PGE Portland General Electric Company

July 6, 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 additional responses prepared by Becntel Power Corporation covering most of the outstanding questions from your letter of May 18, 1979. As discussed with your staff, answers to the balance of these questions will be submitted Tuesday, July 10. The scheduled revision of PGE-1020 will follow the responses to the remaining questions by one week as committed in the June 15 meeting with your staff, Bechtel Power Corporation and PGE in San Francisco.

Sincerely,

Ronald W. Johnson

R. W. Johnson
Corporate Attorney
Portland General Electric Company

RWJ/JGL/4sb6A25
Enclosure

478 ~~461~~ 086

(55)

A001
S 1/1

7907110479

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 July 6, 1979, Licensee's letter to the Director of Nuclear Reactor Regulation dated July 6, 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

478
~~[illegible]~~ 087

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

Dr. Harold I. Laursen
1520 N. W. 13th
Corvallis, Oregon 97330

Ronald W. Johnson
Ronald W. Johnson
Corporate Attorney
Portland General Electric Company

Dated: July 6, 1979

4sb66.27B8

~~[illegible]~~ 088
478

Q. 6. (a)

For the "Criteria for Bolts", provide the following:

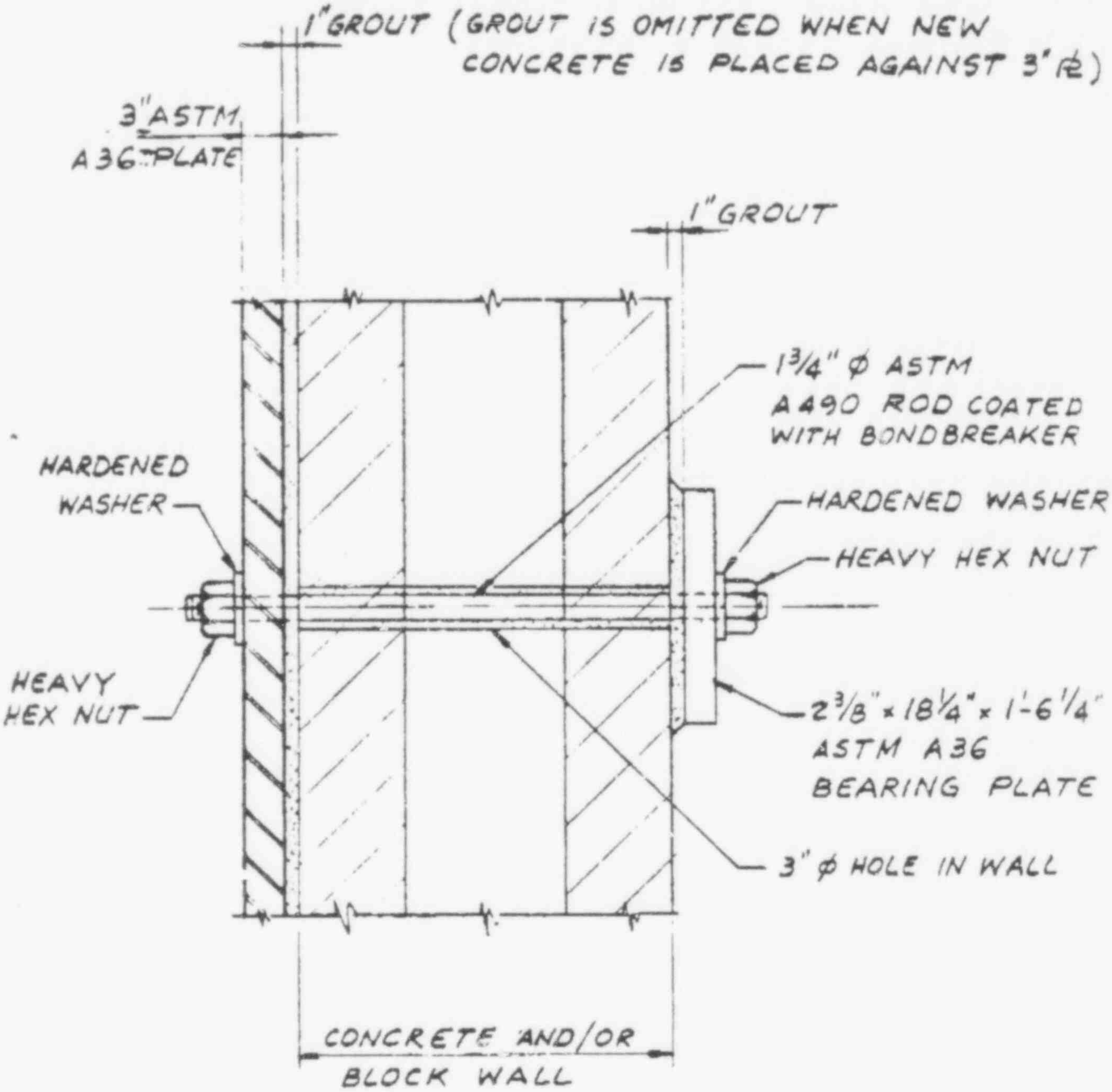
A clear description of the bolt assembly and hardware arrangement.

Answer:

The bolt assembly consists of a 1-3/4" diameter ASTM A490 rod threaded on both ends, a 2-3/8 in. thick, 18-1/4 in. by 18-1/4 in. ASTM A36 bearing plate on the inside of the Control Building, the 3" thick ASTM A36 steel plate on the outside of the Control Building, and a heavy hexagon nut and hardened washer on each end of the rod. One inch of grout will be placed between the 3" steel plate and the existing wall, and between the bearing plate and any concrete or block surface. The bolt assembly and hardware arrangement are shown in the attached figure.

478

089



BOLT ASSEMBLY

SEE RESPONSE No 6(a)

478

FIGURE 6(a)-1

090

For the "Criteria for Bolts", provide the following:

The basis for the formula to calculate the allowable shear force for the bolt including the contact area between the wall and the steel, the stress distribution at the wall/steel interface and the maximum compressive stress induced in the wall at this interface along with justification for the value.

Answer:

The bolted conection of the steel plate to the concrete wall provides a means of transferring forces from one structural element to the other. The transfer of forces is based on the principle that a clamping force between two potential sliding surfaces provides resistance to sliding. The magnitude of the sliding resistance is equal to the clamping force times the coefficient of friction and is expressed by the following formula:

$V = \mu F$

where;

V = resistance to sliding

μ = coefficient of friction

F = clamping force

The clamping force, F, is supplied by the tension in the bolts. The loss factor, (L), is added to take into consideration the losses in the bolt tension. Adding a factor of safety (F.S.) the resistance of one bolt becomes,

$$V = \frac{\mu L F}{F.S.}$$

It is seen from the above relation that the contact area between the wall and the steel is irrelevant to the magnitude of the shear resistance.

The size of the bearing plates was determined by dividing the initial bolt tensile load of 200 kips by the allowable masonry bearing stress of 600 psi which was calculated from Table No. 24-H of UBC 1976. Since the allowable bearing stress for masonry is lower than that for concrete, the masonry controls in the design of the bearing plates.

Using formulas for plates on elastic foundation, the pressures under the 3" plate and the 2-3/8" thick bearing plate have been calculated. For the 3" plate, formulas for a point load on an infinite plate were used. The maximum compressive stress under the 3" plate is 1120 psi. The stress reduces to 600 psi at 6-1/2" and to 150 psi at 12" away from the center of the bolt. The pressure under the 2-3/8 " plate was calculated assuming a point load on a plate with a diameter of 18-1/4". The maximum compressive stress is 1760 psi.

