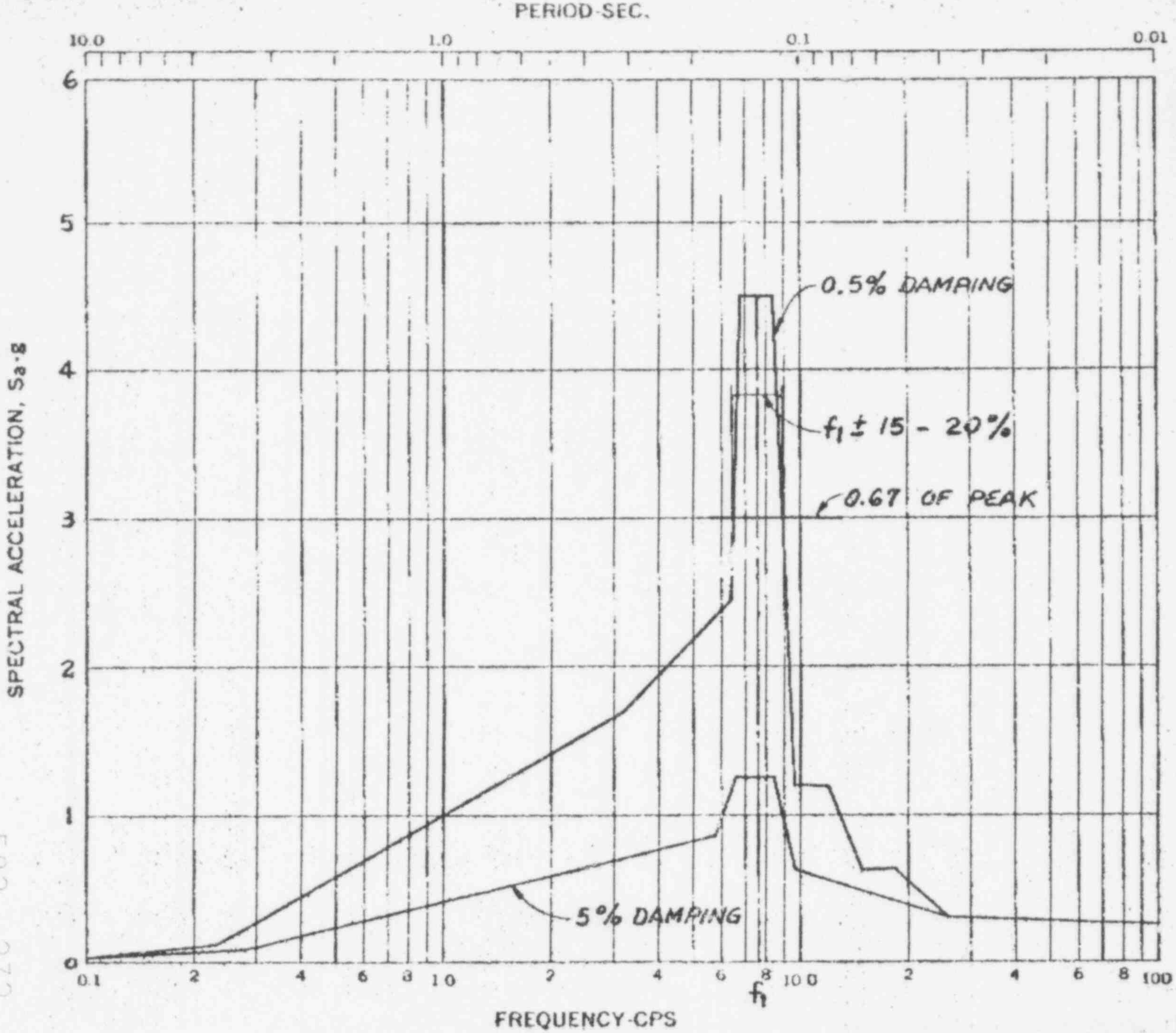


Figure 49-1 Representative Floor Response Spectra : 0.15g OBE

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