These local stresses are justified because the average stress under the bearing plate will be equal to the code allowable bearing stress and at no point under the plates will the stress exceed the compressive strength of the block.

For the "Criteria for Bolts", provide the following:

The basis for the assumed loss factor.

Answer:

The loss factor, L, is the percentage of tension remaining in the bolts after all losses (shrinkage, creep, bolt relaxation and temperature effects) have been subtracted from the initial tension force. The losses considered and values used to develop a conservative loss factor for the bolts are as follows:

1. Shrinkage:

   The following upper limit values have been used in determining the loss due to shrinkage:

   New concrete, 355×10^{-6}
   Existing block, 200×10^{-6}

   Shrinkage in the existing block occurs where new concrete is placed against it. This is due to swelling caused by water migrating from the new concrete.

2. Creep

   Creep in both the new concrete and existing block is taken as 1.6 times the elastic deformation.

478
094

Creep in the 1" thickness of grout under the base plates and 3 inch plate is taken as 2.0 times the elastic deformation.

The creep losses are conservatively calculated using the maximum compressive stresses given in response to Question No. 6 (b).

3. Bolt relaxation:

The long term bolt relaxation is taken as 2.5% of the initial bolt tension. Initial bolt relaxation will not contribute to the bolt losses because of the two-pass tensioning described below.

The bolts will be tensioned with a stud tensioner in a two pass program. The bolts will be tensioned to the required tension in the first pass. A second pass will be made 24 hours or more after the first pass to compensate for the losses due to initial bolt relaxation, elastic deformation, and the influence of adjacent bolts. The tension in the bolts will be monitored and checked.

4. Losses due to temperature:

Losses in the bolt tension due to a temperature rise of 50° have been included.

478
~~42~~ 005

The values for bolt losses due to creep and shrinkage have been determined by considering the properties of the low shrinkage and creep concrete specifically designed for the Complex modifications. The value for long term bolt relaxation has been obtained from "Guide for Design Criteria for Bolted and Riveted Joints", by J. W. Fisher and J. Struik, 1974.

The design value losses were determined for the three different attachment conditions: 1) bolts attaching the plate to new concrete; 2) bolts attaching the plate to new concrete and existing block; 3) bolts attaching plate to existing block. The maximum decrease in tension for the three conditions is less than 25% of the initial tension. A tabulation of the losses for the three conditions is given in the attached table.

478
~~[illegible]~~ 096

TABLE 6C-1

Attachment Conditions	Creep			Shrinkage		Bolt Relaxation	Temperature Losses	Total
	New Concrete	Existing Block	1" Grout	New Concrete	Existing Block			
Plate to New Concrete	5.8%	-	0.6%	10.7%	-	2.5%	1.7%	21.3%
Plate to New Concrete & Existing Block	3.0%	6.1%	0.9%	4.8%	3.0%	2.5%	1.7%	22.0%
Plate to Existing Block	-	9.5%	1.4%	-	-	2.5%	1.7%	15.1%

BOLT LOSSES

See response No. 6(c)

478

097

Q. 8.

Verify that all resistances and stiffnesses based upon dead load considerations considers the dead load to be reduced by the vertical earthquake component.

Answer:

The shear capacity of a wall panel, as given by the flexural equation described in Section 3.4.2.2 of PGE-1020, is increased by a compressive axial load. The axial load, derived from the dead load, has been decreased to account for the vertical component of the earthquake.

As indicated by the test results, the stiffness of the walls depends on the axial load. During the earthquake, the axial load on the walls oscilates about the mean value which results from the static dead load. In determining the mean frequency, the mean value of the axial load is derived from the direct dead load and the influence of the deviation from the mean value is discussed in response to question 47.

478
~~46~~ 098

Justify the ductility limit of 4 for the outer rebar in the flexural calculations. Also, considering displacement compatibility for the entire structure using the stiffnesses indicated by the test results, what are the strains predicted in the outer rebar. Justify their acceptability in light of your assumptions. Additionally, for the flexural analysis equations justify the use of a compression zone length of 10% of the total effective length, and supply the maximum values of E_c and justify the use of a linear stress-strain relationship for the concrete in compression.

Answer:

Both Section 10.2 of ACI 318-71 and Section 2610 of UBC-76 allow for certain assumptions in determination of the ultimate strengths of members subjected to flexure and axial loads, when the members satisfy the applicable conditions of equilibrium and compatibility of strains. The referenced sections of the above codes provide that, for strains in reinforcement greater than those corresponding to the specified yield strength, f_y, the stress in the reinforcement shall be considered independent of strain and equal to f_y. The codes, therefore, impose a limitation on the stress in the reinforcement and do not have any restriction on the strain level. The maximum usable strain in the concrete at the extreme compression fiber, however, shall be limited to 0.003. Therefore a ductility ratio, used for purposes of illustration in PGE-1020, is consistent with code provisions provided the maximum concrete strain is less than 0.003.

The three dimensional finite element analysis of the Complex is performed considering the Complex as an assemblage of linear elastic systems of structural elements having stiffness indicated by test results. The purpose of the analysis is to determine the loads on the various structural components based upon their relative rigidities when the analysis satisfies conditions of equilibrium and compatibility. The capacities of the structural elements are evaluated based on the "strength method" and compared with their demand loads. This procedure is generally followed in reinforced concrete design and analysis and is consistent with code provisions. Because a non-linear cracked analysis is not performed on the entire Complex and also because the individual structural components - viz, concrete, reinforcing steel, embedded structural steel - are not modeled explicitly, the present analysis does not determine the level of strain in those components.

A displacement computation has, however, been made for a 31'-0" x 15'-6" x 24" panel, having E_c equal to 3.92×10^6 psi. With the assumption of double curvature and a ductility in the outer rebar as 4, the horizontal displacement is found to be approximately 0.05 in.

As explained in response to Question No. 44, using the stiffnesses indicated by the test results and considering

478

100

the displacement compatibility of the entire structure, displacements associated with factored OBE loads will increase by a factor of approximately 4.3 over those associated with the unfactored OBE loads. Based on the above, the computed ultimate interstorey displacement can be estimated to be between 0.03 in. and 0.04 in. for a typical panel of comparable dimensions as above. Also, the response to Question No. 43 indicates that the double curvature assumption provides for conservative evaluation of capacity and a more flexible structure is expected. A more realistic assessment of the behavior of the walls will provide for a displacement closer to the ones above.

The relationship between the concrete stress distribution and the concrete strain, according to the code provisions, may be assumed to be a rectangle, trapezoid, parabola, or other shape which results in prediction of strength in substantial agreement with the results of comprehensive tests. Many researchers in the past have obtained good correlation with experimental results by assuming a triangular stress distribution. Reference may be made to the paper, "Strength of High-Rise Shear Walls - Rectangular CrossSection" by Cardenas and Magura in publication No. SP-36 of ACI. The theoretical flexural equation described in Section 3.4.2.2 of PGE-1020 has been analyzed by computer for a 24inch thick wall by assuming a value for the modulus of elasticity of concrete, E_c, equal to 3.92×10^6 psi, as given by the procedure described in Appendix B of PGE-1020. In calculating E_c, the compressive strength of concrete is taken as 5000 psi. The results of this analysis showed that both for the reinforced and unreinforced cores and for a range of vertical stress as exists in Complex walls, the calculated values of shear stress capacity could be obtained within a bound of ±5% if the assumption was made that the compressive zone was 10% of the wall's effective length. Hence the asumption of 10% is made to simplify the flexural equation and facilitate hand calculation without any loss of accuracy. The analysis also showed that the maximum concrete strain was 0.0016. For strains in this range, stress-strain relationships may be considered linear.

Provide the basis for your calculation for the block and the beam to column connection capacities. Include a discussion of the strain compatibility of the two, and the basis for the 100 psi allowable vertical shear on the block at corners which seems to include a 1/3 increase in UBC allowable stresses which would not be appropriate nor in line with current practice.

Answer:

Section 3.5 of PGE-1020 describes the procedure followed to examine the mechanism of vertical shear transfer from sidewalls to end walls of the Complex. An alternative approach is discussed below where the capacity of the corners of the walls to transfer the vertical shear is shown to be a combination of the contribution from the beam-column connection and the shear friction developed by the continuous horizontal reinforcing steel in the concrete block masonry.

In order to determine the ability of the beam-column connection to transfer the vertical shear, it is necessary to consider both the ultimate strength and the load-deformation characteristics. These aspects will be examined in light of the failure plane envisaged and the type of connection.

The beam-column connections of the structural framing system embedded in the Complex walls have been designed as simple bearing type connections according to the working stress method of AISC. They consist of connection angles bolted to the column and the web of the beam. In order for a failure to develop at the boundary between a wall panel and an embedded steel column, a crack plane would have to form through the joint, thus generating a potential for a vertical slip between the connection angle and the column.

If it is conservatively assumed that no interaction take place between the wall panel and the embedded column, this potential vertical slip will be resisted by two mechanisms:

1) The resistance of the beam-column connection

2) The shear friction and dowel action developed by the horizontal reinforcing steel of the concrete block masonry crossing the crack plane.

In order for the effects of the two resistance mechanisms to be additive, both the beam-column connection and the shear-friction mechanisms must carry their ultimate loads at a comparable value of slip.

1. Consistency of deformation

a. Beam-column connection

The experimental load-slip data for the steel connections are obtained from References 1 and 2.

The load-slip curve for a six-rivet joint loaded in tension presented by Davis and Woodruff in Ref. 1 indicates that the ultimate load for the connection is attained in the range of 0.02 inch to 0.03 inch. The load-slip characteristic by Bendigo et al., in Ref. 2 has shown the following:

"It should be noted that riveted joints often experience slip despite the customary assumption that hot driven rivets fill the holes completely. In these tests, slip amounted to approximately 0.02 inch, which is about one-third to one-quarter of the slip experienced by the bolted joints".

The conclusion is, therefore, that the bolted connection will attain a slip at ultimate load equal to 0.06 inch to 0.08 inch. Applying the "one-third to one-quarter" relationship to the Davis and Woodruff results would, however, indicate an ultimate slip of about 0.06 inch to 0.12 inch. A reasonable value of slip at ultimate load for high strength bolt in a simple connection is considered to be 0.08 inch.

b. Shear-friction of reinforcing steel

Survey of the existing literature has not provided any experimental data on load-slip characteristic of masonry block with in-fill concrete.

However, results of tests reported in Ref. 3 can be used to predict the behavior.

The concrete strengths of the test specimens used to develop the load-slip curves in Ref. 3 varied from 3930 psi to 4090 psi. These strengths are comparable to the strength of the concrete filled masonry blocks of the Complex walls. The paper shows that the maximum shear resistance for all specimens with prepared joints reached their peak at a slip between about 0.02 inch and 0.03 inch. However, at a slip between 0.08 inch and 0.10 inch, the resistance was still maintained without any appreciable loss in the capacity. The extent of the slip is further supported by the shear wall testing program of Appendix A, PGE-1020. Specimen D, which had no core concrete, exhibited a slip of about 0.10 inch and the specimen maintained the shear resisting mechanism.

It can, therefore, be concluded that the shear resistance mechanisms, as provided by the beam-column connections of the fully embedded structural framing system and the shear-friction of the continuous horizontal reinforcing steel in the monolithic concrete block masonry units, will develop their ultimate resistance at about a slip of 0.08 inch.

2. Ultimate Resistance

a. Beam-column connection

In order for the vertical shear crack plane to form through the connection, the ultimate strength of the connection will be governed by the shear strength of the A-325X bolts and by the bearing on A-36 material under the bolts. Concrete encasement of the fully embedded joints will preclude any prying action induced by bending moment. This suggests that the evidence derived from experiments on lap joints and butt splices should apply to this type of connection also, with the exception that the tensile failure on the net section experienced by butt and lap splices would not be expected, due to the arrangement of the connected pieces.

Wallaert and Fisher (Ref. 4) gave a conservative estimate of ultimate shear strength of A325 bolts as 75 ksi. Since AISC specifications permit a design shear stress of 22 ksi, this indicates a 75/22 = 3.41 factor of safety with respect to ultimate. The factor of safety for bearing may be obtained as follows:

Jones (Ref. 5) has shown that the tensile strength of lap and butt joints would not be impaired if the nominal bearing stress was less than 2.25 times the nominal tensile stress on the net section. This ratio has been adopted by AISC, where the allowable bearing stress is restricted to $1.35f_y$. It implies, therefore, that the factor of safety for bearing is at least equal to the factor of safety for tension on the net section of the connected materials. The tests performed on the structural steel material used in the Complex indicated an ultimate tensile strength of approximately 62 ksi. Taking the AISC allowable tensile stress of $0.60f_y$ = 21.6 ksi on the net section will, therefore, provide a factor of safety for tension of 62/21.6 = 2.87. A factor of 2.8 will, therefore, be taken as the factor of safety against bearing type connection designed by AISC working stress methods. This value is conservative since the type of tension failure which the bearing stress criteria is intended to prevent would not occur in a beam-column joint in the Complex walls.

b. Shear friction of reinforcing steel

Section 11.5 of the ACI Building Code, ACI 318-77, allows design for shear transfer to be based on the "shear friction" hypothesis proposed by Birkeland (Ref. 6) and Mast (Ref. 7). In this approach, it is assumed that for some unspecified reason a crack exists in the shear plane. The shear resistance is then assumed to be developed entirely by the frictional resistance to sliding of one crack face over the other, when acted on by a normal force equal to the yield strength of the reinforcement crossing the shear plane. A value of the apparent coefficient of friction, μ, is used to qualify this behavior. For a crack in monolithic construction, μ is taken as 1.4. This value, as suggested by ACI 318-77, provides a conservative estimate of the shear transfer strength of concrete cracked along the shear plane (See Ref. 8). Pauley et al. (Ref. 3) obtained values of the apparent coefficient of friction for specially prepared rough surfaces equal to 3.4, 2.0 and 1.6 for reinforcement ratios of 0.31, 0.69 and 1.23 respectively. As can be seen, for this well prepared joint, the value of μ increases as reinforcement ratios decrease. Results of tests in Ref. 3 also indicate that, especially for specimens with smaller reinforcement percentage, the capacities as given by shear-friction were maintained during a large number of alternating load cycles. In the Complex walls, the continuous horizontal reinforcing steel in the concrete block masonry constitutes a percentage of 0.12, and therefore, based on the above data, an apparent coefficient of friction, μ, can be taken as 1.4.

~~461~~ 478

109

The ultimate shear resistance will, therefore, be taken as

$V_u = 2.8 \times$ (Beam connection value of Table I, AISC)
$+ (1.4 \times A_s \times f_y \times h)$

where,

A_s = Area of continuous horizontal reinforcing steel in masonry (in^2/ft)

f_y = Yield strength of reinforcing steel = 40 ksi

h = Height of wall (ft.)

The vertical shear resistance corresponding to the unfactored OBE condition is obtained by dividing the ultimate capacity by the load factor of 1.4 after reducing the contribution of shear-friction of reinforcing steel by the capacity reduction factor of 0.85.

Based on the above, the values given in Section 3.5 of PGE-1020, will be revised as follows:

Corner	Vertical Shear Force (kips)	Capacity (kips)
R-55	2357	2742
N-55	1260	1763

Q. 16.

References for Response No. 16

1. Davis, R.E., and Woodruff, G. B., "Tension Tests of Large Riveted Joints", Transactions of the ASCE, 1940

2. Bendigo, R.A., Hansen, R.M. and Rumpf, J.L., "Long Bolted Joints", Journal of the Structural Division, ASCE, December, 1963

3. Pauley, T., Park, R., and Phillips, M.H., "Horizontal Construction Joints in Cast-in-Place Reinforced Concrete", ACI Publication No. SP42-27

4. Wallaert, J.J. and Fisher, J.W., "Shear Strength of High-Strength Bolts", Journal of the Structural Division, ASCE, June, 1965

5. Jones, J., "Bearing Ratio Effect on Strength of Riveted Joints", Transactions of the ASCE, November, 1956

6. Birkeland, P.W. and Birkeland, H.W. "Connections in Precast Concrete Construction", Journal of the American Concrete Institute, March, 1966

7. Mast, R.F., "Auxiliary Reinforcement in Concrete Connections", Proceedings, ASCE, June, 1968

8. Mattock, A.H. and Hawkins, N.M., "Shear Transfer in Reinforced Concrete-Recent Research", PCI Journal, March/April, 1972

Discuss in detail the effects on the in-plane wall shear capacity of any tension induced in the walls by the gross overturning moments and the "plate bending" of the walls generated by the earthquake component perpendicular to these walls.

Answer:

In general, the tension forces developed from seismic responses on one side of a structure due to gross bending are offset or balanced by an equal amount of compression on the other side of the structure. On a smaller scale, this type of behavior developed in the test specimens and is reflected in the observed capacities. The criteria for capacity as given in PGE-1020 is linear with respect to axial load, therefore, no major influence on the capacity is expected from tension induced by gross bending. This agrees with current shear wall design practice since there is no special consideration for this effect.

The plate bending effect in the shear walls which are carrying the primary shear loads is due to the component of earthquake perpendicular to the component causing the primary shear. The Trojan FSAR does not require the effects of the two horizontal components to be combined; but, if it were considered, the effect would be small. Taking into account that the peak response due to two horizontal components of the earthquake do not occur simultaneously, the effects can be combined using the recommendation of Newmark[1] which is" -- take the combined effects as 100 percent of the effect in one particular direction and 40 percent of the effects corresponding to the two directions of motion at right angles to the principal motion considered". Under these conditions, the plate bending effect is small. If 40 percent of the transverse inertia loads were considered as a static load, the load could be resisted by the vertical reinforcing steel in the block of the composite walls only without yielding. Considering the ultimate case of the effect of the full longitudinal shear force combined with the dead load, some vertical reinforcing steel may yield. With the imposition of 40 percent of the transverse inertia loads and considering the transient nature of the load, the load could cause slight additional yielding as the energy associated with the transverse inertia loads is absorbed. The amount of additional yielding can be bounded by considering the deformation during a static application of the loads. As indicated above, the load can be resisted with the vertical block reinforcing steel only and the steel does not yield. The strain energy in the steel for this loading condition is less than one half the yield strain times the yield stress ($0.5\ e_y f_y$) since the strain energy is the area under the stress-strain diagram and the stress increases linearly with strain up to the yield strain. The additional strain that will develop is that required to develop enough strain energy to balance that from the transverse inertia loads. If the reinforcing steel in the zone at the wall under consideration is at yield or beyond, the strain in the reinforcing steel increases without an increase in stress. Since the area under the stress-strain diagram is the strain energy and the energy to be absorbed is less than 0.5 $e_y f_y$, the additional strain would be only one half the yield strain. This amount of additional strain will not reduce the in-plane shear capacities.

Reference

1. Newmark, N. M., W. J. Hall, Comments on Inelastic Seismic Capacity of Nuclear Reactor Structures, Civil Engineers and Nuclear Power, Vol 2, ASCE Convention, Boston, April 2-6, 1979.

POOR ORIGINAL

Describe in detail the modifications necessary to ensure the seismic qualification of the complex as a result of the strengthening or stiffening of the structure and sequence in which they will be performed.

Answer:

The review of safety-related equipment, components, and piping within the Complex has been made in accordance with Appendix B of PGE-1020. The review of the safety-related equipment and instrumentation has not been completed. However, results to date indicate that no modifications are required by the revised response spectra. The results of this review to date have revealed that some modifications to the cable tray supports and piping supports are required.

The cable tray supports which require modification to remain seismically qualified to the revised response spectra are indicated on the attached Table 29-1. The modifications of these cable tray supports will be made prior to the structural modifications to the Complex.

The piping system supports which require modifications are indicated in Table 29-2. This modification work will be performed in a sequence for those piping systems required for safe-shutdown, ECCS, or to mitigate mitigate consequences of accidents which could result in releases exceeding 10CFR100 guideline limits, the change to the support to meet SSE

478
~~461~~ 115

requirements will be implemented prior to the structural modification that necessitated it. These items are identified by asterisk in Table 29-2.

Evaluations have been performed to confirm that all safety related piping, cable trays and equipment attached to non-shear walls in the Complex would retain their support capabilities when subjected to a seismic event. The walls in question are identified in Table 29-3. Evaluations have confirmed that the stress levels in these walls are low and that non-linear behaviour which could reduce this support function will not occur.

The safety related equipment, cabling and piping dependent upon these walls for support are identified in Table 29-4. Their support location and configurations have been confirmed by plant survey.

TABLE 29-1

CABLE TRAY SUPPORTS REQUIRING MODIFICATION

Building	Elevation	Support Number
Auxiliary	77' Area 3	17,18,21,22,23,24,35
Control	61' Area 6	60
	77' Area 6	119,121,122
	93' Area 13	7,13,15

POOR ORIGINAL

478 ~~461~~

TABLE 29-2

PIPING SUPPORTS TO BE ADDED

Large Pipe

10"	GCB-9-2	SR-300	*
3"	CS-151R-9-3	SR-300	
4"	CS-151R-12-2	SR-301	
4"	CS-151R-26-1	SR-300	
4"	HBD-91-4	SR-300	
4"	HBD-91-4	SR-301	
	HCC-62-1	SR-300	
	HCD-2-7	SA-300	
4"	RC-151R-1-2	SR-302	
4"	RC-151R-1-2	SR-303	
3"	RC-151R-19-1	SR-300	
8"	HBD-91-2	SR-302	
3"	HCD-19-1	SR-300	
10"	GCB-7-1	SS-300	*
10"	GCB-9-1	SS-304	*
12"	HBD-28-2	SS-301	*
6"	HBD-33-1	SS-302	*
6"	HBD-33-2	SS-303	*
4"	HCC-23-2	SS-300	*
	HBD-22-3	SR-301	*
	HBD-22-3	SR-302	*
4"	SI-1501R-1-1	SR-301	*
3"	SI-1501R-1-1	SR-302	*
10"	HCC-49-1	SR-300	*
6"	HFD-3-6	SR-300	*
4"	CS-151R-6-1	SS-300	
4"	CS-151R-6-1	SS-301	
4"	CS-151R-6-1	SS-302	
4"	CS-151R-6-3	SS-300	
4"	CS-151R-6-3	SS-301	
3"	CS-151R-9-2	SS-300	
3"	CS-151R-9-2	SS-301	
3"	CS-151R-9-2	SS-302	
3"	RC-151R-19-1	SS-301	
3"	HCC-29-3	SS-300	

Small Pipe (2 1/2">O.D)

3/4"	HKD-1-53	SR-317	*
1-1/2"	HKD-1-58	SR-322	*
1-1/2"	HKD-1-60	SR-320	*
3/4"	HKD-1-60	SR-325	*
2"	HKD-1-69	SR-318	*
2"	HKD-1-69	SR-319	*
3/4"	HKD-1-69	SR-321	*
1"	HKD-1-76	SR-323	*
1"	HKD-1-76	SR-324	*
2"	HKD-2-64	SR-303	*
2"	HKD-2-64	SR-304	*
1"	HKD-1-69	SR-320	*

*Installation required prior to modifications strengthening the Complex

TABLE 29-2

PIPING SUPPORTS TO BE ADDED

10"	HCC-48-1	SR-301	1"	JBD-31-61	SA-309
10"	HCC-49-2	SR-300	1-1/2"	JBD-31-67	SA-312
3"	HCC-65-2	SR-300	1"	JBD-31-70	SA-311
4"	JBD-35-1	SA-301	1"	JBD-31-96	SA-310
4"	JBD-36-1	SA-300			
3"	HCB- 3-1	SR-300			
10"	HCC-48-1	SR-302			
8"	HCC-46-1	SR-303			
10"	HCC-48-1	SR-304			
	HCC-48-4	SR-300			

ATTACHMENT

TABLE 29-2

PIPING SUPPORTS TO BE ADDED

Large Pipe			Small Pipe (2 1/2">O.D)
3" CS-151R-30-1	H-27	*	
3" CS-151R-30-1	H-28	*	
3" CS-2501R-28-2	SR-78	*	
3" CS-2501R-28-2	SR-81	*	
8" RH-601R-7-1	SA-19	*	
10" GCB-9-2	SR-31	*	
10" GCB-9-2	SR-41	*	
12" HBD-28-1	SA-243	*	
12" HBD-30-1	SA-241	*	
14" HBD-31-2	SA-807	*	
4" HCC-23-2	SA-129	*	
8" CS-151R-5-1	SR-26	*	
14" SI-151R-10-1	SR-76	*	
10" GCB-9-2	SR-38	*	
24" HBD-27-3	SR-300	*	
24" HBD-27-4	SR-815	*	
14" HBD-31-2	SR-111	*	
6" HBD-33-1	SR-193	*	
4" CS-151R-6-1	SR-17		
4" CS-151R-6-3	SR-24		
3" CS-151R-9-1	SR-97		
3" CS-151R-9-3	SR-87		
14" SI-151R-10-1	SA-81		
3" CS-151R-12-2	SR-157		
4" CS-151R-12-3	SR-110		
4" CS-151R-12-3	SR-111		
4" CS-151R-12-3	SR-112		
4" CS-151R-12-3	SR-113		
4" CS-151R-12-3	SR-117		
4" CS-151R-12-5	SA-149	*	
3" CS-151R-16-1	SA-159		

*Support modification required prior to modifications strengthening the Complex

ATTACHMENT

TABLE 29-2

PIPING SUPPORTS TO BE MODIFIED

Large Pipe	Small Pipe (2 1/2">O.D)
4" CS-151R-26-1 SR-41	
4" CS-151R-26-1 SR-155	
3" CS-2501R-5-4 SR-75	
14" SI-601R-5-1 SR-41	
4" SI-2501R-3-3 SR-46	
4" SI 2501R-3-3 SR-50	
8" GBD-18-4 SA-1	
8" GBD-18-4 SA-6	
8" HBD-91-3 SR-70	
8" HBD-91-3 SR-73	
8" HBD-91-3 SA-72	
8" HBD-91-3 SA-75	
3" HCC-27-1 H-2	
3" HCC-27-1 H-4	
3" HCC-27-1 H-6	
3" HCC-39-3 SA-156	
3" HCC-62-1 SR-23	
4" CS-151R-5-3 SR-11	
3" CS-151R-9-3 SR-83	
4" CS-151R-12-3 SR-107	
4" CS-151R-12-3 SR-115	
3" CS-151R-12-5 SR-138	
3" CS-2501R-5-4 SR-78	
8" HBD-91-1 SR-53	
8" HBD-91-1 SR-54	
8" HBD-91-1 SR-69	
3" HCC-12-2 SR-27	
3" HCC-62-1 SR-1	
12" RH-601R-7-1 SR-63 *	
4" SI-151R-10-7 SR-2 *	
4" SI-1501R-1-1 SR-65 *	
4" SI-1501R-1-1 SR-69 *	
10" GCB-9-1 H-21 *	

* Support modification required prior to modifications strengthening the Complex

ATTACHMENT

TABLE 29-2

PIPING SUPPORTS TO BE MODIFIED

Large Pipe		Small Pipe (2 1/2">0.D)
6" HBD-33-2	SR-105*	
14" HBD-34-3	SA-242*	
6" HFD-2-2	SR-68 *	
12" RH-601R-7-1	SR-62 *	
4" SI-151R-10-7	SR-5 *	
14" HBD-27-1	SR-225*	
14" HBD-27-1	SR-227*	
30" HFD-1-1	SR-216*	
30" HFD-1-1	SR-217*	
6" HFD-2-6	SR-75 *	
6" HFD-3-6	SR-15 *	
10" HCC-48-1	SS-84	
8" GCB-7-1	SR-61	
4" HBE-11-1	SR-6	
10" HCC-48-2	SR-70	
10" HCC-49-2	SA-103	
3" HKD-2-1	SR-60	
8" GCB-7-1	SR-63	
10" HCC-48-1	SR-37	
10" HCC-48-2	SR-82	
10" HCC-48-2	SR-86	

*Support modification required prior to modifications strengthening the Complex

TABLE 29-3

LIST OF NON-SHEAR WALLS

Building	Floor Elevation	Wall Location
Control Bldg.	65'-77'	Walls surrounding the Battery room
	61'-77'	Col. line 51 E-W
	61'-77'	Col. line 53 E-W (check)
	61'-77'	West of N
	77'-93'	South of 51 E-W and N-S walls
	77'-93'	Computer room North and East walls
	93'-105'	Wall on 51 N-S
	105'-117'	Wall on 51 E-W
		Wall on "0" N-S
Auxiliary Bldg.	45'-61'	Wall H between 54 and 55 N-S
	45'-61'	East of line H all the 8'-0" high small walls
	61'-77'	West of line E between 60 and 61 N-S
	77'-93'	Line E and West of Eenclosing Valve Compartment; 8'-0" high walls

TABLE 29-4

SAFETY-RELATED COMPONENTS ON NON-SHEAR WALLS

Area/ Elevation	Piping or Restraint	Conduit	Pull Box	Control Panel	Others
Auxiliary/45'	SI-151R-10 HCC-71-50	BB469X BB470X AB468X BB434X BB435X BB436X BB490X AB462X BB4153	BPB457 APB442 BI-4908 BI-4909	None	Tray support BIR 121 BBR 411
Auxiliary/61'	CS-151R-6-3 SI-601R-9-50 HCB-9-1 CS-601R-4-51 SI-601R-9	BB-4107 BB-4050 AB-4992 AB-4969	None	None	Valve leak-off line for for MOV-8809B
Auxiliary/77'	CS-2501R-28-50 CS-2501R-28-51 CS-151R-9-58 CS-151R-9-4				
Control/61'	HKD-2-51	AB-1011 BB-1010 CB-1012 BI-1033 BI-1034 BI-1035 BI-1036	BPB-108 BPB-116 BPB-109	C-182 C-181 C-180 L-12 L-28 Q-23 D-62 L-05 L-27 C-262 L-29 D-09	Room cooler supply & return (V-145B & C)

TABLE 29-4
(Continued)

SAFETY-RELATED COMPONENTS ON NON-SHEAR WALLS

Area/ Elevation	Piping or Restraint	Conduit	Pull Box	Control Panel	Others
Battery room	None	BP-1013 BP-1091 BB-1072 BB-1185 BB-1187 AB-1021 BB-1072 BB-1186 AP-1005	None	None	Space Heater Circuit breaker Q-28 Q-22
Control/77'	None	AB-1011 AB-1045 BB-1138 AI-1044 AI-1050 AI-1051 BI-1051 AI-1057 AI-1044 AB-1045 AB-1069 BB-1091	APB-136 CPB-165	C-249 C-243 C-179	
Control/93'		DI-1901 DI-1902 BB-1168 BB-1022 AB-1081 AB-1017	CPB-158 DPB-182		1 Tray CIA-207 Duct work
Control/105'	HKD-1-53		APBV-09 BPBV-09 APB-125 BTB-104 APBV-17 BPBV-17 ABV-007 BBV-007	C-254 C-255 C-178 C-259-1 C-259-2 C-260-1 C-260-2	

478
~~461~~

Summarize the details of your evaluations which determined that placement of the reinforcing steel, the forms and the concrete will not significantly degrade the seismic capability of the Complex. Include a definition of significant.

Answer:

The evaluation considered the effect on the existing Complex of the forms, reinforcement and the placing of concrete. The columns that will be exposed during the modification work were investigated and they were found to be capable of resisting the loads induced by the fluid concrete in combination with an earthquake, in addition to the loads in the columns due to dead load, live load and earthquake loads. The existing block walls were investigated for the effects of fluid concrete in combination with an earthquake and were found to be adequate.

The 3-inch thick plate will be used as the outside form where new concrete is placed on the R-line wall up to Elevation 76'-3". Before concrete is placed, the bolts will be installed through the plate and the existing block walls. They will be tightened, but not tensioned. The bolts will prevent the steel plate from moving during concrete placement and in the event of an earthquake. The existing block walls are adequate to withstand the loads induced in them by the plate and uncured concrete during an earthquake.

Another investigation considered the increase in strength of the new walls as a function of time and the associated wall loads and capacities. The loads on the new concrete walls were obtained by proportioning the total shear on the wall between the new and existing concrete according to their relative stiffnesses. The stiffnesses and shear capacities of the new walls at various time intervals were based on the increase of f'_c with time. Upon comparing the shear capacity of the new walls with their load demand, it was found that the capacity always exceeded the demand.

Q. 37. (a)

Provide the correlation, wall for wall, between the test specimens and the actual walls, and justification for the applicability of the test specimen results to the actual wall including a discussion of the similarities of such items as reinforcing steel ratio and continuity, encasement, material strengths, joint preparation (especially where drypack was used), etc.

Answer:

The results from a particular test specimen were not applied to a specific wall. In the case of wall capacity, the test specimens formed the basis for the use of analytical flexure equation but the results from the test specimens were not used directly. The applicability of the flexure equation is discussed in response to Question No. 43. In the case of wall stiffness, the results from a group of test specimens, which represent the range of conditions existing in the actual walls, were used to develop the non-dimensional stiffness reduction factors. The applicability of this information is discussed in response to Question No. 46.

Q. 37. (b) Page 1 of 2 pages

With regard to the drypack, refer to the article by Kahn and Hanson entitled, "Infilled Walls for Earthquake Strengthening" in the February 1979 ASCE Journal of the Structural Division. This article describes a "brittle" failure of a test specimen with a drypack joint. Discuss the implications of this with respect to the walls in the Trojan complex with the drypack joints and the applicability of the test results from specimens without drypack joints.

Answer:

The article in question was based upon a thesis report of the same title written at the University of Michigan under NSF Grant No. GI-39123. A review of this report indicates that there are significant differences between the drypack zones in the Michigan test specimens and the drypack zones in the Complex walls.

The following differences were noted:

1. The dimensions of the drypack are quite different. In the Michigan test specimen the horizontal drypack region is 3 inches high and only 3 inches thick with the vertical reinforcing bars running down the centerline. This arrangement, with little cover over the reinforcing steel, would seem to promote spalling of the drypack (after the drypack-concrete bond is broken), with the reinforcing steel acting as wedges to cause longitudinal splitting and eventual spalling. In the Complex the drypack regions are at least 14 inches thick with two rows of vertical rebars. These two rows are external to

~~461~~ 129

478

a large portion of the drypack, and would act to confine the drypack, and inhibit spalling. Spalling would also be inhibited by the greater concrete cover in the Complex walls (approx. 4" vs 1-1/2"). The delay of spalling would delay the deterioration of the shear friction mechanism.

2. The Michigan test specimens were deliberately designed as heavily reinforced shear walls inside a non-ductile frame. The "brittle" failure described by the authors appears to be due to the interaction of the infill panel and the frame. Because of the relative stiffnesses, the columns of the Michigan test specimens carry little shear until the shear friction transfer between the drypack and surrounding concrete has been extensively degraded. At this point the shear load is largely transferred to the columns, which then fail in shear in a brittle fashion (since they were designed to be non-ductile). This situation would not be present in the Complex walls.

The major walls in the Complex do not have any drypack.

Describe in detail how the constant bending moment applied to the test specimens via the auxiliary loading system in conjunction with the main loading system compares to that which would exist due to end restraint in the actual Trojan walls, to justify the applicability to the test specimens results directly to the actual walls.

Answer:

In the shear wall testing program the test specimens without steel struts or embedded steel columns had free vertical boundaries. This condition, therefore, was not an exact representation of the actual Complex wall panels where the behavior of a panel is dependent upon its interaction with the adjacent panels. Since it was not possible to simulate the interaction effect in the testing program, the test specimens were subjected to a loading condition where the auxiliary loading system in conjunction with the main loading system moved the point of contraflexure near the mid height of the specimen. As will be shown in the following analysis, the shear capacity of the wall panels obtained from this test set up is a conservative assessment of the actual panel capacity.

In developing the shear capacity of individual wall panels by application of the flexural analysis equation, credit was taken only for the fully embedded vertical reinforcing steel, which provided moment resistance at top and bottom of the panel. The vertical faces of the panel were considered to be totally free. In actual Complex walls a significant amount of vertical shear resistance is generated at the panel vertical interfaces. Moreover, no credit was taken for the bond between the embedded steel columns and the surrounding concrete core, and no interaction has been assumed to have taken place at this interface. The vertical shear resistance is given solely by the continuous horizontal block reinforcing steel through the mechanism of shear friction and the beam-column connection activated by the panel when the panel tends to rotate and pushes against the beam flange.

The following analysis evaluates the shear capacities of wall panels by considering the vertical shear resistance at the side boundaries and a conservative assumption of the single curvature cantilever action of the panel. In evaluating the vertical resistance of horizontal block rebars, the coefficient of friction, μ, will be taken as 1.4 for the continuous masonry construction. The ultimate strength of the beam-column connection will be taken as 2.8 times the working stress capacities given in Table I of AISC for ASTM A325-X bolts in bearing. The factor of safety of 2.8 and the apparent coefficient of friction of 1.4 are established in response to Question No. 16. The compatibility of deformation between masonry and the slip in the beam-column connection is also discussed in the same response.

Figure 43-1 below shows the free body diagram of a typical wall panel.

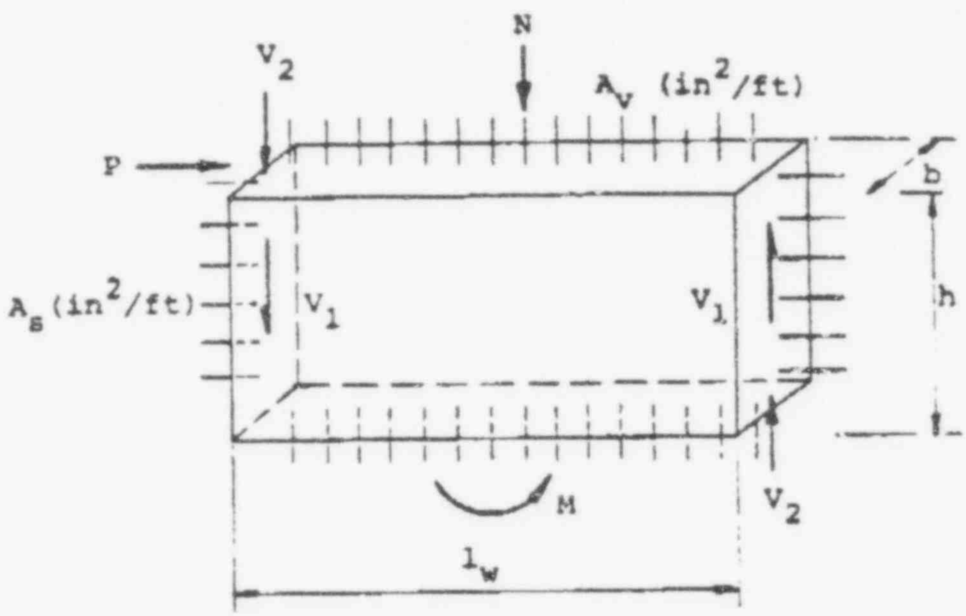


Figure 43-1

A_s = Horizontal continuous block reinforcing steel
A_v = Vertical continuous reinforcing steel
For other definitions see Section 3.4.2.2 of PGE-1020
Vertical shear resistance $V = V_1 + V_2$

V_1: Shear friction developed by block reinforcing steel, 4-#5 @ 24" o.c., typically

A_s = 0.62 in^2/foot height of wall

$V_1 = \mu . A_s . f_y$

$= (1.4 \times 0.62 \times 40 \times h)/12$

= 2.9 h kips, where h is in inches

~~461~~ 478 133

V_2: Shear resistance through beam-column connection: For illustration purpose a typical floor beam, W24 x 68 will be considered. The allowable working stress bearing values of the connection from AISC table I-B7 = 126 kips

$$V_2 = 2.8 \times 126$$
$$= 353 \text{ kips}$$

The moment resistance given by the continuous vertical reinforcing steel and vertical load N (See Section 3.4.2.2 of PGE-1020) is:

$$M = 0.465\ A_s f_y l_w^2 + 0.467\ N l_w$$

Assuming single curvature,

$$P.h = (V_1 + V_2)\ l_w + M \quad \text{, or}$$

$$P = \frac{l_w}{h}(2{,}900\ h + 353{,}000) + \left(0.465\ A_s f_y \frac{l_w^2}{h}\right) + \left(0.467\ N \frac{l_w}{h}\right)$$

$$\tau = \frac{P}{b l_w}$$

$$\tau = \frac{2{,}900}{b} + \frac{353{,}000}{bh} + \frac{l_w}{h}(0.465\ \rho_v f_y + 0.467\ \sigma_n)$$

Where τ is the ultimate shear stress in psi. Figure 43-2 shows the values of shear stress given by the above formula for values of ρ_v equal to .0021 and .0012 respectively for various values of σ_n and for a panel which is 31 ft. long, 16 ft. high and 27 inches wide.

The shear stress as calculated by the formula given in Section 3.4.2.2 of PGE-1020, assuming double curvature, is also plotted for comparison. As can be seen from the figure, the shear stresses based on the double curvature assumption are very low for a range of σ_n between 0 and 100 psi when compared to those obtained by assuming single curvature and shear resistance along vertical faces. Since all Complex walls are within that range of vertical stress it can be concluded that application of the double curvature principle provides a conservative assessment of the shear capacity of the Complex walls.

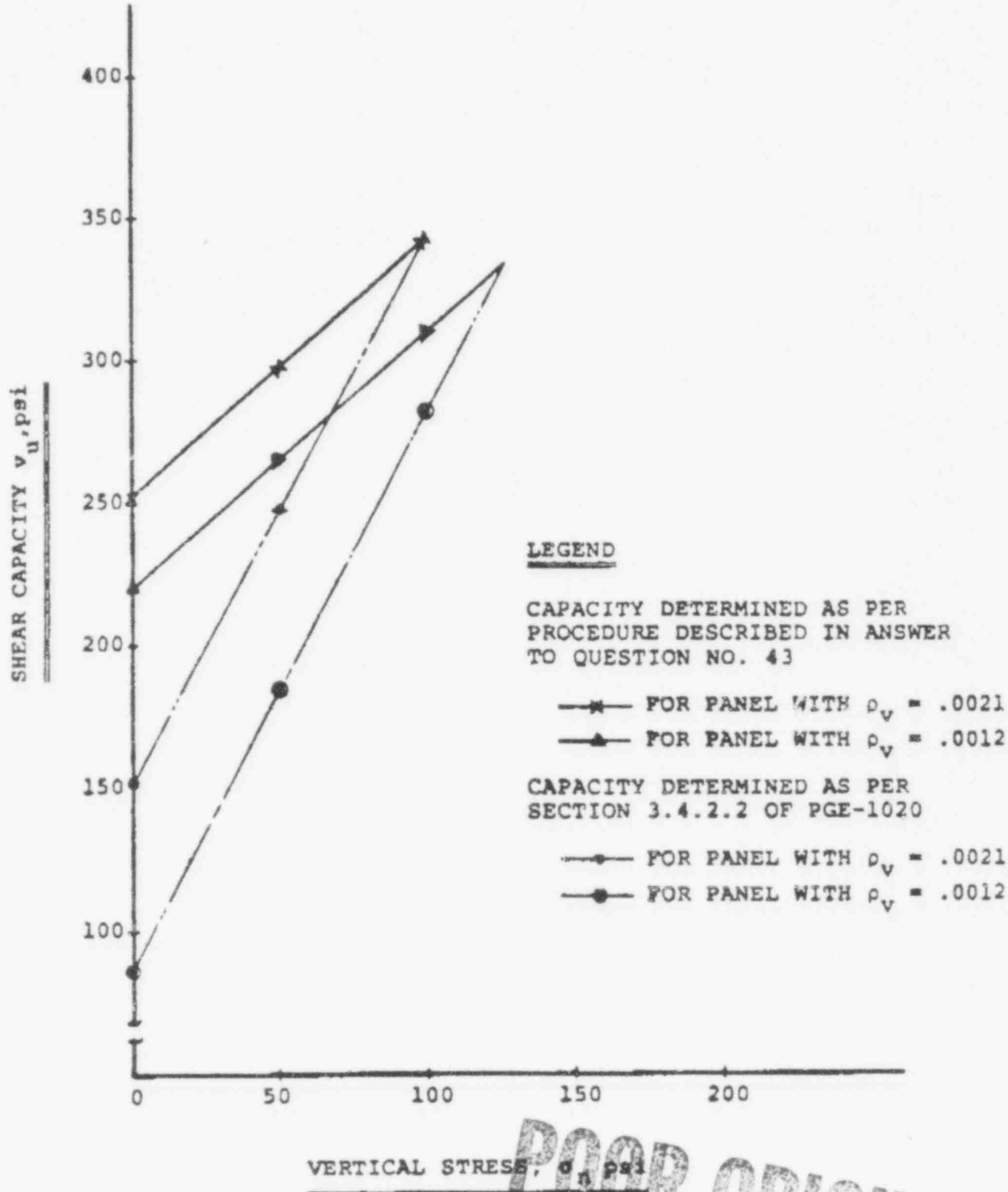

400
350
300
250
200
150
100
SHEAR CAPACITY v_u, psi
0
50
100
150
200
VERTICAL STRESS, σ_n psi
LEGEND
CAPACITY DETERMINED AS PER PROCEDURE DESCRIBED IN ANSWER TO QUESTION NO. 43
FOR PANEL WITH ρ_v = .0021
FOR PANEL WITH ρ_v = .0012
CAPACITY DETERMINED AS PER SECTION 3.4.2.2 OF PGE-1020
FOR PANEL WITH ρ_v = .0021
FOR PANEL WITH ρ_v = .0012

FIGURE 43-2

Q. 45.

Considering the strength of the column connections for the actual walls, demonstrate that they are capable of resisting the axial forces indicated by those results for the columns in specimens L1 and L2. Justify any exceedances of the beam/column connection capacity.

Answer:

The response to Question No. 39 explains that the specimens L1 and L2 were tested to obtain a knowledge of the behavior of the Complex walls where the structural steel columns are continuous through the floors and where bond is assumed to exist between the embedded columns and the surrounding concrete. The stiffness obtained from these test data was used in the STARDYNE analysis since an upper bound stiffness provided for a more critical condition for the floor response spectra without causing significant changes in the level of shear forces in the Complex walls. The shear resistances of the specimens L1 and L2, however, were not used for evaluation of the shear capacity of the walls. Furthermore, as explained in response to Question No. 43, even if the bond and interaction between the steel columns and the core concrete is conservatively ignored, the beam-column connection resistance combined with the shear-frictior provided by the continuous horizontal reinforcing steel in the masonry concrete will generate capacities for the walls which [illegible]e considerably higher than those considered in Section 3. [illegible]2.2 of PGE-1020 in the range of dead load that exists in the Complex walls.

POOR ORIGINAL

~~461~~ 137
478

Provide the detailed bases for each of the variations assumed in Table B-2 in the calculations of the peak broadening percentage.

Answer:

As indicated in PGE-1020, the variation in frequency is due to variation in the mass and stiffness. During the development of the response spectra for interim operation, the variation in mass was estimated to be $\pm 5\%$. Since the weight of the new structural elements being added is small and known to at least this accurracy, the variation in total weight will still be taken at $\pm 5\%$. The variation in the initial modulus results from variation in the properties of the reinforced concrete, concrete block, steel plate, etc. Again, the variation in the properties of the new materials are known as well as those of the existing structure. Therefore the variation used here is the same as for interim operation.

The variation in the stiffness reduction factors are due to variations in the shear stress, dead load and experimental uncertainties. The significance of variations in these parameters can be put into prospective by estimating the frequency if no stiffness reduction was used. By using various intermediate results in the stiffness reduction iteration process and approximate calculations, the frequency of the uncracked structure for the fundamental N-S mode is estimated to be 8.1 cps. When the stiffness reduction factors are used, the frequency of the same mode as determined in the

STARDYNE analysis is 7.6 cps, indicating a 6 percent reduction. Considering the accuracy of the calculations, the expected reduction should be in the 5 to 7 percent range. This provides a useful upper limit on the variation in frequency due to the various parameters.

The variation in the shear stress is also based on the intermediate results from the stiffness reduction iteration process. From the wall which shows the maximum variation in shear force as the stiffness reduced factors converged, the shear force was 3150 kips for the uncracked stiffness. It reduced to 2350 kips on the first iteration and then became 2960 kips and 3000 kips on successive iterations. This provides confidence that the shear force is known to ±200 kips or 7%. Other walls showed less variation. This indicates the ±10% variation used in PGE-1020 is conservative.

The variation in the dead load results from two effects. First is the variation of the actual weight of the structure. The 5 percent variation used for the mass is appropriate for this consideration. The second source of variation is the load path. Since the majority of the dead load in the walls is due to self-weight and not the floor system, the load path is very simple, straight down the wall, and can be calculated to a variation of less than 5 percent. The load path for the other weights - equipment and non-structural walls - is known well enough so that ±20 percent used in PGE-1020 is conservative.

As indicated in Table B-2 of PGE-1020, the variation in the total structural stiffness is ±15 percent resulting in frequency shift at ±7.2 percent. This frequency shift is greater than the estimated frequency reduction, 5 to 7 percent, due to the inclusion of the stiffness reduction factors. This 5 to 7 percent frequency shift corresponds to an average reduction in stiffness of 12 percent, $(0.94)^2 = 0.88$. Since most structural elements have a stiffness reduction factor of 0.85 or higher, the ±15 percent variation in stiffness reduction factor will accommodate a 100 percent increase in the amount of stiffness reduction. A smaller variation is indicated by the shear stress-deflection curves shown in Figures A2-2 and A2-3 of PGE-1020. Some of the variations among these specimens are due to differing steel ratios. If this variation was eliminated as is done for the stiffness reduction factor relationship shown in Figure B9, B10 and B11, the variation would be smaller. These two groups of specimens are the only ones with similar enough properties to provide a meaningful comparison. This consistency of results and the small amount at overall stiffness reduction indicates the ±15 percent variation in the stiffness reduction factor due to experimental uncertainties is adequate.